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Numerical modeling to evaluate the effect of soil liquefaction on the bearing capacity of piles

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ABSTRACT

The performance of piles in liquefied soils is much more complex than the performance of piles in non-liquefied soils; Because in this case, in addition to the fact that the pile is subjected to different dynamic loads, both from the structure and from the soil, the strength and hardness of the soil decreases over time due to nonlinear behavior of the soil and increasing pore water pressure. And several numerical analyzes have been done in this field, the mechanism of interaction of the structure-soil pile set during liquefaction has not been determined yet. Thus, the presented study attempted to provide the numerical modeling to understand the piles behavior in liquefied soils. In this regard, the finite element codes by Plaxis software was use to simulate and extract the deformation status for piles constructed on deep liquefied soils. According to the prepared modeling successfully used for deformation, liquefaction, and piles performance analyses.

1. Introduction

Usually due to earthquake stimulation and shear stress in the soil, semi-dense and loose saturated masses of soil, especially layers of clean sand and silty sand or even silt tend to compact and settle (Leung et al., 2010). As a result of this tendency for the grains to condense and move in the empty space between the particles that are filled with water between the cavities, additional pressure is created in the cavity water. In most cases, due to the short time of the earthquake and the low permeability coefficient of this type of soil, water does not have the opportunity to drain and the water pressure of the holes created in each loading cycle is collected by the overpressure created in the next cycle (Xu et al., 2021). Eventually the pore water pressure rises so high that the contact between the soil grains disappears and the effective stress between the solid soil grains becomes zero. Under such conditions, the soil completely loses its shear strength and acts as a viscous fluid with a density equal to the specific gravity of soil saturation, and large deformations occur in the soil (Tang et al., 2021).

Soils that are subject to liquefaction are usually classified in the category of non-sticky soils, which have the ability to liquefy, respectively: clean sands, silty sands with low dough properties, are non-plastic siltes and sands (Azarafza and Asghari-Kaljahi, 2016). Cohesive soils are usually not at risk of liquefaction. However, in cases where adherent clay soils are prone to liquefaction, all of the following criteria must be met. In other words, if any of the following criteria are not met, the sticky clay soil will not be liquefied (Abdoun and Dobry, 2002):

- The weight percentage of fine grain (soil dry weight) less than 0.005 mm, less than 15%.
- Psychological limit (LL) is less than 35%.

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- The moisture content of the studied soil is more than 0.9 of the psychological limit of the studied soil.

Occurrence of liquefaction causes damages such as loss of bearing capacity of foundations, soil subsidence and compaction of liquefied layers, boiling of sand, protrusion from buried massive structures and most importantly the phenomenon of deformation or lateral expansion (Zhan-Fang et al., 2021). The theoretical methods and behavioral models application to predict the excess water pressure in the soil and perform effective stress analysis with the help of software developed in this field is in the stages of research and development. They can play an important role in completing the mentioned experimental methods and predicting the behavior of adjacent structures and liquefied layers during earthquakes and the interaction of structure and soil (Kheradi et al., 2019).

Due to soil vibration during an earthquake, periodic shear stress τ_h is applied to the soil component. Therefore, any laboratory test to study the liquefaction phenomenon should be arranged in such a way as to create the conditions of a constant vertical stress and a cyclic shear stress on a plate of the soil sample (Mokhtar et al., 2014). So far, various laboratory methods have been developed, for example, the following can be mentioned (Ebeido et al., 2019):

- Cyclic triaxial test,
- Simple cyclic shear test,
- Vibration-table test.

