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Deterministic assessment and numerical simulations of buckling instability of piles in liquefiable ground

Évaluation déterministe et simulations numériques de l'instabilité de flambement des pieux dans un sol liquéfiable

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ABSTRACT: This paper discussed an alternative to compute and analyze the buckling instability of piles in a liquefiable ground during seismic loading. A key aspect of the study is a comparison of different approaches on buckling assessment of piles using deterministic and numerical method. The study discusses the assessment of buckling instability of piles due to liquefaction of foundation soils for a coal-fired power station (CFPS) in Indonesia. Liquefaction analysis is performed based on a SPT-N approach and results indicate the foundation soils of study area would be prone to liquefaction due to design earthquake. Results of buckling assessment using a deterministic approach proposed by Bhattacharya show a pile buckling index $(I_B) \geq 1.0$, with an average I_B value 5.3, indicating the pile foundation of CFPS should be safe from buckling failure due to design earthquake and soil liquefaction. The numerical simulations are undertaken based on a 3D finite element method (FEM) using OpenSeesPL. To explore the effect of shaking, soil and pile behaviors are examined due to various factors. This study sets out to gain further understanding on soil responses and soil-pile behaviors during seismic excitation. For the case history examined by Bhattacharya approach, FEM results show the computed bending moments in piles (M_N) are less than the ultimate bending moments of piles (M_D) due to design earthquake and soil foundation. Accordingly, both deterministic and numerical approaches confirm the piles would be safe from buckling failure against soil liquefaction during seismic loading for the CFPS foundation examined.

RÉSUMÉ : Cet article a discuté d'une alternative pour calculer et analyser l'instabilité de flambement des pieux dans un sol liquéfiable pendant le chargement sismique. Un aspect clé de l'étude est une comparaison de différentes approches sur l'évaluation du flambement des pieux en utilisant une méthode déterministe et numérique. L'étude traite de l'évaluation de l'instabilité de flambement des pieux en conséquence à la liquéfaction des sols d'une centrale électrique au charbon (CFPS) en Indonésie. L'analyse de liquéfaction est réalisée sur la base d'une approche SPT-N et les résultats indiquent que les sols de fondation de la zone d'étude seraient sujets à la liquéfaction en raison du design sismique. Les résultats de l'évaluation du flambement en utilisant d'une approche déterministe proposée par Bhattacharya que un flambement des pieux $(I_B) \geq 1{,}0$, avec une valeur moyenne du $I_B$est 5.3, indiquant que la fondation sur pieux du CFPS devrait être à l'abri de la rupture de flambement comme un effet du design sismique et liquéfaction du sol OpenSeesPL est utilisé sous forme de simulations numériques par une méthode d'éléments finis 3D. Pour explorer l'effet de l'agitation, les comportements du sol et des pieux sont examinés en raison de divers facteurs. Cette étude vise à approfondir la compréhension des réponses et des comportements du sol lorsque de l'excitation sismique. Pour l'histoire de cas examinée par l'approche Bhattacharya, les résultats FEM révélé que le calculation de moments de flexion des pieux (M_N) sont inférieurs aux moments de flexion ultimes des pieux (M_D) en raison du design sismique et de la fondation du sol. Par conséquent, les approches déterministe et numérique confirment que les pieux seraient n'éprouver aucun échec de flambement des pieux contre la liquéfaction du sol pendant le chargement sismique pour la fondation du CFPS examinée.

KEYWORDS: Pile behavior; soil response, soil liquefaction; seismic shaking; numerical simulation.

1 INTRODUCTION

During strong earthquakes, soil liquefaction can result in extensive damages to buildings, bridges, port facilities, other infrastructures, and loss of lives. In liquefiable ground, pile foundations need to resist additional seismic loading while continuing to carry the normal gravity loads from superstructures (Madabhushi 2012).

