Pile analysis with numerical method of finite difference in soil with liquefaction potential

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Abstract

In recent years, in spite of the remarkable advances in soil mechanics and geotechnical engineering in recent decades, it is still difficult to determine the load capacity of piles. Unique physical and mechanical properties of soil such as heterogeneity, water, various soil compositions in nature, creep potential and the complicated behavior of stress-strain in one hand and the variety of piles in terms of materials, sections, methods of construction and erection, etc. on the other lead to complex interaction of the structural elements of the pile and the surrounding soil. The current research aims to investigate and analyze the behavior of piles in liquefied soil with special emphasis on compressive load bearing capacity. The earthquake stresses in sandy areas increase the pore pressure, reduce soil resistance and ultimately create a liquefaction state in the soil that would have destructive and costly effects on deep foundations. For this purpose, using computer modeling, a two-dimensional model of piles and soil was designed in static conditions and analyzed using a finite difference method in the software. The results were compared with the theoretical methods such as Meierhov, Vesik and jumbo β, α and λ methods. Then the axial load bearing capacity of the pile was studied in soil liquefaction conditions and the force-settlement of pile was investigated. In modelling, the pile behavior was considered in accordance with elastic model, the soil behavior in static case was considered with Mohr-Coulomb model, and the dynamic case (liquefaction) was assessed using the Finn model. Axial force reduction mechanism was used to distribute the axial force across the pile due to shear strength of the soil and pile displacement respect to surrounding soil and load transfer from the pile to the soil. The results of this study indicate that the displacement of the pile helmet in dynamic and liquefaction conditions of soil is approximately 15 to 20 times the displacement in static state. Moreover, the axial load capacity of the pile in the soil liquefaction state has decreased to about 50% of the load capacity in the static state. Parametric studies have shown that the frequency of earthquakes, the thickness of the soil layer and the groundwater level are factors that affect the axial load capacity and the behavior of the piles in susceptible liquefied soils.

Keywords: Pile, Liquefied Soils, Axial Load Capacity, Finite Difference, FLAC 2D, Shear Strength

1 Introduction

The proper definition of liquefaction has always been the subject of debate among geotechnical experts. Soil liquefaction and related collapses that is commonly associated with large earthquakes, is called reduction of the shear strength of saturated granular soils due to the formation and increase of pore water pressure during dynamic loading. In a more precise definition, liquefaction is a phenomenon in which a mass of soil that is subjected to uniform, successive or sudden loading, loses a great deal of its shear strength and flows in a fluid state until the applied shear stresses is reduced equal to shear resistance. In a more general view, variation the soil state from solid to liquid, as a result of increasing pore water pressure and effective stress reduction, is called liquefaction. The seismic design of a pile in liquefied soils creates complex problems in the analysis and design. In soils with a liquefaction potential, even before the soil is in the full liquefaction state and soil stiffness is minimum, the pile may be exposed to considerable shakes. During the shaking, pile is susceptible to any damage, including the expansion of cracks, joint formation and ultimately failure.

2 Liquefaction phenomenon

Soil liquefaction is one of the phenomena in nature that causes serious damages. This phenomenon is considered as a major factor of rupture in saturated silt sand sediments. It was not recognized until 1953 and then a similar phenomenon was by Mugami and Kubo, and the name of liquefaction was attributed to it as an important engineering issue. Nigata earthquake in June 1964 with 7.5 magnitude was another milestone in this field. Nigata that confined by a great fire in 1995, was reformed and renovated as a result of extensive development work in the urban area and it was known as a new city with modern equipment and facilities. Therefore, the earthquake in 1964 imposed a huge blow to the city and caused unprecedented destruction. The Nigata earthquake can be traced as a symbol of the first incident in the world that destroyed all of the lifelines of the city. The main reason for the failure of this earthquake was soil liquefaction. For this reason, the issues of liquefaction attracted a lot of attention among the geotechnical community. Since the Nigata earthquake in 1964, many seismic geotechnical engineers tried to standardize the liquefaction mechanism and provide recommendations and guidelines for liquefaction based on field and laboratory data. In Iran, one of the reasons for the massive destruction of the Manjil earthquake in the morning of June 31, 1991, especially in the Astaneh Ashrafieh and its suburbs, was land liquefaction. Liquefaction is a complex phenomenon that is not fully understood. In engineering projects, the most reasonable and most appropriate way is needed to encounter with liquefaction in the reclamation areas. In sandy soils, sand particles are stored by particle bonding and force can be transferred through these connections. These causes shear stress and the structure on the surface of the ground is carried by the soil. When the sand soil is deformed by shear stress due to vibration, the connection between the particles decreases. As a result, the force that is generally carried vertically and among the connection points becomes the pore water pressure. This is related to liquefaction. After liquefaction, the connection between soil particles is completely restored, and this happens when the pore water pressure is destroyed. Liquefaction can be a dipping of the building on the ground, either tilted considerably or buried structures floats in the soil and closes to the surface of the earth. Slippage and acceleration in rupture in dams and slopes lateral displacement of foundations are related to liquefaction and causes the failure of structures, roads, etc. (Mir Hosseini, 2014).

