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ANALYSIS OF BORED PILE FOUNDATION IN POTENTIALLY LIQUEFIED SOIL (CASE STUDY: ANUTAPURA MEDICAL CENTER BUILDING)

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ABSTRACT
Pile foundations placed until hard soil layer and passed through a liquefied layer can be a mitigation effort against liquefaction hazards to buildings. Nevertheless, liquefaction can still impact the stability of the pile. The Anutapura Medical Center (AMC) building at the Anutapura General Hospital complex, Palu city, is a building that is planned to be built on potentially liquefied soil. The foundation of the building was planned to use bored pile foundations to mitigate the possibility of liquefaction. This study aims to analyze and compare the stability of the bored pile group foundation of the AMC building under non-liquefaction and liquefaction soil conditions. The study was conducted by manually calculating the bearing capacity of the bored pile based on soil data. Further analysis was carried out by modeling the pile foundation using Geo5 Pile Group to determine the deformation and internal forces acting on the pile group. The analysis was carried out in 2 cases, i.e., non-liquefaction and liquefaction conditions. The results show differences in the bearing capacity, deformation, and internal forces in non-liquefaction and liquefaction soil conditions. The study results are expected to be a reference and consideration in designing pile foundations in liquefaction-prone locations.

Keywords: *Liquefaction, Pile foundation, Bearing capacity, Geo5 Pile Group.*

1. INTRODUCTION
Deep foundations, such as pile foundations, have been widely used worldwide as a solution to transmit structural loads through soft or problematic soils to reach stable hard soil layers [1]. In the case of liquefaction, pile foundations placed to a depth of hard soil through the liquefied layer can mitigate the liquefaction hazard to the building [2–4]. As well known, liquefaction is a phenomenon where the soil loses its stiffness and effective stress and changes its consistency to become liquid due to cyclic loads (earthquakes) [5,6]. However, liquefaction can still affect the stability of the pile foundation and can cause the failure of the above structure if it is not considered in the design process [7]. Many kinds of literature and studies have shown that liquefaction can affect the bearing capacity, displacement, and the enlargement of the bending moment in pile foundations [8–15].
The construction of Anutapura Medical Centre (AMC) Building at Anutapura General Hospital, Palu City, is a building that was planned to be built in a liquefaction-prone location. Based on soil investigation data, the construction area of the AMC Building has

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the potential to liquefy [16]. The foundation of the building is planned to use bored pile foundations placed to a depth of hard soil at an elevation of 43 meters below ground level to mitigate the possibility of liquefaction.
In the current study, the authors aim to analyze and compare the stability of one of the bored pile group foundations of the AMC building in the normal soil condition and the liquefaction soil condition. Conducted by manually calculating the axial capacity of the pile foundation with empirical equations based on soil data. Further analysis was carried out by numerical simulation of the pile group foundation using Geo5-Pile Group [17], a finite element method (FEM) software, to determine the deformation and internal force acting on the pile group foundation. The analysis in the Geo5-Pile Group was carried out in 2 cases. The first case was an analysis in the normal soil condition, while the second was in the liquefaction soil condition.

2. DATA AND METHODS

2.1 Soil and Pile Fondation Data

The soil data at the study location consist of 3 boreholes incorporated with SPT and laboratory test data. Based on the liquefaction potential analysis results, all boreholes in the AMC building area had the potential to liquefy [16]. However, this study used only the BH-01 borehole because it had the worst conditions where the potentially liquefied layer was thickest. In addition, the BH-01 borehole was also dominated by non-cohesive soil, so its bearing capacity will be relatively smaller than the others. Figure 1 shows soil profile data of BH-01 incorporated with the SPT data, potentially liquefied and non-liquefied layers. The soil in BH-01 had the potential to liquefy at a depth of 5-7 meters and 11-17 meters if an earthquake occurred.