Liquefaction was reported as the main cause of damages of pile foundations during major earthquakes, for instances, 1964 Alaska, 1989 Loma-Prieta, 1995 Hyokoken-Nambu (Kobe), 1999 Chi-Chi, 1999 Koceli, 2001 Bhuj, 2011 Tohoku, 2018 Palu earthquakes (Kramer 1996; Finn & Fujita 2002, EERI 1999; Andhika & Bhattacharya 2008; Bhattacharya & Goda 2013; GEER 2019). The collapse of pile-supported structures is a major concern in the geotechnical earthquake engineering. The performance of pile foundations in liquefiable ground is a very complex process involving kinematic interactions among structure and pile, and seismically-induced pore pressure and non-linear responses of soils due to earthquake motions.

Structures supported by pile foundation can be subjected to axial and lateral loadings. In structure terms, beam bending and column buckling are developed differently. Bending is a stable mechanism as long as pile remains elastic and secondary failure is not a possibility, however, buckling is an unstable mechanism and occurs suddenly and drastically when the elastic critical load is reached. When the pile is driven into the soil, the surrounding soil is compressed. As a consequence, a lateral stress will be imposed on the pile shaft. The force and deformation behaviors of structural piles during earthquake shaking as shown in Fig. 1.

Seismic performance and design of pile foundation in the fields of liquefaction and lateral spreading have been studied extensively in recent years. Design for liquefaction conditions using a variety of beam on soil spring models have been proposed with a range of recommendations regarding parameter selections and loading details (Tokimatsu & Asaka 1998; Boulanger et al. 2003; Liyanapathirana & Poulos 2005; Cubronovski & Ishihara 2005; Brandenberg et al. 2007).

The buckling failure of pile under the combined lateral and axial loads have received a little attention. Buckling instability has been identified as a possible mechanism of pile failure in liquefaction ground and this failure mechanism is not explicitly mentioned in the most of design code. Specifically, the following issues could provide a more comprehensive understanding and propose an alternative approach, by critically analyzing the effects of soil-pile responses due to seismic loading and behavior of pile buckling during soil liquefaction, with deterministic assessment (Bhattacharya 2015) and numerical simulation through OpenSeesPL (Lu et al. 2011).

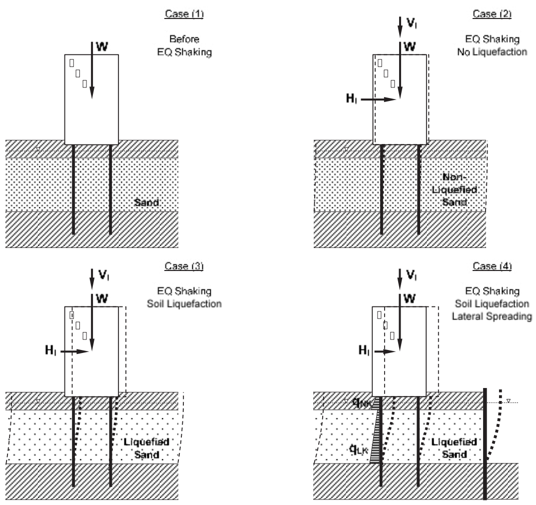

Figure 1. The deformation of pile foundations under liquefaction of lateral spreading.

2 EVALUATION OF BUCKLING INSTABILITY DUE TO SOIL LIQUEFACTION

Seismic design of foundation requires information on a number of characteristics of the earthquake at the site. The following characteristics are used to evaluated the pile buckling, such as design earthquake magnitude (M_w), peak ground acceleration (a_{max}), additional axial load (ΔP), acting moment (M_B), dynamic axial load ($P_{dynamic}$), unsupported length of pile (D_L), and critical length of pile (H_C).

The unsupported length of pile (D_L) is assessed based on the liquefaction profile. In the subsequent phase of the calculation of the acting moment (M_B) and dynamic axial load ($P_{dynamic}$), we can compute the maximum axial load acts on the pile. It can be known that the additional axial load (ΔP) needs to calculated first to obtain the dynamic axial load of the pile ($P_{dynamic}$). To do this, the additional axial load (ΔP) and bending moment of inertial force of the structure for post-liquefaction (M_B) must be considered then distributed to all of resisting piles of the superstructure.

An estimate of the maximum axial loads acts on a pile is given by Eq. (1):

$$P_{dynamic} = P_{static} + \Delta P = (1 + \alpha)P_{static} \tag{1}$$

where P_{static} is the static load acting on each pile under the building, ΔP is the additional load on the piles due to seismic shaking on the superstructure, and α is a term of dynamic axial load factor, which is function of type, dimension and mass of superstructure, characteristic of seismic shaking, as well as material properties and geometry of the piled foundation.