3 Methodology

3.1 Pile and Soil modelling

The tested soil is in three layers with a thickness of 20m, which the elastic coefficient of upper level with a thickness of 3m is 31930 KPa, the elastic coefficient of middle layer with thickness of 7m is 30352 kPa, and the elastic coefficient of bottom layer is 32478 KPa. Density and other required parameters for Mohr-Coulomb model that are from geotechnical studies are given in Table (1). The diameter of the pile is 80 cm and its length is 15 m, and the elastic coefficient of the pile is equal to 2.5 $\times 10^7$ KPa. Other pile characteristics for the elastic behavior model is also given in Table (2). The shear strength parameters the pile and model are equal to half the resistance properties of the soil (with Mohr-Coulomb model).

Table 2- required parameters for different soil layers in Mohr-Coulomb pattern (Permayon Company).

permeability K	Porosity n	Density Gs	Cohesion KPa	Internal friction angle (degrees)	Shear modulus KPa	Bulk modulus KPa	Modulus of elasticity KPa	Bulk density $\frac{KN}{m^3}$	hight m
1×10^{-7}	0/43	2/68	10	35	12310	26207	31930	19/20	0-3
1×10^{-6}	0/37	2/65	0	40	12000	21500	30352	17/30	3-10
1×10^{-5}	0/5	2/68	10	35	12480	27230	32478	21/10	10-20

Table 3- required parameters of pile (Permayon Compnay)

diameter m	Length m	Shear modulus KPa	Bulk modulus KPa	Modulus of elasticity KPa	Density $\frac{KN}{m^3}$
0/8	15/0	1×10^7	$1/422\times10^7$	$2/5\times10^7$	25

and surrounding soil are included in the model configuration that is shown in figure 4-1. In the meshing design, it is tried to make the zones with square geometries because, if the geometry of the zones is irregular, due to a large node displacement, the software will send an error message.

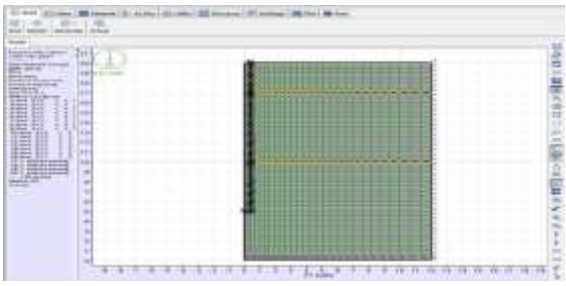

Figure 2 Model meshing in the software before load application

4 Analysis

When a pile is subjected to compressive loading, the load-displacement curve is in the form of figure 2. The upper curve is the variation of applied load on the pile tip based on pile tip displacement and other two curves show frictional load and tip load according to the displacement of the pile tip. See the diagrams of forces equivalence in figure 3 to observe this behavior. When the compressive force is applied to the pile tip, the downward displacement of the pile starts, which mobilizes the shear strength of the soil. This process causes the load transfer from the pile to surrounding soil, so the transferred axial compressive force to pile tip is very small that is shown in figure 3. According to the point A on the curve of Fig. 2. With the increase of load on the corresponding pile head to point B on curve 2, the frictional resistance of the pile is mobilized in the entire length of the pile and its value is maximized. From this stage, the pile tip directly carries any increase in load at the pile head. When the applied load on the pile head reaches to its maximum value, the pile tip resistance corresponding with point C on figure 2 is fully mobilized. The solid curve in Figure 3 shows the load transfer curve in the pile at the final resistance. In addition, the force equilibrium of the model is shown under different loads in the software in figure 3.

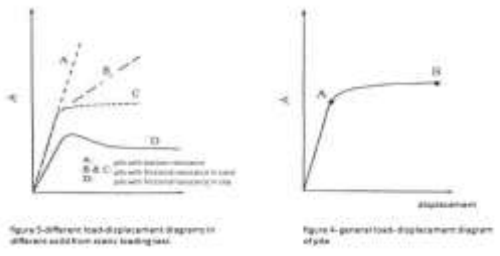

figure 3- general load-displacement diagrams for pile under compressive axial loading (deep pile, Amel Sakhi, p. 35).