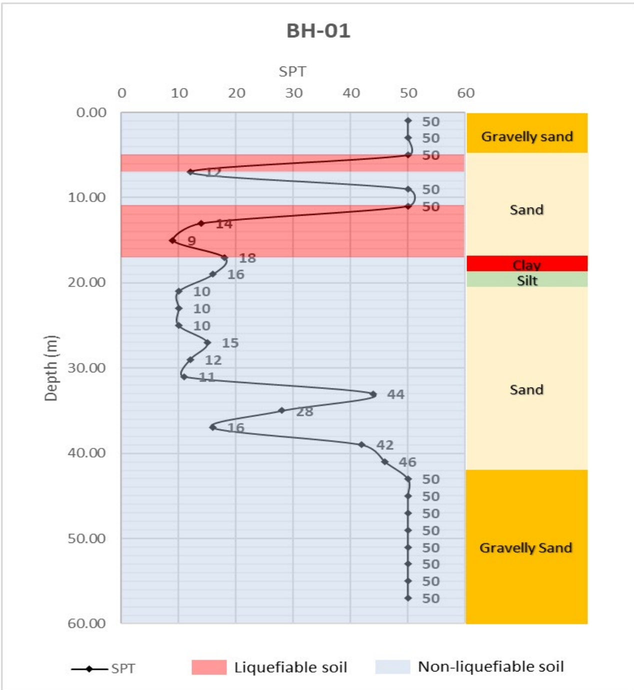

Figure 1. Soil profile data incorporated with the soil type profile, potentially liquefied and non-liquefied layers of BH-01

Figure 2 shows the dimensions and configuration of the bored pile group foundation, which will be analyzed in this study. The pile group foundation consists of 6 piles with a

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net length of 40 meters and a diameter of 0.8 meters. The length of the pile cap is 5.9 meters, the width is 3.5 meters, and the thickness is 1.4 meters.
The base of the pile cap was planned to be at an elevation of 3 meters below the ground surface. The foundation structure used *fc'* 30 concrete, *fy* 520 longitudinal reinforced steel, and *fy* 490 transverse reinforced steel. The foundation was designed to withstand loads under service and seismic conditions, as tabulated in Table 1. Figure 3 illustrates loads on the pile cap in normal and earthquake conditions.

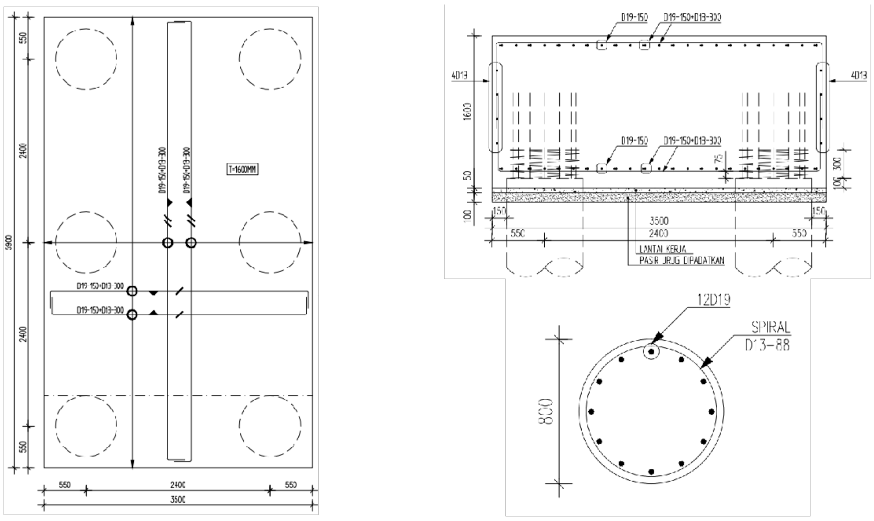
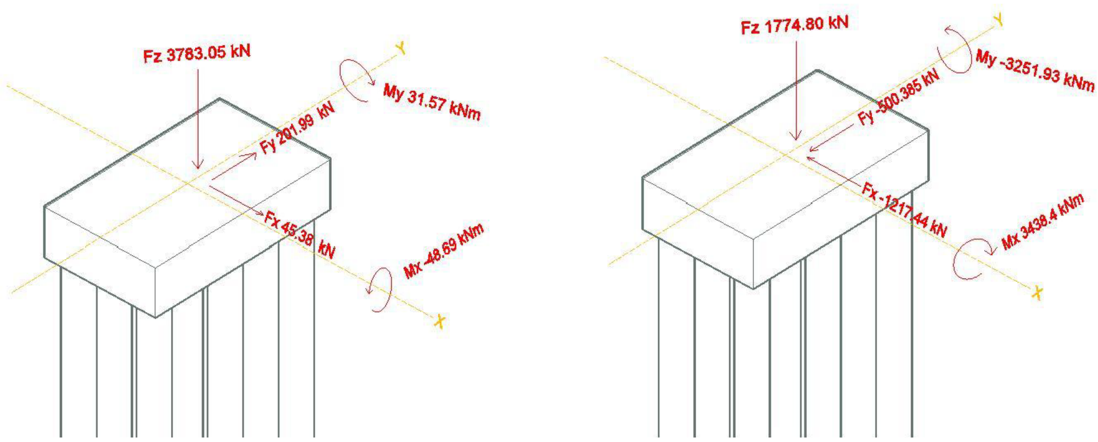


Figure 2. Configuration of pile group foundation

Table 1. Loads on the pile cap during normal and earthquake conditions.