To assess of critical length of pile (H_C), the limit state condition of failure is assumed, $P_{dynamic} = P_{failure}$ and $L_{eff} = H_C$. The actual failure load $P_{failure}$ can be written as:

$$P_{failure} = \phi P_{cr} = \frac{\phi \pi^2 EI}{K^2 H_C^2} \tag{2}$$

By rearranging Eq. (2), the H_C can be evaluated by:

$$H_C = \sqrt{\frac{0.35\pi^2 EI}{K^2 P_{dynamic}}} = \sqrt{\frac{0.35\pi^2 EI}{K^2(1+\alpha)P_{static}}} = \sqrt{\frac{0.35\pi^2 EI}{K^2 P_{static}+\Delta P}} \tag{3}$$

The evaluation of H_C is based on the computation of the critical buckling load for a pile foundation surrounded by liquefied soil. The values of ϕ and K can be selected based on engineering judgement.

A buckling index (I_B) is adopted as the ratio between the H_C for buckling and D_L due to liquefaction foundation soils. With the assessed H_C, liquefaction-induced pile buckling is indicated if $H_C < D_L$. The buckling index (I_B) is expressed as follow:

$$I_B = \frac{H_C}{D_L} \tag{4}$$

As $I_B \geq 1.0$, the $H_C > D_L$ and the pile is considered safe. Otherwise, if $I_B < 1.0$, the pile will be buckle due to seismic loading and soil liquefaction. The prediction of the buckling of piles instability and the effect of soil-pile interactions in liquefiable ground have been widely investigated (Fansuri et al. 2019a, 2019b, 2020).

3 DETERMINISTIC ASSESSMENT OF BUCKLING INSTABILITY OF PILES IN LIQUEFIABLE GROUND

The case study is related to a construction of a coal-fired power station in Central Java Indonesia, with capacity of 2×1000MW. The study area covers three main facilities, including Boiler Units 5 & 6 and Central Control Building (CCB).

In general, the project site is located on an alluvial deposit of the Muria mountain sediments. The materials consist of coarse sand, fine sand, and clay, formed by old and recent river alluvium and shore deposits. The upper deposits comprise sublayers of very soft to soft clays and very loose to loose silty sands.

The study reported that the Pati earthquake of 1880 with a magnitude (M_w) of 6.8 occurred at a distance of 45 to 50 km away from the project site with estimated MMI scale of VII. The peak ground acceleration (a_{max}) is adopted from the Indonesian generic response spectral of the Center of Research and Development of Housing and Settlement (Puskim), Ministry of Public Work & Public Housing (2011) database consistent with regional motion.

The liquefaction potential of the site is assessed based on the SPT-N approach by Youd et al. (2001) in association with the depth-weighted method by Iwasaki et al. (1982). A total of 16 boreholes logs has been collected for this study.

This study sets out to assess the cyclic stress ratio (CSR) and the cyclic resistance ratio (CRR) in wet season. As a basis for comparison, the following values of parameter are assumed: the depth to groundwater table during earthquake (GWT) = 0.9m, the groundwater table during the subsurface exploration and testing (GWT_0) = 1.92m, hammer energy ratio (ER) = 70%, a unit weight above groundwater (γ_m) = 16.5 kN/m^3, below groundwater (γ_{sat}) = 16.68 kN/m^3. In particular, the assignment of groundwater table would be problematic if GWT and GWT_0 are set as equal, or one groundwater table scenario. In this situation, the CSR of soil is not exactly computed by the groundwater table during seismic shaking and the erroneous GWT assignment would result in unanticipated liquefaction potential index (LPI) evaluation (Chang et al. 2020).

This study analyzes for cases of liquefaction ground in both dry and wet seasons, while only the wet season is the main focus of this study. The results of D_L from representative 47 piles can be separated into three main facilities indicating more than 50% of computed piles with D_L value less than 4m, which are consistent with LPI results with very high liquefaction potentials. The results of H_C indicate the minimum length that the on-site pile will buckle is 18m, when the foundations soils are liquefied due to design earthquake in wet season. The study confirms that the H_C is not associated with D_L. This inconsistency may be due to the value of $P_{dynamic}$ on each pile. A note caution is might due to arrangement of the piles that causes ΔP and $P_{dynamic}$ to increase while H_C decreases.

