International Journal of Earth, Energy and Environmental Sciences
ISSN: 2517-942X
Vol:13, No:2, 2019

Seismic Behavior of Suction Caisson Foundations

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***Abstract*—**Increasing population growth requires more sustainable development of energy. This non-contaminated energy has an inexhaustible energy source. One of the vital parameters in such structures is the choice of foundation type. Suction caissons are now used extensively worldwide for offshore wind turbine. Considering the presence of a number of offshore wind farms in earthquake areas, the study of the seismic behavior of suction caisson is necessary for better design. In this paper, the results obtained from three suction caisson models with different diameter (D) and skirt length (L) in saturated sand were compared with centrifuge test results. All models are analyzed using 3D finite element (FE) method taking account of elasto-plastic Mohr–Coulomb constitutive model for soil which is available in the ABAQUS library. The earthquake load applied to the base of models with a maximum acceleration of 0.65g. The results showed that numerical method is in relative good agreement with centrifuge results. The settlement and rotation of foundation decrease by increasing the skirt length and foundation diameter. The sand soil outside the caisson is prone to liquefaction due to its low confinement.

***Keywords*—**Liquefaction, suction caisson foundation, offshore wind turbine, numerical analysis, seismic behavior.

I. Introduction

ONE of the most vital concerns of humans is the environmental pollution. This pollution is caused by several factors, and one of the most important factors is the fossil fuels. Offshore wind energy, as a green and renewable energy, can be a safe alternative to other energies like fossil fuels [1]-[3].

Offshore wind power, one of the most promising sources of renewable and clean energies, has been utilized for decades on a large scale and has developed rapidly in the world [4]. The first of offshore wind farms was built in Denmark in a shallow depth of water (between 2-4 m) [5] after that it was developed in other countries.

Two common types of foundations for offshore wind turbines are gravity base and large diameter mono-pile foundations [4], [6]. These two foundations are mostly used for water at low depths. Mono-pile foundations are used for depths below 30 meters, and the gravity base foundations are used up to 40 meters in depth [2], [7]. One of the ways to generate more wind energy is to install wind turbines in the deeper water. In order to achieve this goal, suction caissons are the best alternative to traditional foundations. Suction caissons are the cylindrical and hollow large structures made of steel or concrete with an open bottom and closed top and also usually there are several holes on the top for suction [8]-[10].

The use of suction caisson has several advantages, such as high installation speed, no need special and large equipment for installation do not create noise during drilling, and can easily be destroyed at the end of service [11]-[13].

In the design of the suction caisson, several factors such as type of soil, type and load intensity, L/D (length of skirt (L) to diameter of suction caisson (D)), marine and construction conditions are effective. L/D is one of the most important factors affecting the behavior of suction caisson. Generally, the maximum L/D for suction caisson is 8 which is very small compared with a pile foundations which is about 60 [14]. Considering that some of the offshore wind farms are located in earthquake areas, the study of seismic behavior of suction caisson is one of the most vital engineering issues. Liquefaction is one of the phenomena that may occur during an earthquake and cause wind turbine structure destruction [15]. Investigation of seismic response of a turbine can be another important issue in the study of seismic behavior of suction caisson [16]. The acceleration response in both free-field and near suction caisson should be investigated, as it may cause pore water pressure and deformation of soil which ultimately affect the function of the foundation.

In the current research, the results obtained from the numerical method performed by the 3D FE software of ABAQUS [17] have been compared with the results of centrifuge test. This method can be used in the absence of an earthquake in situ, which is actually an appropriate alternative to prototype and real situations. Numerical method is another way to study the seismic behavior of suction caisson which can be done with various software. In this study, three models of Toyoura sand [18] soil and suction caisson with different dimensions have been used. In all the analysis, finite element method (FEM) [19], [20] is used and the results of them in different points of model by considering the interaction between soil and foundation are performed and compare with results of centrifuge test [21]. Dimension of suction caisson foundation (aspect ratio D/L) has different effects on the seismic behavior of suction caisson so in order to investigate this issue three different suction caisson have been chosen. In all models, a 1D earthquake with a dominant frequency of 1 Hz was used [21], [22]. In most studies, this earthquake has been used [23]. Generally, after comparing the results obtained from different models, we conclude that increasing the diameter of the suction caisson is one of the best ways to reduce the seismic response of the whole system.