Loads	Fx (kN)	Fy (kN)	Fz (kN)	Mx (kNm)	My (kNm)
Service loads	45.38	201.99	3783.05	-48.69	31.57
Seismic loads	-1217.44	-500.38	1774.80	3438.40	-3251.93



(a) (b)

Figure 3. Illustration of loads on pile cap: (a) service loads; and (b) seismic loads

2.2 Bored pile axial bearing capacity analysis

Analysis of bored pile bearing capacity using empirical static equations developed by Reese and O'Neill [18,19]. The pile end resistance of cohesionless soil per unit area was calculated by the average corrected SPT value from the pile tip up to 2 times the pile diameter, shown in equation (1).

$$f_b = 0.6\sigma_r N_{60} \tag{1}$$

Where f_b is the pile end resistance per unit area, σ_r is a reference stress of 100 kPa, and N_{60} is the average corrected SPT value from the bottom end of the pile up to 2 times the pile diameter.

The pile frictional resistance of cohesionless soil per unit area was calculated based on the beta coefficient, as shown in equations (2)-(4).

$$f_s = \beta P_o' \tag{2}$$

$$\beta = 1.5 - 0.245\sqrt{z}\,;\; N_{60} > 15; 0.25 \leq \beta \leq 1.2 \tag{3}$$

$$\beta = \frac{N_{60}}{15}\left(1.5 - 0.245\sqrt{z}\right) \tag{4}$$

Where f_s is the pile frictional resistance per unit area, β is the coefficient of friction resistance calculation, P_o' is the average overburden pressure in the middle of the soil layer, z is the depth in the middle of the soil layer, and N_{60} is the corrected SPT value.

As for cohesive soils, the pile end resistance and frictional resistance per unit area were calculated using equations (5) to (8).

$$f_b = 9\mu . c_u \tag{5}$$

$$f_s = 0.45 c_u \tag{6}$$

Where c_u is the value of soil cohesion, N_c' is the bearing capacity factor, α is the adhesion factor, and P_r is the reference pressure of 100 kPa.

After obtaining the value of end resistance per unit area and frictional resistance per unit area, the bearing capacity of the bored pile foundation can be obtained using equations (9)-(12).

$$Q_b = A_b f_b \tag{7}$$

$$Q_s = A_s f_s \tag{8}$$
$$Q_u = Q_b + Q_s - W_p$$

$$Q_a = \left(\frac{Q_{ult}}{FS}\right)\eta \tag{9}$$

Where Q_b is the pile ultimate end resistance, A_b is the cross-sectional area of the lower end of the pile, Q_s is the pile ultimate frictional resistance, A_s is the pile blanket area, W_p

is the pile weight, Q_u is the pile ultimate bearing capacity, FS is the safety factor value of 2.5, η is the efficiency factor of the bored pile group and Q_a is the allowable bearing capacity of the pile.
The efficiency factor of the bored pile group referred to the value suggested by Loehr et al. [20] based on spacing (s) and pile diameter (d), as shown in Table 2.

Table 2. The efficiency factor of the bored pile group [20]

Bored pile on	Pile spacing (s)	Efficiency factor (η)
Cohessionless soils	2.5d 4d 2.5d to 4d	0.65 1.00 interpolated between 0.65 to 1.00
Cohesive soils	2.5d 6d 2.5d to 6d	0.65 1.00 interpolated between 0.65 to 1.00

I n the normal condition, all frictional resistances in each layer were fully calculated. While in the liquefaction condition, referring to the Indonesian national standard for geotechnical design (SNI 8460:2017) [13], the frictional resistance of the liquefaction layer was not considered. In addition, negative skin friction (NSF) must be considered due to soil settlement during liquefaction. NSF causes the pile to get an additional drag load, reducing the bearing capacity of the pile. The drag load was estimated based on Prakash [21]. The Q_a in the liquefaction condition was calculated using equations (13) and (14).

$$Q_{NSF} = K_s \sigma'_{vo} \, tan \, tan \, \delta \, A_s \quad (10)$$

$$Q_a = \left(\frac{Q_{ult}}{FS}\right)\eta - Q_{NSF} \quad (11)$$

Where K_s is the lateral soil pressure coefficient of 0.5 for bored pile in cohesionless soil, σ'_{vo} is the average of effective stress, δ is the friction angle between the ground and the pile of 0.5 φ, and Q_{NSF} is the additional drag load due to NSF.