As shown in Table 1 and Fig. 2, the study areas will be safe from buckling instability, with calculated I_B value of greater than 2 and an average of I_B value of 7, for all of the piles at the study area. One unanticipated finding is that LPI is medium to high, but it does not give severe effects to buckling index (I_B). A decrease in unsupported length of pile (D_L) in this study contributes to the buckling index (I_B). The study confirms that the axial loads each pile (P_{static}) is associated with the dynamic axial load ($P_{dynamic}$). When the $P_{dynamic}$ decreases, the H_C increases. In accordance with the present results, in keeping the D_L as constant, the increasing of H_C would then increase the I_B.

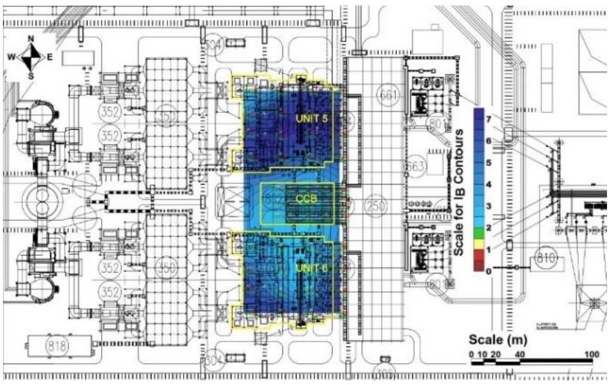

Figure 2. I_B contours for post-liquefaction in wet season.

4 NUMERICAL SIMULATIONS OF BUCKLING INSTABILITY OF PILES IN LIQUEFIABLE GROUND

4.1 *Numerical analysis tool*

The simulations are carried out by using a computation platform, OpenSees (Mazzoni et al. 2006; McKenna et al. 2010). The OpenSees is open-source software developed by the Pasific Earthquake Engineering Research (PEER) Center. The software is capable of modeling the coupling response between the soil skeleton and the pore fluid as well as the redistribution of pore pressure during shaking either in two or three dimensions (Fonseca et al. 2017).

The OpenSeesPL approach is adopted in this study to create a finite-element model for numerical simulation. The OpenSeesPL has a graphical user interface (GUI) for three dimensional and ground surface analysis under seismic loading condition (https://soilquake.net/openseespl/). In the static analysis, it can adopt simulations with linear, bilinear, or advanced (i.e., non-linear fiber) element formulations. It has capabilities for carrying out a large variety of 3D finite-element simulations based on the OpenSees computational platform (Lu 2006; Elgamal & Lu 2009).

Table 1. I_B Category and numbers of pile

I_B Category	Number of pile computed	
	Wet season	Dry season
$I_B \leq$ -5	0	0
-5 $< I_B \leq$ 0	0	0
0 $< I_B \leq$ 5	0	0
5 $< I_B \leq$ 10	0	0
10 $< I_B$	47	47
Total	47	47

4.2 *Analyses procedures and outputs*

All simulations were developed and executed using OpenSeesPL based on u-p formulation. The 3D finite-element soil domain is represented by 8-4-node, fully coupled (soil-fluid) brickUP elements. In this scenario, the profile corresponds to an 8-m thick sand layer underlain by a 4-m thick soft clay, which is then overlain an 8-m thick hard clay layer. The model of numerical simulations was performed with a groundwater level of 1-m below ground surface. The model adopted in numerical analyses is shown in Fig. 3.