However, the most common of these methods are cyclic triaxial testing and simple cyclic shear testing. Laboratory tests have shown that the number of loading cycles required for the occurrence of flow liquefaction decreases with increasing shear stress range and decreasing relative density. Whereas liquefaction rupture in loose specimens will occur with only a small number of cycles of large shear stress. Thousands of low-amplitude cycles are required to create liquefaction failures in dense specimens. The relationship between density, cyclic stress amplitude, and number of cycles that cause liquefaction can be plotted graphically with laboratory "cyclic resistance curves". Cyclic resistance curves often because the effective overhead pressure to be normalized to produce a "cyclic stress ratio (CSR)" must be defined differently for different CSR tests (Tasiopoulou et al., 2013). It is also possible to calculate the cyclic stress required for liquefaction (liquefaction resistance) by sampling from the desired location and performing laboratory tests, cyclic stress (Berrill and Yasuda, 2002). However, sampling of granular soils and transferring them to the laboratory causes a lot of soil damage. Special sampling methods such as ground freezing can also be used, which are also costly. Therefore, it is preferable to use in-site tests. These tests include standard penetration test (SPT), cone penetration test (CPT), shear wave velocity (V_s), and Becker penetration test (BPT). Each of these methods depends on various factors such as the equipment available, the condition of the test site, costs, and so on (Ghorbani et al., 2020).

Table 1. Advantages/disadvantages of liquefaction's field methods

Specification	Tests			
	SPT	CPT	V_s	BPT
Prevalence of experiments	A lot	A lot	A few	Rarely
Stress-strain behavior type	Large strain	Large strain	Small strain	Large strain
Quality and reproducibility	Poor to good	Very good	Good	Weak
Identify changes in soil layers	Good	Very good	Medium	Medium
Recommended for soils	Not gravel	Not gravel	All kinds	Gravel
Provide samples	Yes	No	No	No
Measurement of engineering properties or index	Indexes	Indexes	Engineering properties	Indexes

Table 1 summarizes the advantages and disadvantages of each of these methods. The basis of methods based on in situ tests is a large amount of experimental data collected from various sites that have been exposed to earthquake stimulation and loading, and some of them have been liquefied. And some have not been liquefied. Detection of liquefaction in construction sites is based on physical observations made from the site, which include sand boiling, heterogeneous settlements after the earthquake, cracks formed on the soil surface, or lateral expansion (Knappett and Madabhushi, 2006).

Sensitivity to the occurrence of phenomena such as soil liquefaction and more accurate analysis of the situation on the site in terms of their occurrence has led researchers to use computer-based methods to evaluate and increase the dimensions of studies. Many researchers today have used numerical methods to determine the nature of liquefaction.

Numerical methods include various methods such as finite element method (FEM), discrete element method (DEM), boundary element method (BEM), and etc. (Li and Motamed, 2017). Among them, the finite element method (FEM) has a good efficiency in soil environment studies due to its algorithmic assumptions that are used for analysis in continuous environments. By definition, soil is a continuous and homogeneous environment (Njock et al., 2020). Considering the behavior of soils as elastic-linear plastics and the validity of the Mohr-Coulomb failure criterion, the mechanical behavior of soils against the applied forces and local stresses of different structures can be determined by numerical methods such as finite element analyze method (Esfeh and Kaynia, 2019).

2. Materials and Methods

2.1. Fundamentals of Liquefaction Analysis by Plaxis

Plaxis software is a two-dimensional computer analysis program for analysis of stability, deformation, subsidence, compaction, consolidation and leakage under static and dynamic conditions in the field of geotechnics (Toloza Barría, 2018). Numerical aspects of liquefaction under dynamic soil conditions can be analyzed and performed by finite element methods and finite element codes by Plaxis by considering some assumptions (Abdelmonem and Osman, 2017). In dynamic analyzes by Plaxis, three important conditions for dynamic boundary conditions, elastic viscous boundaries, degree of lattice, and degrees of spatial freedom must be considered and met (Zardari et al., 2017). Boundary conditions and allocation of behavioral criteria are the most important part in modeling soil structures under liquefaction conditions (Toloza Barría, 2018). In analyzes of finite element methods, the most basic equation for time-dependent motions under dynamic loads such as earthquakes is expressed as Eq. 1 (Barrueto et al., 2017).