2.3 Numerical model of pile group

Geo5-Pile Group software version 2017.81 [17] was used in this study. The pile group foundation shown in Figure 1 was modeled on the Geo5-Pile Group software. This software can analyze pile group foundations using the finite element method (FEM). The bored pile group foundation embedded in the soil was modeled as a vertical beam supported by a spring constant (spring method). The pile group model in the Geo5-Pile Group can be seen in Figure 4.

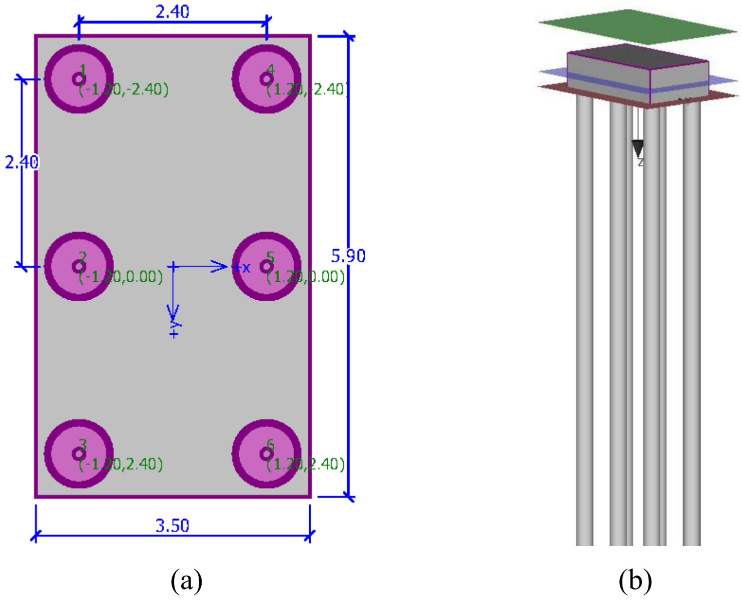


Figure 4. Pile group modeling in Geo5-Pile Group software: (a) pile group configuration; and (b) 3D model

The analysis was carried out in 2 stages. Stage 1 was an analysis in the normal condition, and stage 2 was an analysis in the liquefaction condition. Soil data inputted in the modeling include specific gravity (γ and γ_{sat}), internal friction angle (φ), cohesion (c), modulus of elasticity (E), and lateral modulus of soil reaction (k). The value of k was determined, referring to the value recommended by Bowles [22]. Material quality and pile dimensions were also inputted into the software.

Figure 5 shows the soil profile and assignment in Geo5-Pile Group software. In stage 1, the soil data was inputted according to the normal soil data conditions. In stage 2, the loss and degradation of stiffness in the liquefied layer were represented by reducing the modulus of the soil reaction. The modulus of lateral soil reaction in the liquefied layer was reduced by the reduction factor recommended by Brandenberg [10,11], as shown in Table 3. In addition, the internal friction angle of the soil on the liquefied soil was considered to be zero.

Table 3. Reduction factor of the modulus of soil reaction for liquefaction [10]