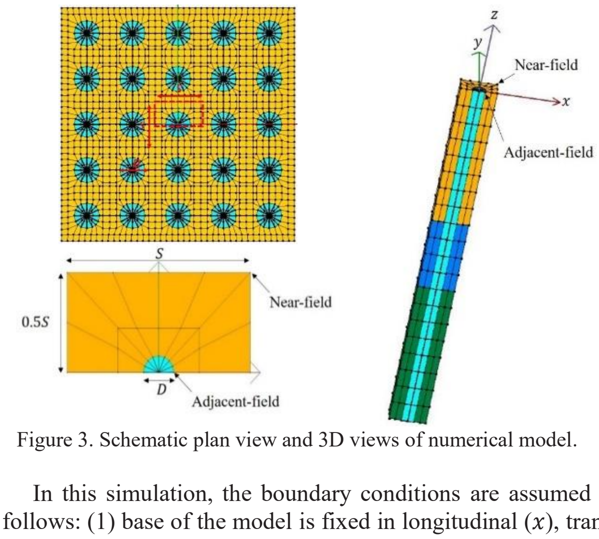

Figure 3. Schematic plan view and 3D views of numerical model.

In this simulation, the boundary conditions are assumed as follows: (1) base of the model is fixed in longitudinal (x), transverse (y), and vertical (z) directions. All meshes are fixed in the y and z directions and free in x direction. The axial load is applied at the pile head in the z direction; (2) the scenarios are analyzed using meshes with spacing 1-m in the z direction, the x and y directions using the different meshes based on the dimensions of scenarios; (3) the input motion is imposed on the base in the x-direction (E-W acceleration record) embedded at a 20-m depth; (4) the record motion is adopted from PEER NGA West2 during Niigata Chuetsu-Oki earthquake.

To investigate the effects of seismic shaking and the behavior of the soil liquefaction on the pile, the various factors with different axial loadings, ground inclinations, pile spacings, and diameters are considered. In this study, we describe the results obtained from 3D seismic response with considering the behaviors of soil in adjacent-field (i.e., at the contact with the pile) and near-field (i.e., at the center of soil among the piles), as illustrated in Fig. 3. The numerical simulations present soil response shaking (i.e., acceleration vs. time and spectral acceleration), behavior of soil (i.e., excess pore pressure and unsupported length of pile, D_L), and behavior of the pile (i.e., displacement and bending moment profiles as well as critical length of pile, H_C).

4.3 *Example case description*

In this study, a series numerical analysis was carried out to investigate the influence of various factors, including axial load, ground inclination, pile spacings and diameters. The physical and mechanical properties of soil layer and reinforced concrete pile are presenter in Tables 2 & 3, respectively. The shaking data was based on the earthquake of Niigata Chuetsu-Oki, Japan (2007), as shown in Fig. 4, which has a moment magnitude M_w, shear velocity V_s, and the distance from epicenter close to the parameters selected in the previous method by deterministic approach. Moreover, this earthquake has been related to large ground failures and extensive liquefaction phenomena observed at the site.

A number of scenarios that would demonstrate the effects of non-linear soil response and compare their influences on the soils in the adjacent and near fields, as shown in Table 4. In total, 9 simulations were performed to explore the different combinations of axial loadings, ground inclinations, pile spacings and diameters.

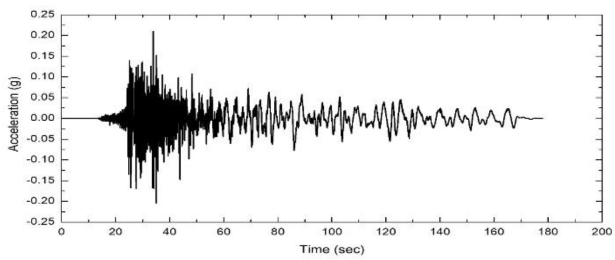

Figure 4. Base input motion.

5 COMPARISON OF DETERMINISTIC AND NUMERICAL ASSESSMENT ON PILE BUCKLING IN LIQUEFIABLE GROUND

The buckling of piles in liquefiable ground can be assessed by deterministic approach (Bhattacharya 2015), which considers the unsupported length of pile D_L and critical length of pile H_C. Prior to assessment of pile buckling, we should evaluate the liquefaction potential (LPI) in the study areas (i.e., Unit 5 & 6 and CCB) using Youd et al. (2001). The results of LPI show the deposit soils will liquefy due to assigned seismic loading. However, the subsequent buckling assessment indicates the project piles will be safe from buckling failure for the given pile axial loads and pile diameters.