In recent years, a few numerical and experimental studies have been carried out to survey this issue. Reference [24] has analyzed 3D dynamic behavior of suction caisson under earthquake loads. In this study, nonlinear soil behavior with

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kinematic hardening model with Von Mises failure criterion and associated flow rule has been considered. The dimension of foundation effects on accumulation of foundation rotation which can affect the turbine serviceability. In order to investigate, the seismic behavior of suction caisson [18] has tested several centrifuge models. Saturated sandy soils were used in all models then subjected to earthquake loading and the results were compared with each other. All the results obtained from centrifugation models were reasonable. References [22] and [25] have used centrifuge method in order to study seismic response of suction caisson. In general, tripod suction caisson has better resistance to structural settlement and lateral displacement.

II. Centrifuge Models

A. Centrifuge Device and Facilities

All centrifuge models were performed at Case Western Reserve University. The maximum acceleration of this centrifuge is 100 g and 200 g for dynamic and static tests respectively. The radius of the arm is 1.07 m but during flight it will increase to 1.37 m. The payload capacity of this geotechnical centrifuge is 20 g-ton and can carry up to 182 kg. All centrifuge models were performed in a rigid container with internal dimensions of 53.3 cm × 24.1 cm × 17.7 cm (length × width × height). Toyoura sand (D50=0.17 mm) with relative density about 68% were used for all models and the layer was constructed at 1g. The water level was 1.5 meters above the surface of the saturated soil in order to simulate the marine conditions.

B. Test Configuration and Properties of Soil and Earthquake

Three models with different aspect ratio (D/L) were subjected to centrifuge test. The tower head was simplified as a lumped mass of 10.6 ton. A general view of suction caisson is illustrated in Fig. 1.

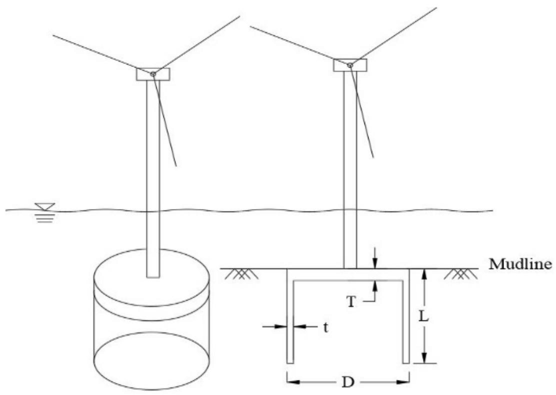


Fig. 1 Offshore wind turbine with a suction caisson foundation

As shown in Table I, models 1, 2 and 3 have a different aspect ratio with the same weight. The first model is considered as the reference model and we compare the results of the other models with it. In different parts of the numerical model, the values of acceleration, settlement and pore water pressure were measured and compared with the corresponding locations in the centrifuge model. Model configuration in prototype is shown in Fig. 2.

TABLE I
DIMENSION OF SUCTION CAISSON MODELS

	D (cm)	L (cm)	W (kg)	Aspect Ratio (D/L)	T (cm)	T (cm)
Model 1	400	175	18700	2.28	10	40
Model 2	600	175	18700	3.42	10	40
Model 3	400	250	18700	1.60	10	40

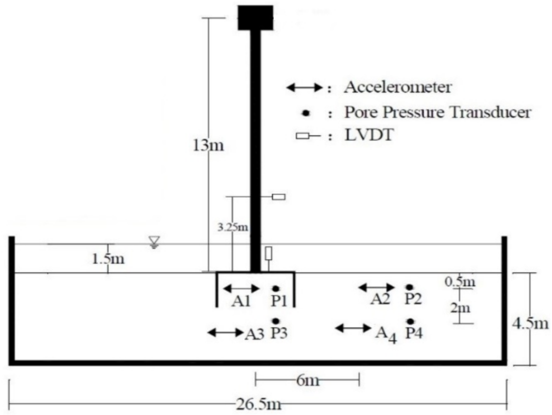


Fig. 2 Model configuration

The earthquake load applied to the base of all models is a one-dimensional synthetic earthquake with a maximum acceleration of 0.65 g and a dominant frequency of 1 Hz (Fig. 3). The properties of sand are summarized in Table II. It should be mentioned that [26] has investigated the seismic behavior of Toyoura sand, so the results can be checked with the previous study in order to reliability of the recorded data.