$(N_1)_{60cs}$	m_p
< 8	0-0.1
8-16	0.1-0.2
16-24	0.2-0.3
> 24	0.3-0.5

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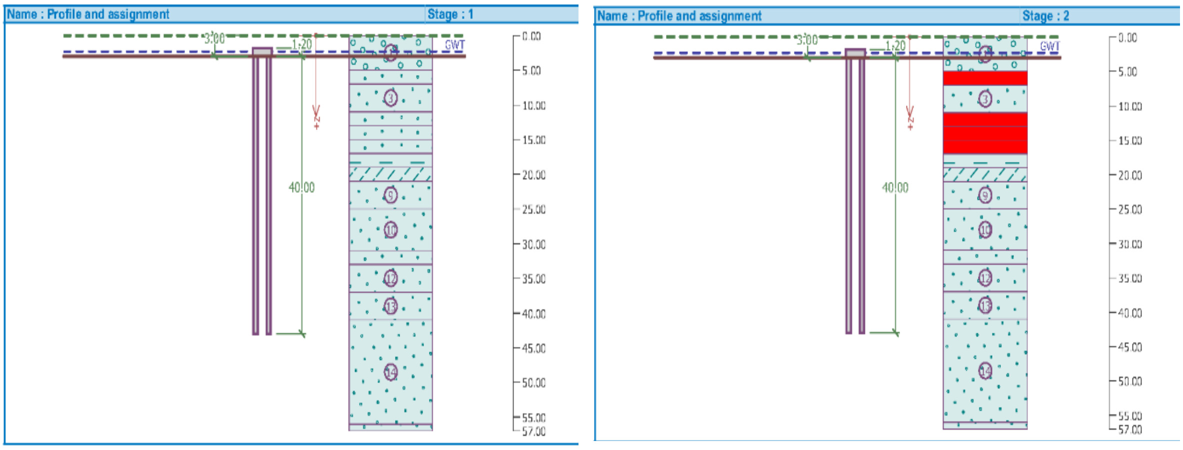


(a) (b)

Figure 5. Soil profile and assignment in Geo5-Pile Group software: (a) normal condition (stage 1); and (b) liquefaction condition (stage 2).

The loads listed in Table 1 were also inputted as the service load received by the pile group foundation. In stage 1, the inputted loads were the loads under service conditions. While in stage 2, the inputted loads were the loads during seismic conditions.
The analysis outputs of Geo5-Pile Group analysis included the deformation of the pile group foundation, the internal forces acting on the pile foundation, and the material capacity of the pile. Then, the deformation of the foundation structure, i.e., the horizontal displacement and vertical displacement (settlement), were checked against the allowable displacement requirements. The internal forces acting on the pile were checked against the bearing capacity of the pile, moment capacity, and shear capacity.

3. RESULT AND DISCUSSION

Analysis of the axial bearing capacity of the bored pile foundation has been carried out under normal and liquefaction conditions. The analysis was carried out up to a depth of 43 meters below the ground surface, the depth of resting the tip of the bored pile foundation. **Table 4** shows the analysis results of the bored pile allowable bearing capacity.

Table 4. The results of the bored pile allowable bearing capacity

Condition	Q_s (kN)	Q_b (kN)	W_p (kN)	Q_u (kN)	Q_{NSF} (kN)	Efficiency factor (η)	Q_a (kN)
Normal	7132.23	1930.97	482.74	8580.46	-	0.77	2631.34
Liquefaction	4712.49	1930.97	482.74	6160.72	-709.12	0.77	1180.16

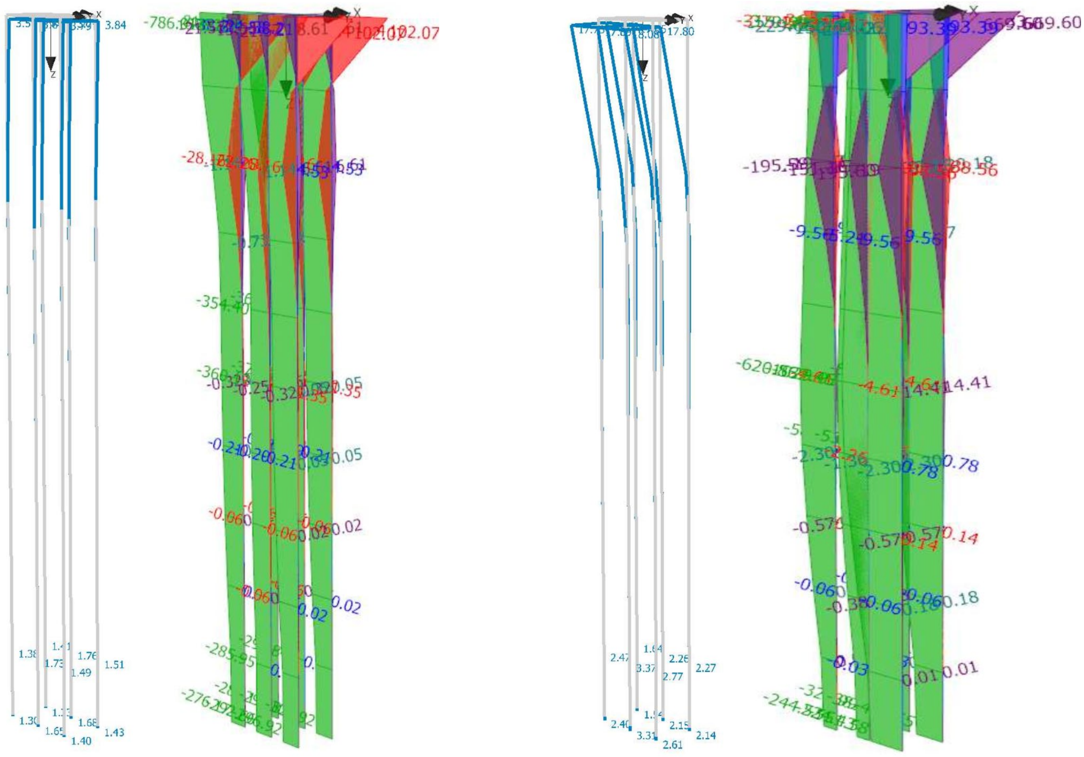
In the normal condition, all the frictional resistance in each layer along the pile, i.e., layers 3-43 meters, were fully calculated. Meanwhile, in the liquefaction conditions, frictional resistances in the potentially liquefied layers, i.e., layers 5-7 meters and layers 11-17 meters, were not considered. In addition, the potentially liquefied layers and the layers above the potentially liquefied layers, i.e., layers 3-17 meters, were considered to provide an additional drag load due to negative skin friction (NSF) of 709.12 kN. Based on the result, the allowable bearing capacity of the bored pile (Q_a) decreased by 55.15% from 2631.34 kN in the normal condition to 1180.16 kN in the liquefaction condition, as shown in Table 4.