$$F= M \cdot \ddot{u} + C \cdot \mathring{u} + K \cdot u \tag{1}$$

In this regard, the values of M, C and K are the mass matrix, damping matrix and stiffness matrix, respectively, and the vector F is introduced as the dynamic vector of forces (Barrueto et al., 2017). Since the wave in nature is damped as it progresses in the environment and its energy decreases, so this phenomenon should also be considered for dynamic analysis. Plaxis software uses Riley damping or local damping to determine the ambient attenuation, the mathematical model of which is chosen so that the energy consumed in numerical calculations is similar to the energy consumed in the physical system. For rail damping, it is necessary to first define the rail damping value in the natural frequency range by determining the natural frequency value of the prepared model. According to this method, the damping matrix C as a linear relationship between the mass matrix M and the stiffness matrix K can be written as follows (Toloza Barría, 2018):

$$C = \alpha_R M + \beta_R K \tag{2}$$

The α_R and R_β are called R Rayleigh damping coefficients. Eq. 3 is solved and implemented based on Newmark integration plan in Plaxis software as follows (Barrueto et al., 2017).

$$\left[c_0 M + c_1 C + K\right]\Delta u = F_{ext}^{t+\Delta t} + M\left(c_2 \dot{u}^t + c_3 \ddot{u}^t\right) + C\left(c_4 \dot{u}^t + c_5 \ddot{u}^t\right) - F_{\text{int}}^t \tag{3}$$

In this regard, the values c_0 to c_5 of the Newmark coefficients related to the parameters of the Newmark function in time units are introduced.

2.2. Numerical modeling by Plaxis

In order to achieve a correct modeling of the conditions prevailing in the mass, in this dissertation, most of the parameters considered in the analysis of soil liquefaction are proposed and applied in the model. Therefore, the model is prepared and implemented in four stages: geometric modeling of the mass, boundary conditions, assignment of properties and definition of behavioral models, and mechanical modeling under seismic conditions. The following is a brief description of the modeling process (Toloza Barría, 2018):

Geometric modeling: Dealing with soils containing problematic materials is always natural and possible in geotechnical engineering. Soils are widely used as materials in design and construction. In other words, it can be said that geotechnical structures are made and executed from soil, with soil and in soil. Therefore, the existence of problematic soil (which in this study is liquefied soil) is an inescapable possibility. In order to cover the problem and investigate in the most critical possible conditions, we tried to collect information about the collision and importance of liquefied soils and based on statistical analysis and the normal distribution function on the data and normalize them the most important The type of collision should be considered as the basis of geometric modeling in this study (Abdelmonem and Osman, 2017). The parameters in modeling a pile as a concrete pile are in situ in saturated sandy soils and prone to liquefaction under dynamic load. The geometric model of the liquefied embankment is shown in Fig. 1.

The reduction in the amount of computational error under the time of earthquake which is usually applies the maximum time range of vibration and vibration in the model (Mohamed et al., 2020). This type of boundary condition is used in one-dimensional wave propagation analysis and is able to absorb propagated waves from internal sources (reflective waves as a result of dynamic loading, drilling and explosion). This boundary condition is as follows (Li and Motamed, 2017):

Viscous boundaries: Viscous boundaries can be used as Neumann boundary type where boundaries are intended to prevent reflection.

Free boundaries: This type of boundary condition, as its name implies, allows movement and displacement for the mass in the lateral boundaries. It can be appropriate to use this boundary to determine the status of stimulus stresses and particle mobility in soils.

The results of the implementation of boundary conditions in the model prepared are shown in Fig. 2.