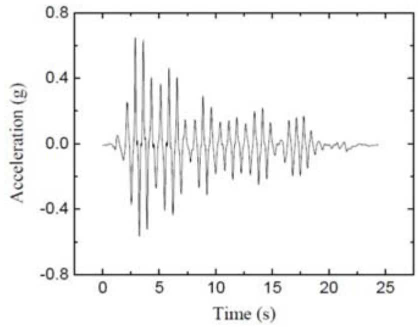


Fig. 3 Input earthquake

TABLE II
PROPERTIES OF TOYOURA SAND

Parameter	C_u	C_C	Specific Gravity	D_{50}	D_{10}	Max. void ratio	Min. void ratio
Value	1.59	0.96	2.65	0.17	0.16	0.98	0.60

III. Numerical Models

In general, more studies have been done on the seismic behavior of piles compared to the suction caisson. Due to the difference in the geometric shape of caisson (a hollow foundation with a cap or lid) relative to the pile, such as dimension of diameter, length of skirt and aspect ratio (D/L), there can be different wave propagation mechanisms, due to the contact of the soil with the cap. This was investigated by Latini [27]. In this research, the bearing behavior of suction caisson subjected to earthquake load in saturated sand is investigated by means of numerical simulation with the 3D FEM in the time domain using ABAQUS software which is able to analyze the complex problems in mechanics. The material behavior of suction caisson and soil is described using linear elastic isotropic and an elasto-plastic Mohr–Coulomb constitutive model respectively with perfect contact between the suction caisson and sand during the analysis. For the friction resistance between interface of skirt caisson and soil, a friction angle of 20◦ is considered.

Mohr-Coulomb (M-C) criterion is based on the assumption that the maximum shear stress is the decisive measure of yielding. The failure function of M-C criterion is expressed in terms of the principal stresses [28], [29], and it is assumed that are ordered according to σ1≥σ2≥σ3. Fig. 4 shows the Mohr-Coulomb in the principal stress space.

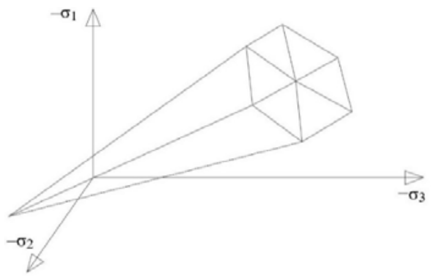


Fig. 4 M-C failure criterion in the principal stress space

Mohr–Coulomb model presents the plastic potential (g) and effective stress (σ_e):

$$\sigma_e = m\,\sigma_1 - \sigma_3, \quad g = n\sigma_1 - \sigma_3 \tag{1}$$

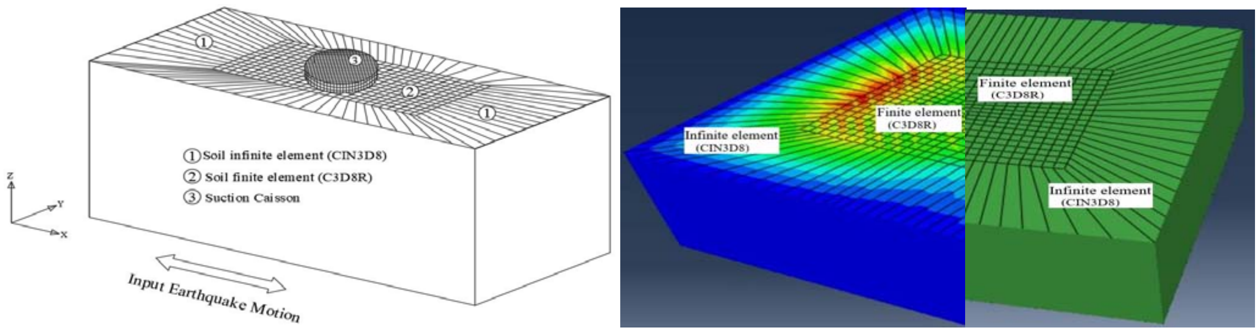
$$m = (1 + \sin\phi) / (1 - \sin\phi) \tag{2}$$

$$n = (1 + \sin\psi) / (1 - \sin\psi) \tag{3}$$

where: σ1: maximum principal stress, σ3: minimum principal stress, m: pressure sensitivity parameter related to the friction angle of the material, n: non-associativity parameter related to the dilation angle of the material. ϕ and c are the parameters of friction angle and cohesion, respectively.