The next step in this study was to model and analysis pile group foundations in the FEM Geo5-Pile Group software. As explained in the previous section, the analysis was carried out in 2 stages, i.e., normal (stage 1) and liquefaction (stage 2) conditions. Soil data inputted in Geo5-Pile Group software were tabulated in Table 5. In stage 2, The modulus of lateral soil reactions in the potentially liquefied layers, i.e., 5-7 meters and 11-17 meters, was reduced. The internal friction angle of the soil on the liquefied layers was also considered to be zero, as shown in Table 5.

Table 5. Soil inputted in Geo5 Pile Group

Depth h (m)	γ (kN/m^3)	γ_{sat} (kN/m^3)	c (kN/m^2)	E (MN/m^3)	Stage 1		Stage 2		Description
					φ (°)	k (MN/m^3)	φ (°)	k (MN/m^3)	
0-1	18.60	-	-	70.00	45	400.00	45	400.00	Not liquefied
1-3	18.60	20.00	-	70.00	45	400.00	45	400.00	Not liquefied
3-5	18.60	20.00	-	70.00	45	400.00	45	400.00	Not liquefied
5-7	**14.72**	**17.56**	**-**	**28.70**	**35**	**124.00**	**0**	**26.02**	**Potentially liquefied**
7-9	18.60	20.00	-	70.00	45	280.00	45	280.00	Not liquefied
9-11	18.60	20.00	-	70.00	45	280.00	45	280.00	Not liquefied
11-13	**14.97**	**18.00**	**-**	**29.40**	**36**	**132.00**	**0**	**26.00**	**Potentially liquefied**
13-15	**15.20**	**19.30**	**-**	**26.00**	**34**	**112.00**	**0**	**12.92**	**Potentially liquefied**
15-17	**15.48**	**18.88**	**-**	**30.80**	**37**	**148.00**	**0**	**33.67**	**Potentially liquefied**
17-19	17.68	20.00	112.00	6.38	5	46.21	5	46.21	Not liquefied
19-21	16.88	19.15	70.00	3.50	5	31.72	5	31.72	Not liquefied
21-23	14.47	17.12	-	28.00	35	116.00	35	116.00	Not liquefied
23-25	14.47	17.12	-	28.00	35	116.00	35	116.00	Not liquefied
25-31	14.81	17.71	-	28.93	36	126.67	36	126.67	Not liquefied
31-33	18.60	20.00	-	59.50	43	252.00	43	252.00	Not liquefied
33-37	15.98	19.22	-	32.20	38	164.00	38	164.00	Not liquefied
37-41	18.54	20.00	-	59.50	43	252.00	43	252.00	Not liquefied
41-57	18.60	20.00	-	70.00	45	280.00	45	280.00	Not liquefied

Figure 6 shows the deformation results on the pile group foundation, and Figure 7 shows the critical pile length-horizontal displacement graph. The foundation deformation (i.e., horizontal displacement and settlement) that occurs in the liquefaction condition looks more significant than in the normal condition. Similar results also occur in the internal forces acting on the pile. In the liquefaction condition, the internal forces acting on the pile foundation are bigger than in the normal condition. Table 6 shows the summary of pile foundation displacement results that were checked against allowable requirements. The conclusion compares the displacement and allowable requirements. The analysis is safe when the allowable requirement is bigger than the displacement.