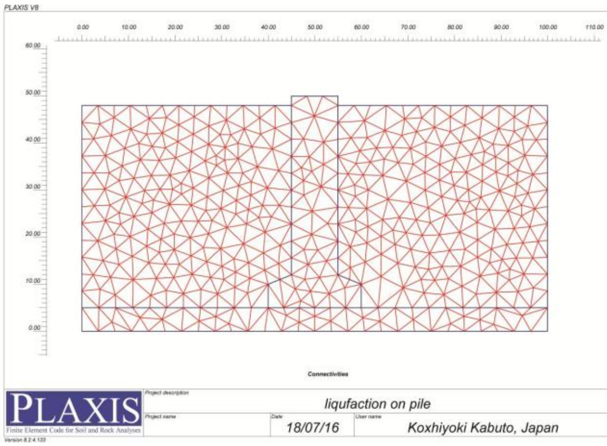

Figure 1. Geometric model prepared in this study

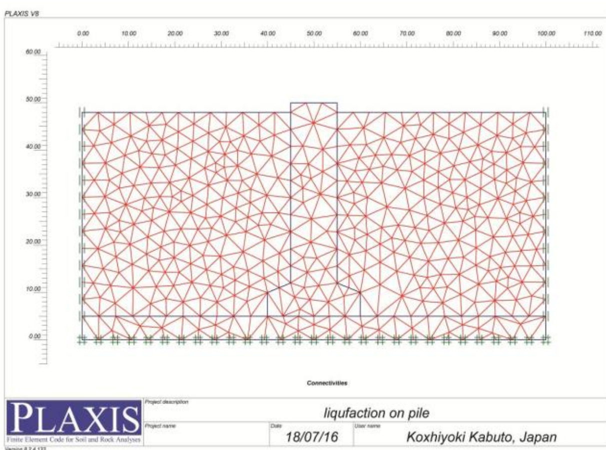

Figure 2. Model prepared from the embankment boundary conditions under study (free and viscous boundaries)

Assignment of materials properties and behavioral models: In order to determine the behavioral properties and to determine the behavioral model for the model of material body selection based on the range of liquefaction range in soils is considered. Soils that are subject to liquefaction are usually classified in the category of non-sticky soils, which have the ability to liquefy, respectively: clean sands, silty sands with low dough properties, non-plastic siltes and sands (Tang et al., 2021). Therefore, it is quite natural to import materials with the properties of this category of soils to analyze the fluidity of the soil in the model under the dynamic conditions of the earthquake. In the present modeling, a concrete pile in a liquefied soil mass is expressed on a resistant substrate. Therefore, we have three materials, the first material is related to the material of the pile, which is concrete, and the geotechnical parameters of concrete are discussed. The second material is related to the parameters in liquefaction-prone soils and the third material is related to bedrock, to which the relevant materials are assigned. Table 2 is proved the input parameters for the model and in Fig. 3 is given the models's properties.

The behavioral model used in this study is the Mohr-Coulomb elastoplastic model. This behavioral model based on rupture cap under normal and shear stresses, makes it possible to analyze rupture in both tensile and compressive (Li and Motamed, 2017). After preparing these materials, the model ready to run and estimate the liquefaction effects on piles.

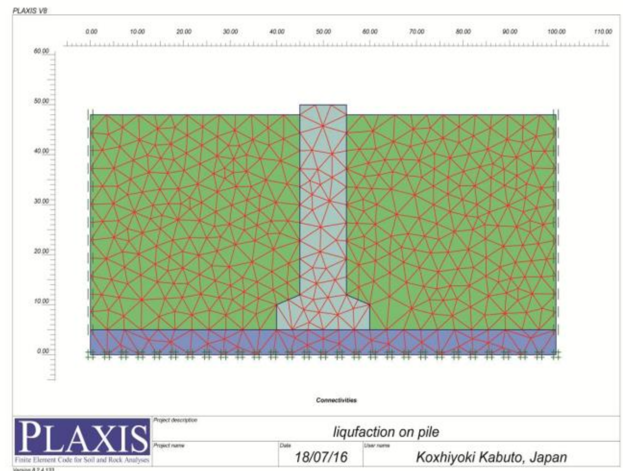

Figure 3. Model prepared after assigning materials properties

Table 2. Input parameters for the model

Section	Parameters	Unit	Value
Concrete pile	γ_{unsat}	kN/m^3	19.00
	γ_{sat}	kN/m^3	19.00
	E_{ref}	kN/m^2	10000000
	G_{ref}	kN/m^2	4167000
	E_{oed}	kN/m^2	11110000
	υ	-	0.2
	C_{ref}	kN/m^2	714
	ϕ	Degree	54.9
	ψ	Degree	0.00
Liquefaction soil	γ_{unsat}	kN/m^3	17.20
	γ_{sat}	kN/m^3	20.00
	E_{ref}	kN/m^2	24000
	G_{ref}	kN/m^2	8888.889
	E_{oed}	kN/m^2	38520
	υ	-	0.35
	C_{ref}	kN/m^2	1.00
	ϕ	Degree	33
	ψ	Degree	0.00
Bed Rock	γ_{unsat}	kN/m^3	18.20
	γ_{sat}	kN/m^3	19.00
	E_{ref}	kN/m^2	5000000
	G_{ref}	kN/m^2	1923000
	E_{oed}	kN/m^2	6731000
	υ	-	0.3
	C_{ref}	kN/m^2	5000
	ϕ	Degree	35
	ψ	Degree	0.00