Infinite element (CIN3D8-elements) has been used for the boundaries of the models in order to deter spurious reflection of waves to get real responses, while other part of models was modelled using FE (eight-node volume C3D8R-elements) and the length of the infinite elements is more than half of the model width [30] (see Fig. 5). Wind turbines are insensitive to earthquake loading because they are low frequencies structures. Therefore, in all parts of the model, mesh sizes are considered small enough to capture stress wave at high frequencies, and also the mesh sizes near the caisson and adjacent soil are finer.

In order to consider the interaction between soil and suction caisson in the right way, two surfaces with the creation of friction between them were considered for the interface by using a "master-slave" modeling concept. Created friction consists of shear and normal friction components which are tangential and perpendicular to the surface, respectively. Coulomb theory was used to gain the friction coefficient between two surfaces on the soil-caisson interface (μ = δ tangφ, δ = 0.6).



(a) (b) (c)

Fig. 5 (a) Input earthquake motion, (b) Finite and infinite element after run, (c) Finite and infinite element before run

Geostatic stress is usually the first step of a geotechnical analysis, followed by a coupled pore fluid diffusion/stress or static analysis procedure to verify that the initial geostatic stress field is in equilibrium with applied loads and boundary conditions. So, in the first step, these stresses as initial stresses were computed by application of gravity loading. k0 = 1 − sin ϕ equation was used for earth pressure or lateral coefficient at rest, due to its self-weight.

In order to calculate and consider the Rayleigh damping coefficients for soil in all models, frequency analysis has been done before seismic analysis.

Considering a dynamic system, we have:

$$[M]\left(\frac{d^2x}{dt^2}\right) + [C]\left(\frac{dx}{dt}\right) + [K]\left(x(t)\right) = F_{stat} + F_{dyn} \quad (4)$$

where x (t) is displacement as a function of time; [M]: mass matrix; [C]: damping matrix and [K]: stiffness matrix.

In the case of Rayleigh damping, [C] is determined as:

$$[C] = (\alpha_M)[M] + (\beta_K)[K] \quad (5)$$

where: [K]: matrix of linear stiffness, αM: constant (S-1) and βk: constant (S). [K] refers to the matrix of linear stiffness with initial tangential stiffnesses. Thus, [C] includes mass and stiffness proportional terms. αM and βk are the model coefficients used to specify the model damping ratio in two modes. Also, according to (4), αM and βk can be determined by choosing appropriate values of damping, to the degree possible, to the modes of the linear system.

Base on the damping ratio, ξi, related to the critical value, can calculate the Damping of mode i and also by αM and βk, ξi can be quantified from:

$$\xi_i = \frac{1}{2\omega_i}\alpha_M + \frac{\omega_i}{2}\beta_K \quad (6)$$

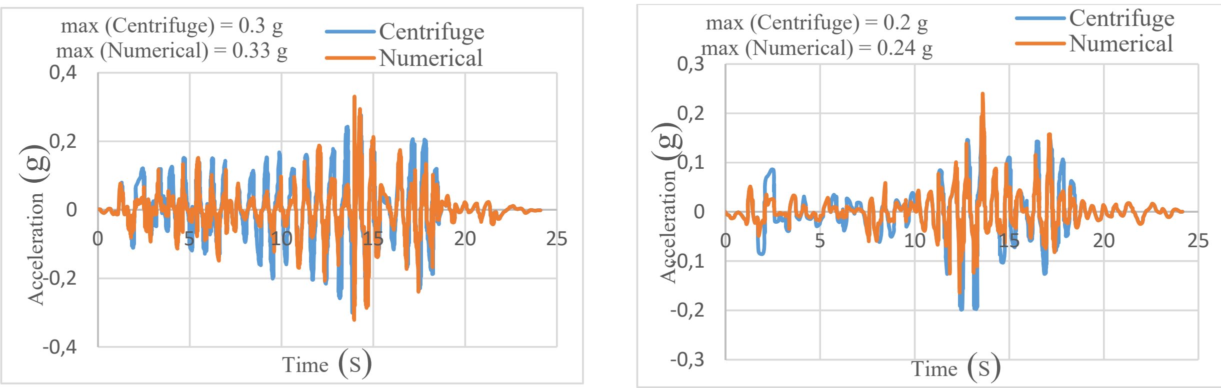
where: ωi (rad/s) is the natural frequency of mode i.