(a) (b)

Figure 6. Deformation and internal forces results: (a) normal condition (stage 1); and (b) liquefaction condition (stage 2).

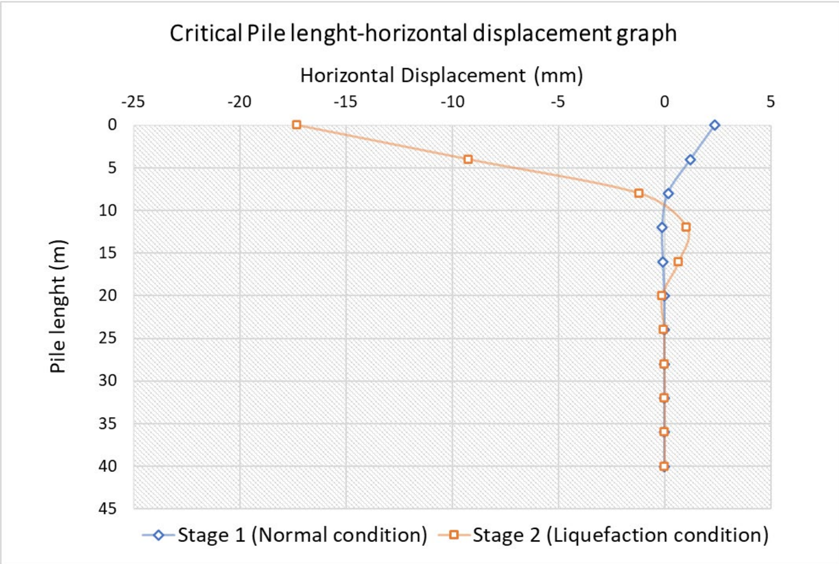


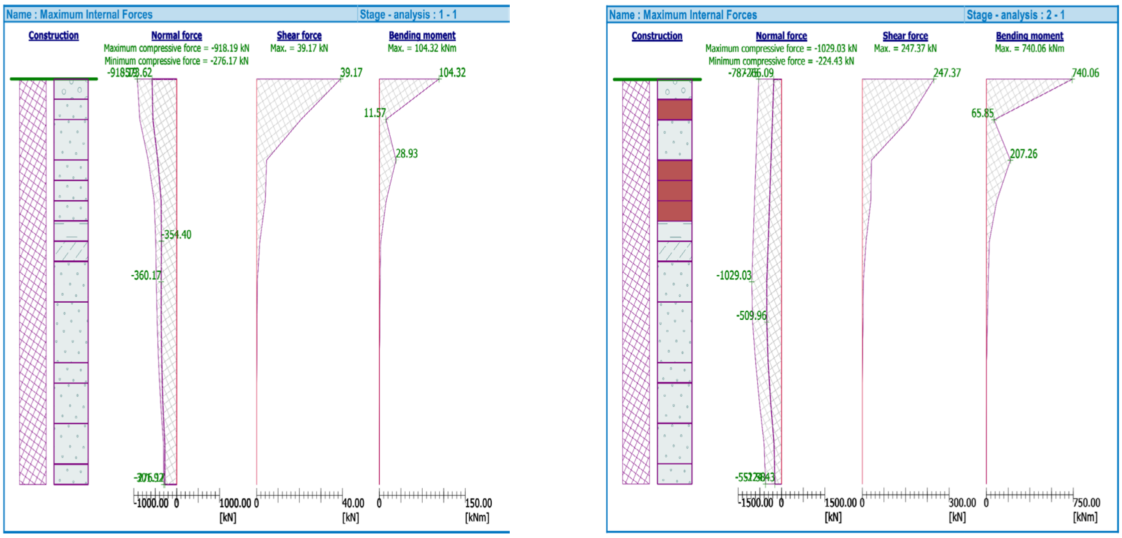
Figure 7. Critical pile length-horizontal displacement graph