In the numerical simulations, the response of excess pore pressure would generally increase with increasing axial loading, ground inclination, spacing and diameter of pile. For instances in Figs 5 & 6, the loose sand develops lesser excess pore pressure in level ground than in gentle slopes. The excess pore pressure responses would be delayed in peaking as the slope becomes more inclining. An increase in ground inclination would generally increase in effective stress in soils. A possibility for explanation is that the pile and soil behave as elastic materials and thus the Poisson's effect plays a role to the results. The increase in ground inclination would slightly decrease the excess pore pressure and thus affects increasing the initial effective stress in soils.

With prediction of excess pore pressure in the adjacent-field, numerical simulations could compare results by Bhattacharya's approach. Through numerical simulations, we would be able to determine the depth of liquefiable soil or unsupported length of pile (D_L).

Table 2. Soil parameters

Parameters	Unit	Sands	Soft clay	Stiff clay
Mass density γ_m	Mg/m^3	1.7	1.3	18
Friction angle ϕ	deg	29	0	0
Cohesion	kPa	0.3	18	75
Hor. permeability k_h	m/s	6.6×10^{-5}	1.0×10^{-9}	1.0×10^{-5}
Ver. permeability k_v	m/s	6.6×10^{-5}	1.0×10^{-9}	1.0×10^{-5}
PT angle ϕ_{PT}	deg	29	-	-
Contr. parameters c_1		0.21	-	-
Dilation param. d_1		0	-	-
Dilation param. d_2		0	-	-
Liq. parameter l_1	kPa	10	-	-
Liq. parameter l_2		0.02	-	-
Liq. parameter l_3		1	-	-

Table 3. Reinforced concrete pile parameters

Parameters	Unit	Reinforced concrete pile with different diameters		
		0.6m	0.8m	1.0m
Weight of concrete γ_c	Mg/m^3	2.5	2.5	2.5
Yielding moment M_y	kN-m	8.8×10^2	2.0×10^3	3.7×10^3
Plastic moment M_u	kN-m	1.3×10^3	2.9×10^3	5.4×10^3
Flexural rigidity EI	kN-m^2	1.8×10^5	5.3×10^5	1.2×10^6
Shear rigidity GA	kN	2.3×10^6	3.7×10^6	5.5×10^6
Torsional rigidity GJ	kN-m^2	2.3×10^4	1.1×10^5	3.9×10^5
Axial rigidity EA	kN	5.5×10^6	8.9×10^6	1.3×10^7

Table 4. Scenarios of numerical simulation for examination of influence on various factors

Scenario no.	Axial load (P)	Ground Inclination (a)	Pile spacing (S)	Pile diameter (D)
	kN	deg	m	m
P1-A1-S1-D1	1500	0	1.8	0.6
P2-A1-S1-D1	3000	0	1.8	0.6
P3-A1-S1-D1	4500	0	1.8	0.6
P1-A2-S1-D1	1500	5	1.8	0.6
P1-A3-S1-D1	1500	10	1.8	0.6
P1-A1-S2-D1	1500	0	2.4	0.6
P1-A1-S3-D1	1500	0	3.0	0.6
P1-A1-S1-D2	1500	0	1.8	0.8
P1-A1-S1-D3	1500	0	1.8	1.0

On the other hand, the increase in ground inclination would increase the maximum lateral displacement. These results are consistent with the influence due to various factors. Fig. 7 shows the maximum values are obtained at the pile head and decrease with depth for all ground inclinations. The maximum lateral displacement at ground inclination a = 10° are about 26% greater than that at level ground. However, the lateral displacement at ground inclination a = 5° increases by approximately 10%. The trend of lateral displacement is due to the soil energy dissipation and the effect of ground inclination.

Fig. 8 shows the bending moment increases as slope angle increases. For a = 5°, the bending moment in pile increases by about 10% than that of the level ground at pile head. When the inclination a = 10°, the bending moment would generally increase approximately 29% than that of the level ground. The bending moment generally increase from pile tip to pile head. As soil type changed in the profile, the density and consistency of soils would influence the propagation of input motion, and in turn, influence the responses in the pile. The bending moments appear to have localized maxima at the interface of liquefied and non-liquefied soils (i.e., depths of 8 to 12-m), implying the soil type and liquefied layer would have a significant influence on the soil-pile responses.