So, both βk and αM can be set to give any damping ratio to any two different modes. The damping amount of other modes can also be computed from (6).

IV. Results and Discussion

A. Seismic Response of Soil Deposit

The results presented in this section are related to the analysis of the first model. Acceleration responses were measured inside and underneath of the caisson (A1 and A3) and in the free field (FF) (A2 and A4). Fig. 6 (a) shows the comparison of the acceleration response inside the suction caisson (A1) in both models of centrifuge test and numerical analysis. The results of these two methods are close to each other, so that the maximum acceleration in centrifuge test is 0.3 g and in the numerical analysis is 0.33g. Fig. 6 (b) represents the seismic response of both methods in the free field (A2). These values are 0.2 g and 0.24 g for centrifuge and numerical model, respectively. In general, the acceleration response inside the suction caisson is greater than the free field, which is due to the presence of the caisson in the soil. In fact the soil inside the caisson foundation was reinforced by the skirt of the suction caisson.



(a) A_1 (b) A_2

Fig. 6 Acceleration response of centrifuge test and numerical analysis for model 1

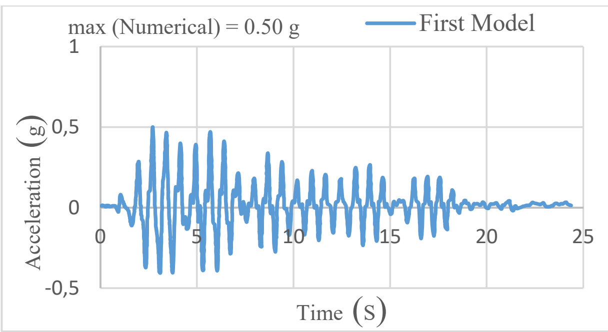


Fig. 7 Acceleration response of numerical analysis for model 1 in A3 & A4

In order to investigate the behavior of the soil under the suction caisson, the acceleration response is measured at a point below the caisson (A3) and also a point below with a distance from the caisson (free field) (A4). The soil behavior under the suction caisson is similar to the free field, and the existence of a caisson does not have any effect on the seismic behavior of the underlying soil. These results are obtained from the numerical analysis of model 1 which are similar to the results of research of [25].

Changes in acceleration response and pore water pressure (PWP) affect the strength and stiffness of the soil. If the pore water pressure increases so much that it approaches the effective stress of soil, the liquefaction phenomenon occurs. To investigate the liquefaction phenomenon, the pore water pressure ratio (Δp /σe0) is measured at various points in the model. Δp is increment of excess pore water pressures and σe0 is initial vertical effective stresses. If this pore water ratio approaches to 1, there is a high probability that the soil will liquefy.

Figs. 8 (a) and (b) show the comparison of the pore water pressure at P1 and P2 respectively (see Fig. 2) in both models of centrifuge test and numerical analysis. As the figure shows, the results obtained from both centrifuge and numerical methods are approximately the same. Pore water pressure at points 1 and 2 which are located at the same level and 0.5 meters below the soil surface, is completely different. This ratio is more than 1 at P2 (in free field) which indicated the occurrence of liquefaction. Generally, the reduction rate of PWP inside the suction caisson is lower than the free field because the pore water can only flow from the bottom of the foundation.