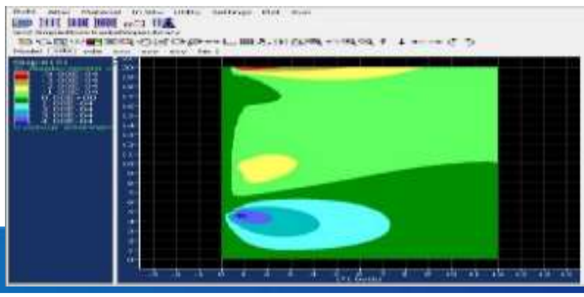

Figure 4- contour of horizontal displacement under compressive axial loading of 2500 k N

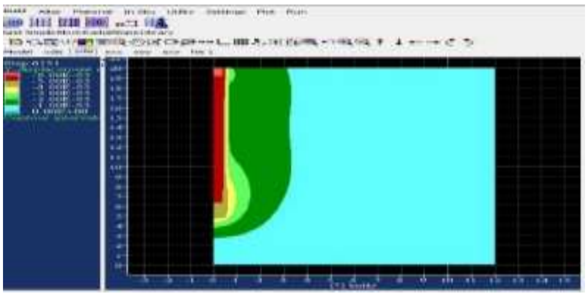

Figure 5- contour of vertical displacement under compressive axial loading of 2500 k N

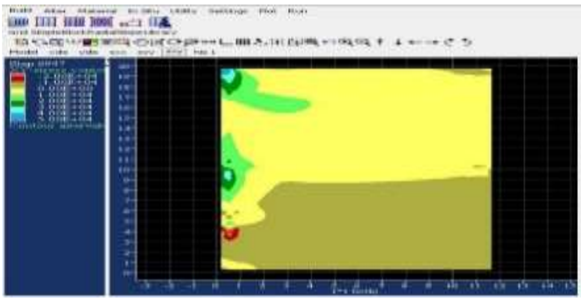

Figure 6- contour of shear stress under compressive axial loading of 2500 k N

These figures show a few very important points about the piles subjected to pressure. First, the load- displacement curve is nonlinear. Second, the total frictional resistance is mobilized by the relatively small displacement of the pile head. Third, the total tip resistance is mobilized by the significant displacement of pile head. This analysis was used in the next sections of this research.

5 Displacement of modelled pile

Model and the software indicate that the displacement of different points of the pile and the surrounding soil for different loads is determined from 1500 to 2500 kN that are shown in figs7. These figures indicate that the pile element is rigid body and has same displacement, but the soil below the pile has different stress contours. Therefore, by moving away from the pile tip, the stress bubble decreases below the tip. However, at higher loading levels, this stress bubble is better seen at pile tip, which is due to the increase in the relative displacement of the pile tip and soil at higher loading levels and mobilization of pile tip strength.

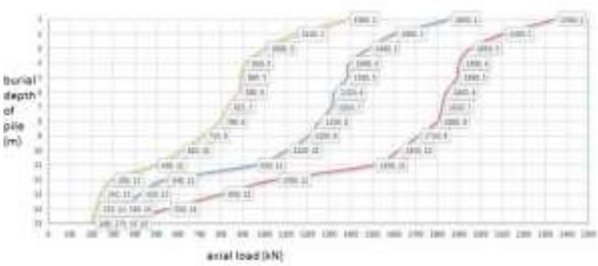

Figure 7- comparison of axial force variation diagrams under the compressive axial load from 1500 to 2500 kN.

6 Axial force variation and frictional resistance across the pile

By applying the static compressive axial force in a stepwise manner, the displacement of the pile is increased respect to the surrounding soil, which also activates the shear strength of the soil. This process causes the transfer of load from the pile to the surrounding soil, and therefore the axial compressive force of the pile decreases with increasing depth. It is shown in figures 8 and show the pile behavior under the axial compressive static load. In figure, by applying a load of 2500 KN to a pile, the pile settlement increases respect to the surrounding soil and it mobilizes the shear strength of the soil, which reduces the axial force by moving away from the pile tip. For example, at 5m depth, the axial force of the pile is about 1890 kN and at the end of the pile, axial force reaches to 325 k N that 2755 k N of reduced force of the axial force that is applied to pile tip, is the share of frictional resistance of pill wall and the remaining 325 k N applied axial load is the share of pile tip resistance. The frictional resistance of the pile wall at low loading level (about 500-1000 kN) and at the low displacement of the pile relative to the soil, reaches its maximum value. At loading levels above 2000 kN, only a slight change occurs in the distribution of the frictional resistance of the wall across the pile but its value remains constant. From the above discussion, it is concluded that the frictional resistance in the pile is created by a relatively smaller relative displacement compared to the pile tip.