To assess pile buckling in liquefiable ground using numerical approach, we compare the computed maximum bending moment profiles M_N from numerical simulations with the axial loads vs. bending moment interaction diagram obtained from the design charts for the project piles, which provides the ultimate bending moments M_D of the piles with different bending rigidities and pile diameters, as shown in Fig. 9. When the computed maximum bending moment M_N is acting more severe than the ultimate value of M_D (i.e., $M_N > M_D$), the pile will buckle; otherwise, the pile will be safe (i.e., $M_N \leq M_D$). We observe that the piles in the project site would be safe from bending or buckling failure, as shown in Table 5. This finding generally supports the results of deterministic approach, indicating both numerical and deterministic approaches predict consistently that the pile will be safe from buckling in liquefiable ground due to design earthquake loading.

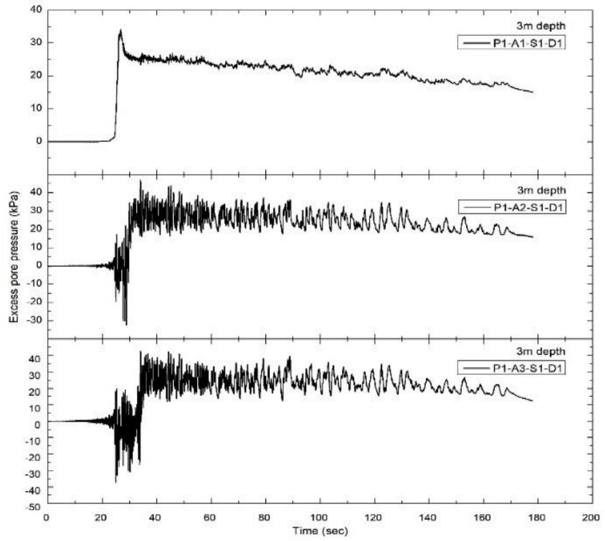

Figure 5. Excess pore pressure time histories with different ground inclinations (0deg/A1, 5deg/A2, 10deg/A3) at 3-m deep for sands in near-field (the initial effective stress is 30 kPa).

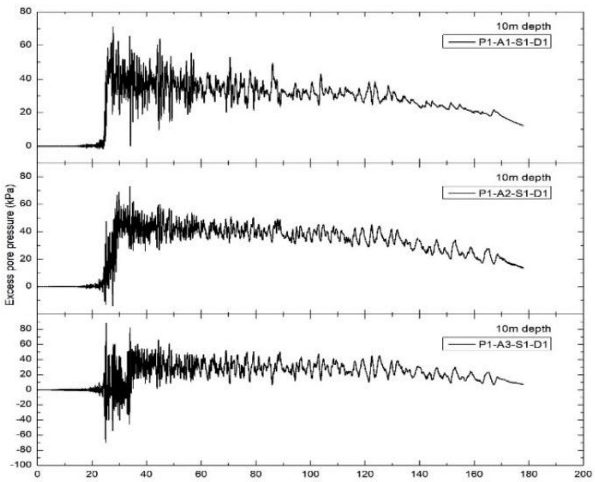

Figure 6. Excess pore pressure time histories with different ground inclinations (0deg/A1, 5deg/A2, 10deg/A3) at 10-m deep for clays in near-field (the initial effective stress is 71 kPa).

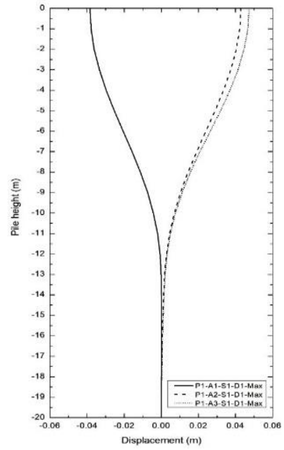

Figure 7. Max. lateral displacement profiles during shaking with different ground inclinations (0deg/A1, 5deg/A2, 10deg/A3)