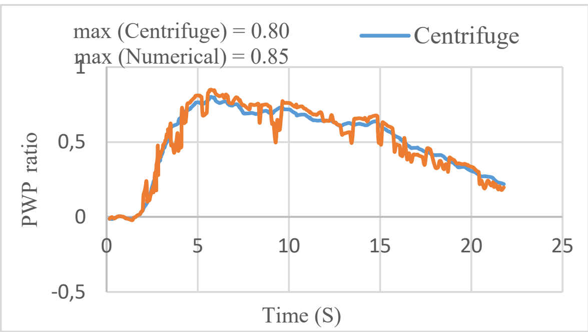
Regarding the values of the acceleration response of the soil and PWP, the liquefaction phenomenon can be investigated. In fact, the tendency of liquefaction increased as the acceleration weakened.

By comparing the graphs of acceleration response and pore water pressure ratio (Figs. 6-8), it can be concluded wherever the acceleration response decreases, the tendency of liquefaction will increase. So, the soil in free field will be liquefied but there will not be liquefaction phenomena inside the foundation because of existence of suction caisson.

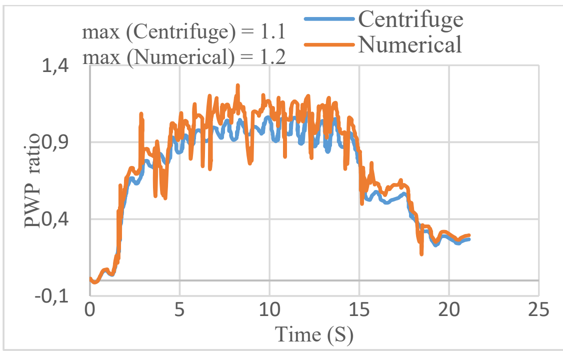
B. Seismic Response of Suction Caisson

In order to investigate the effect of foundation dimensions (diameter and length of skirt or aspect ratio (D/L)) on the seismic response of suction caisson, the results of the second (increase in diameter).

Fig. 9 shows the results of centrifuge testing and numerical analysis. These results are related to the PWP ratio at point P1. Given that the difference between numerical results and centrifuge tests is negligible .The maximum PWP ratio is 0.85 (numerical model) which is related to the first model. This coefficient reduces the soil stiffness and results in more settlement of foundation. The smallest amount of maximum PWP ratio (0.53) is achieved by increasing the caisson diameter .In fact, by increasing the caisson diameter in the second model, the PWP ratio and foundation settlement are significantly reduced because the area of reinforced foundation soil is increased.