3. Results and Discussions

After geometric modeling, determination of boundary conditions and assignment of properties and behavioral model to the model, the model is considered and solved under the conditions. In this research, the model under dynamic forces as seismic force that is widely applied in the earth body and soil changes in the pile foundation area is investigated. The results of this evaluation are presented as a mechanical model and the results are used to interpret the prevailing conditions. Mechanical modeling is performed on the modeled embankment and the results are as illustrated in Figs. 4 to 11.

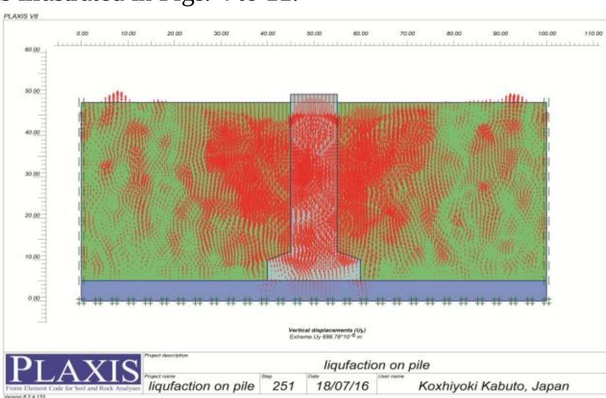

Figure 4. The displacement state during the liquefaction event

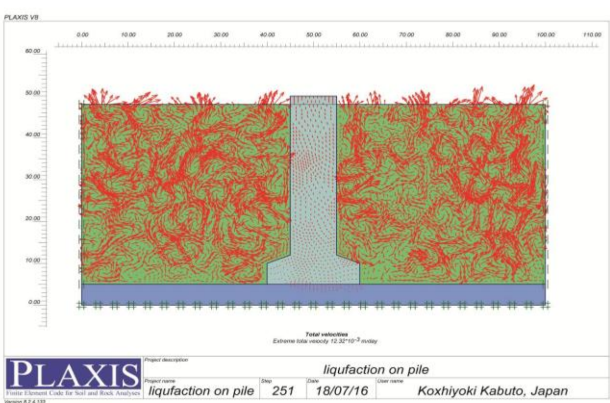

Figure 5. The velocity state during the liquefaction event

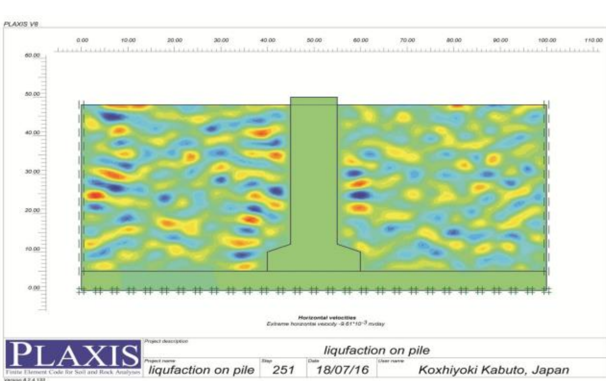

Figure 6. The stress field state during the liquefaction event

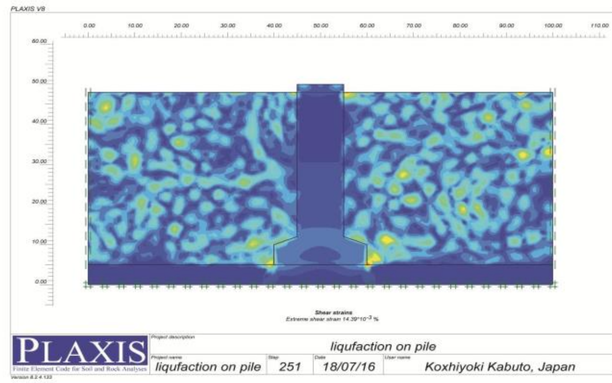

Figure 6. The shear strain state during the liquefaction event

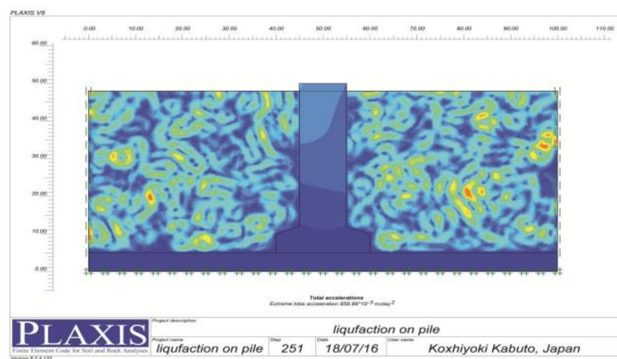

Figure 7. The accelerations state during the liquefaction event

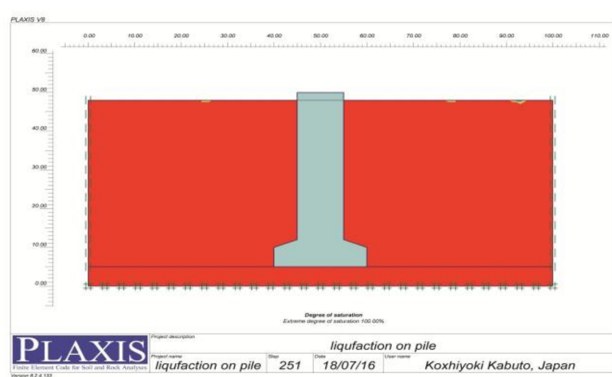

Figure 8. The saturation state during the liquefaction event

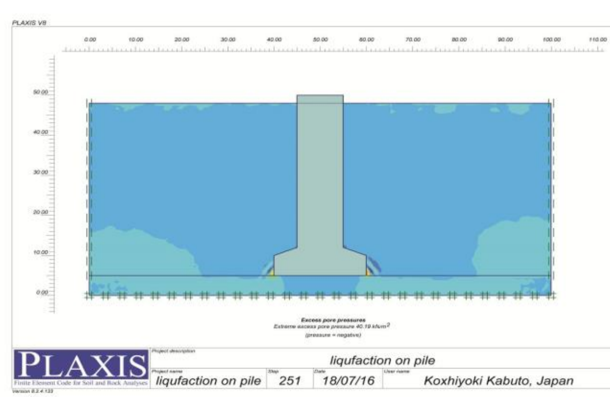

Figure 9. The pore-pressure state during the liquefaction event

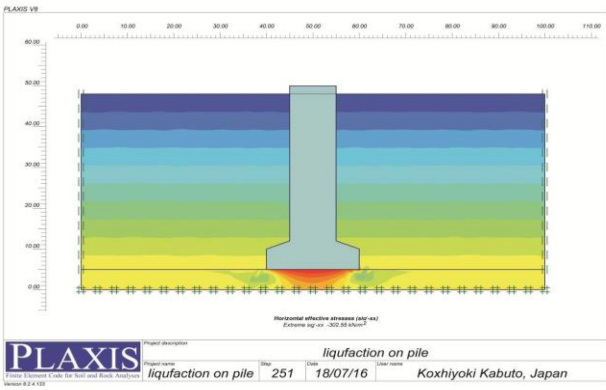