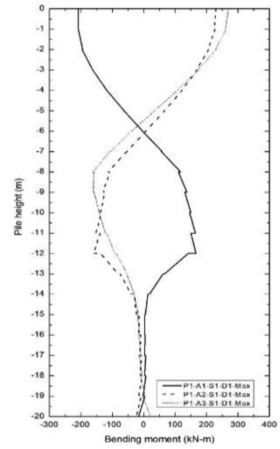

Figure 8. Max. bending moment profiles during shaking with different ground inclinations (0deg/A1, 5deg/A2, 10deg/A3)

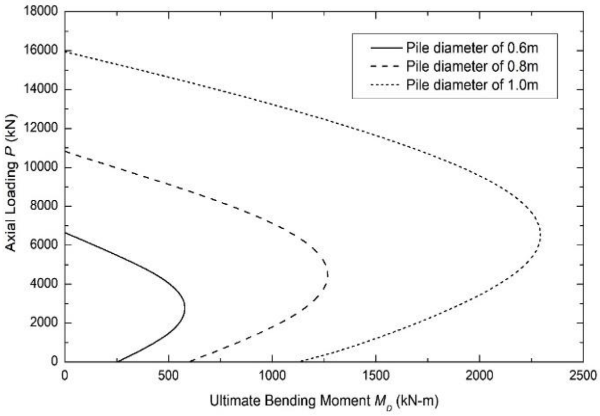

Figure 9. The axial loading vs. bending moment interaction diagrams for the project piles with various diameters (WIKA 2016)

Table 5. Comparison assessment results by computed bending moment (M_N) and ultimate bending moment (M_D)

Scenario no.	Pile axial loading (P)	Pile Diameter (D)	Computed bending moment (M_N)	Ultimate bending moment (M_N)	Remarks
	kN	m	kN-m	kN-m	
P1-A1-S1-D1	1500	0.6	208	510	No buckling
P2-A1-S1-D1	3000	0.6	206	600	No buckling
P3-A1-S1-D1	4500	0.6	201	460	No buckling
P1-A2-S1-D1	1500	0.6	229	510	No buckling
P1-A3-S1-D1	1500	0.6	269	510	No buckling
P1-A1-S2-D1	1500	0.6	236	510	No buckling
P1-A1-S3-D1	1500	0.6	278	510	No buckling
P1-A1-S1-D2	1500	0.8	511	940	No buckling
P1-A1-S1-D3	1500	1.0	1000	1625	No buckling

6 CONCLUSIONS

This study discusses the computation and assessment of liquefaction potential of foundation soils and buckling instability of pile, as well as to explore the effects of various factors of axial load, ground inclination, pile spacing and diameter on the responses of pile and surrounding soil due to seismic loading. This study sets out to compare deterministic and numerical simulation's approaches to gain better understanding on the buckling of piles in liquefiable ground due to seismic loading. Some key points and findings of the study are summarized as follows:

- Liquefaction analysis indicates the foundation soils are prone to liquefaction due to design earthquake, with more than 50% of the boreholes assessed showing $LPI > 5$ (high to very high liquefaction potentials).
- An increase in axial loading would generally increase the excess pore pressure, which in turn would slightly reduce the spectral acceleration and increase the predominant period of soil during seismic loading. An increase in axial loading would not magnify in bending moment, due to the geometry of pile and type of soil which might be insensitive to axial load.
- An increase in ground inclination would generally increase the spectral acceleration and excess pore pressure. When the ground inclination is higher enough, the kinematic force in soil and the soil energy would dissipate more quickly and would induce increase in the maximum lateral displacement and bending moment in pile.
- An increase in pile spacing would generally reduce the responses of acceleration and excess pore pressure in soils and thus would gradually increase the maximum lateral displacement and bending moment in pile, as a result of more soil volume along the piles.
- An increase in pile diameter would generally increase the acceleration and spectral amplitude responses in soils. An increase the diameter of pile would generally increase the rigidity of the pile and would tend to resist more energy released in liquefied soil layers during seismic loading. An increase in pile diameter would slightly decrease the deflection (curvature) of the pile and increase the maximum bending moment in pile.
- Comparisons of the evaluation using deterministic and numerical approaches indicate the piles with various factors will be safe from buckling failure, with computed buckling index $I_B \geq 1.0$ and the computed maximum bending moments less than the ultimate bending moments of the piles (i.e., $M_N \leq M_D$).

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