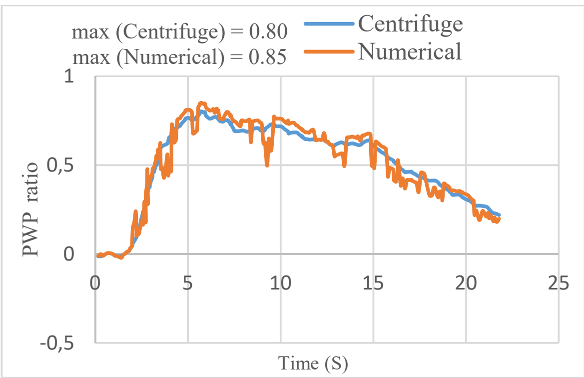


(a) P_1

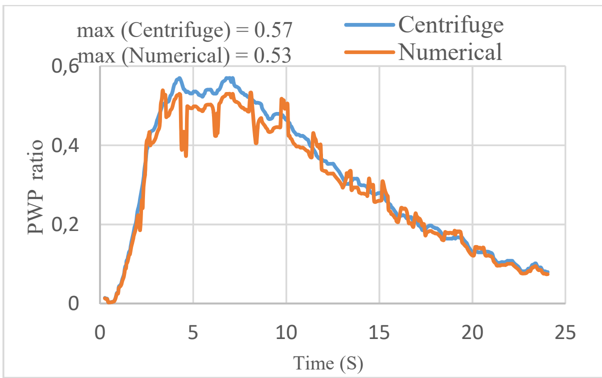


(b) P_2

Fig. 8 Comparison of pore water pressure ratio in first model



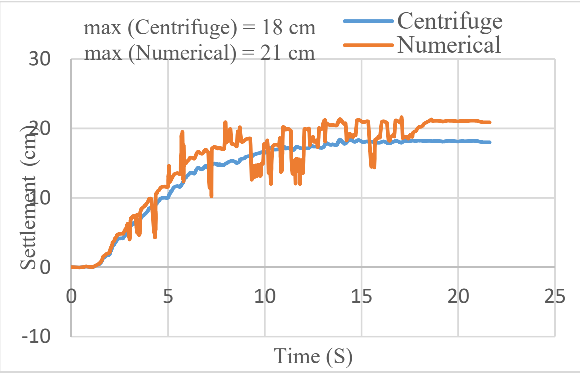
(a) First Model



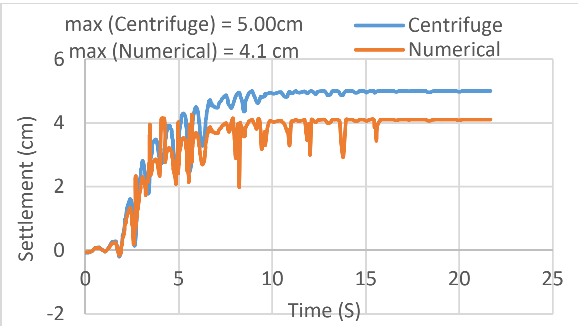
(b) Second Model

Fig. 9 PWP ratio recorded by P1 in the first and second models

Fig. 10 and Table III show the comparison of the suction caisson settlement and rotation for the first and third models in both centrifuge test and numerical analysis, respectively. Since the maximum PWP ratio is high in the first model (see Fig. 9), the foundation soil will be softened. As shown in Fig. 10 and Table III, the largest settlement and rotation of the caisson are related to the first model and decrease with increasing the skirt length. So that the lowest settlement of foundation is occurred with increasing the skirt length in model 3 (4.1 cm). The increased embedment of caisson by increasing the skirt length (or decrease the aspect ratio (D/L) without any changing in diameter) has amplified the interaction between the suction caisson and surrounding soil as well as increased the friction force on both sides of the skirt. Generally, the larger penetration depth might help to resist more settlement during the earthquake loading.



(a) First Model



(b)Third Model

Fig. 10 Maximum settlements in both centrifuge test and numerical analysis in the first and third models

TABLE III
COMPARISON OF MAXIMUM SETTLEMENTS

	Max. Rotation (Centrifuge Test)	Max. Rotation (Numerical Analysis)	Max. Settlement (Centrifuge Test) (cm)	Max. Settlement (Numerical Analysis) (cm)
Model 1	0.64°	0.69°	18	21
Model 3	0.43°	0.50°	5	4.1

V. CONCLUSIONS

Investigation of behavior of suction caisson foundations under different loading especially earthquake loads is inevitable. This paper presents the results of investigation of seismic behavior of suction caisson under earthquake loading in saturated sand soil using FEM, then obtained results were compared with centrifuge models in order to verify the numerical method. One of the parameters influencing suction caisson's behavior is the dimension of foundation or aspect ratio (D/L). The results can be summarized as:

1. There is little differences between the results obtained from centrifuge tests and the numerical method used in this study. Therefore, one of the approaches for investigating seismic behavior of suction caisson foundation is to use a suitable numerical method.
2. Increasing in excess pore water pressure causes liquefaction in the soil. Due to the reinforcement of surrounding soil by the suction caisson, the pore water pressure ratio in the foundation is lower than the free field and also, the tendency of liquefaction increased as the acceleration weakened. According to the second model, PWP decreases by increasing the diameter of suction caisson foundation.
3. The dimensions of the suction caisson or aspect ratio (D/L) affect the soil stiffness as well as the foundation's behavior. The maximum rotation of foundation decreased by increasing skirt length of the caisson in the third model. The maximum rotation of the foundation in the first model (reference model) was 0.69°, which decreased to 0.50° by increasing the skirt length of the suction caisson.
4. Increasing the diameter and skirt length of the caisson greatly affects the foundation settlement. By increasing the skirt length of suction caisson, the maximum settlement decreased from 21 cm to 4 cm. By increasing the skirt length, the friction between soil and skirt will increase, and the inside soil is compressed more and better due to the small diameter.

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