Table 6. Summary of pile foundation displacement results and allowable displacement

Displacement	Condition	Result (mm)	Allowable requirement (mm)	Conclusion
Horizontal displacement	Stage 1 (Normal)	2.4	12	Safe
	Stage 2 (Liquefaction)	17.3	25	Safe
Vertical displacement (settlement)	Stage 1 (Normal)	3.0	150	Safe
	Stage 2 (Liquefaction)	5.4	150	Safe

The displacement in the liquefaction condition was more extensive than in the normal condition. Horizontal displacement increased by 620.83%, from 2.4 mm in the normal condition to 17.3 mm in the liquefaction condition. Meanwhile, the foundation settlement increased by 80%, from 3 mm in the normal condition mm to 5.4 mm in the liquefaction condition. The displacement and settlement of the foundation that occurred both in the normal and the liquefaction conditions still fulfill the allowable requirements on the Indonesian national standard for geotechnical design (SNI 8460:2017) [13]. The allowable horizontal displacement when the normal load condition is 12 mm while the seismic load condition is 25 mm. Meanwhile, the allowable settlement is 150 mm in both conditions.

Figure 8 shows the cross-section of maximum internal forces results on piles in the Geo5-Pile Group analysis. The maximum internal force in the liquefaction condition was more significant than in the normal condition. **Table 7** shows the summary results of the maximum internal forces results acting on the pile that was checked against allowable bearing capacity or structure capacity. The conclusion compares the maximum internal force and bearing capacity or material capacity. The analysis is safe when the capacity is bigger than the maximum internal force result.



(a) (b)

Figure 8. Cross-section of maximum internal forces results in Geo5 Pile Group analysis: (a) non-liquefaction condition (stage 1); and (b) liquefaction condition (stage 2)

Table 7. Summary results of the maximum internal force acting on the pile and pile bearing capacity/material capacity

Force	Condition	Result	Bearing capacity/material capacity	Conclusion
Axial force	Normal	P_u = 918.19 kN	Q_a = 2631.34 kN	Safe
	Liquefaction	P_u = 1029.03 kN	Q_a = 1180.16 kN	Safe
Shear force	Normal	V_u = 39.17 kN	ØV_n = 957.31 kN	Safe
	Liquefaction	V_u = 247.37 kN	ØV_n = 957.31 kN	Safe
Bending moment	Normal	M_u = 104.32 kNm	ØM_n = 772.65 kNm	Safe
	Liquefaction	M_u = 740.06 kNm	ØM_n = 772.65 kNm	Safe

Figure 8 and Table 7 show that the maximum compressive axial force (P_u) increased by 12.07% from 918.19 kN in the normal condition to 1029.03 kN in the liquefaction condition. Meanwhile, a significant increase occurred in the shear force and bending moment. The maximum shear force (V_u) increased by 531.53%, from 39.17 kN in the normal condition to 247.37 kN in the liquefaction condition. Meanwhile, the maximum bending moment (M_u) increased by 609.41% from 104.32 kNm to 740.06 kNm. Based on the Geo5-Pile Group analysis, the shear force capacity ($\emptyset V_n$) was 957.31 kN, and the moment capacity ($\emptyset M_n$) was 772.65 kNm. The internal forces both in the normal and liquefaction conditions, still meet the allowable bearing and structure capacity. Based on the result, the bored pile foundation is relatively safe and stable both in normal and liquefaction conditions.

4. CONCLUSION

Analysis of bored pile foundation in potentially liquefied soil at AMC building has been carried out. The soil at the study location had the potential to liquefy at a depth of 5-7 meters and 11-17 meters. The analysis was carried out in 2 cases. The first case was an analysis in the normal soil condition, while the second was an analysis in the liquefaction soil condition.

The bearing capacity analysis results showed a decrease in the allowable bearing capacity of the pile in the liquefaction condition. Meanwhile, the results of pile group analysis using Geo5-Pile Group software showed a significant increase in displacement and internal forces on pile group foundations in the liquefaction condition. The displacement and internal forces, both in the normal and liquefaction conditions, still fulfill the allowable requirements and the capacity of the structure. It can be concluded that the foundation of the bored pile group is relatively safe and stable both in normal and liquefaction conditions. The study results are expected to be a reference and consideration for various parties in designing pile foundations in liquefaction-prone locations.

5. ACKNOWLEDGMENTS

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