Figure 10. The main stress state of liquefaction inhibition on pile

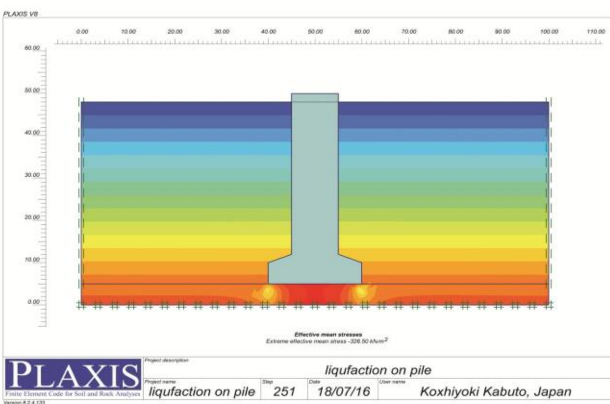

Figure 11. The shear stress state of liquefaction inhibition on pile

Numerical modeling allows data monitoring during analysis under different conditions to achieve an effective result. In experimental analyzes, monitoring is possible based on performing multiple tests to check the current situation. But it requires more money and time than numerical analysis. Based on the parameters considered in the modeling by FEM method used in this research and the results of mechanical modeling of soil liquefaction by Plaxis software, liquefaction event can be interpreted based on its mechanism of occurrence which is presented in Figs. 12 and 13.

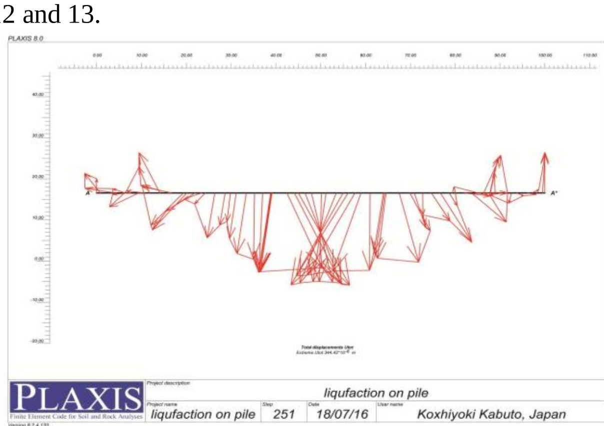

Figure 12. The displacement vectors state as the soil liquefies perpendicular to the pile

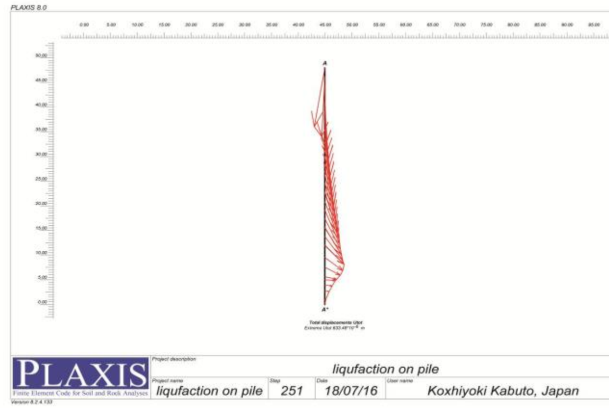

Figure 13. The displacement vectors state as the soil liquefies parallel to the pile

4. Conclusion

The results of this research can be presented is as:

A) If a saturated sand deposit vibrates, it tends to compact and shrink in volume. In this case, if drainage is not possible, the result will be an increase in pore water pressure. If, due to continuous vibration, the water pressure of the cavities in the sand deposits increases, sometimes its value may be equal to the overhead pressure. Under these conditions, the sand will have no shear strength and will become liquid.

B) Liquefaction is a phenomenon that often occurs in fine to medium grain sands and is related to non-stick grain soils. But cases have also been observed in sticky soils.

C) In sandy soils, sand particles are stored by the connection between the particles and energy can be transferred through these connections. During liquefaction, these joints are destroyed and the force between them becomes the pressure of the cavities, and the resistance to the soil becomes zero, and the soil behaves like a liquid whose specific gravity is equal to the saturated soil.

D) The basic mechanism of liquefaction in saturated and loose sand layers is the gradual increase of pore water pressure due to the application of cyclic stresses resulting from the shear wave propagation of the earthquake.

E) Studies show that specific weight gain has similar effects on liquefaction resistance. However, when the cementation reaches a critical level, the effects of specific gravity are overshadowed. Also, the presence of weak layers of cemented sand in a stronger mass reduces the resistance of that mass to liquefaction.

F) The most important methods of soil strengthening against liquefaction are the use of compaction, drainage, cementation, soil replacement, soil improvement, foundations, and structures.

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