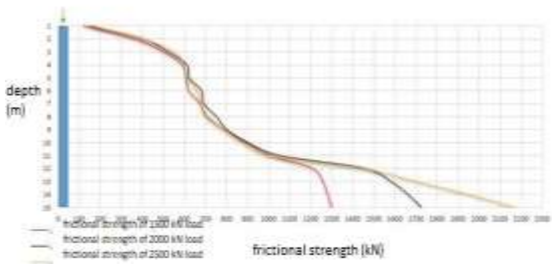

Figure 8- comparison diagrams of frictional resistance variations for1500- 2500 k N axial loads

7 load- displacement graph of the pile

The variation of the axial force along the pile is presented in Fig9. This graph shows that the numerical model has reached total capacity of 8999 kN, but its trend is still increasing. Of course, it should be noted that all the capacity for piles are obtained with safety factor. In the preliminary stages of design, the value of safety factor is chosen between 2.5 and 3. Nevertheless, in the later stages of the project that more information was available through static or dynamic experiments, due to the behavior of the soil at the level of the pile and because of high number of standard penetration tests in close proximity to the rocky layer and the necessary tests for joint conditions in this layer, therefore, the safety factor 4 is considered to determine the bearing capacity of the tip of the pile and for the pile walls, the safety factor value is 3.

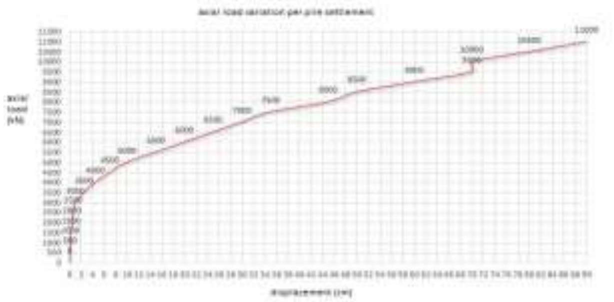

Figure 9- the diagram of compressive axial force variation per pile settlement in static analysis

CONCLUSION

According to numerical modelling by finite difference software in this study, the summary of the results is as follow:

1. Because of soil settlement and compaction due to earthquake and soil liquefaction, the settlement of soil and pile is much higher than static state (up to 20 times) at the same level.
2. The axial load distribution varies across the pile respect to static state and there is no friction between the soil and the pile in the liquefied soil that has no shear strength. As a result, the frictional resistance of the pile wall is closes to zero. Therefore, there should be no frictional resistance to the pile in liquefied layer.
3. The total load bearing capacity of the whole pile has been reduced to about half in respect to static state. It can be concluded that by applying the safety factor 3 in the static state, it is possible to compensate the reduction of load bearing capacity due to soil liquefaction, so that the soil liquefaction does not damage the pile due to lower load bearing capacity. It should be noted, however, that the results of the static load carrying capacity were developed with safety factor one, and it is necessary to apply another safety factor apart from the safety factor 2 that was derived from this analysis. In other words, by applying the safety factor 3 to the total static load bearing capacity, the load bearing capacity is equal to the total load capacity in the soil liquefaction state.
4. With the increase of the thickness of the upper non-liquefied layer, the time to reach the final settlement of the pile is increased.
5. In models with high porosity ratios, pore water pressure is higher and liquefaction is more intense and pile settlement in models that have more porosity ratio, is higher and faster than the models with less porosity ratio.
6. With the increase of the thickness of non-liquefied layer, the pore water pressure is less assuming that the applied acceleration of the model is constant.
7. The increase of pore water pressure is more intense during the first cycles and is decreases soil resistance significantly.
8. Dissipation and deprecation of pore water pressure in the thicker non-liquefied layer in the sample, requires more time duration.
9. In models with high porosity, the settlement rate is very fast, and the amount of settlement is higher than models that are made with lower porosity.
10. In the thicknesses of more than 20 cm for non-liquefied layers in numerical models, the effects were almost the same and the variations of the settlement value reach to their lowest value.
11. The reduction in excess pore water pressure depends on peak ground acceleration (PGA) and the type of non-liquefied soil.
12. The amount of optimum thickness of non-liquefied layer can be appreciably less in gravel layer than in the loose sand layer.
13. At higher depths, the increase of the thickness of the liquefied layer does not have much effect on the load bearing capacity.
14. In the constant thickness and by increasing the liquefied layer depth, load-bearing capacity is doubled.

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