Analysis of Bearing Capacity Decrease of Pile Group Foundation in Liquefiable Area

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Abstract. In Indonesia, a region frequently affected by earthquakes, ensuring the seismic safety of building structures is critically important. This safety consideration must extend beyond the superstructure to include the substructure as well. One significant threat to the substructure is the risk of soil liquefaction, which can markedly reduce the bearing capacity of the soil during an earthquake. This research employs a quantitative approach, involving direct field soil exploration and sampling. The collected soil data is used to assess the soil's susceptibility to liquefaction. At the study site, it was determined that the predominant soil type is loose sand, coupled with a high groundwater table. This combination indicates a potential for liquefaction from depths of 1.5 meters to 13 meters. The research model encompasses two groups of piles, both of which demonstrate a reduction in axial bearing capacity by 79%.

Keywords: Bearing Capacity, Liquefaction, Pile Group Foundation

1 Introduction

In Indonesia, a region frequently affected by earthquakes, ensuring the seismic resilience of building structures is crucial (Setiawan et al., 2022). This encompasses not only the superstructure but also the substructure. A major threat to the substructure is soil liquefaction, which can severely compromise the soil's bearing capacity during seismic events.

Liquefaction occurs when saturated soil loses its strength and acts like a liquid due to seismic vibrations (Seed et al., 1983). This phenomenon poses significant risks as it can drastically reduce the soil's load-bearing capacity, directly impacting the stability of foundations, including pile group foundations.

Pile groups are widely used in construction to support substantial loads (Randolph et al., 1994). However, during liquefaction, the surrounding soil may lose its strength, leading to a reduction in the axial bearing capacity of the pile foundations. Numerous studies have investigated the effects of liquefaction on the bearing capacity of pile foundations:

A. A. N. G. Sapteka et al. (eds.), *Proceedings of the International Conference on Sustainable Green Tourism Applied Science - Engineering Applied Science 2024 (ICoSTAS-EAS 2024)*, Advances in Engineering Research 249,
https://doi.org/10.2991/978-94-6463-587-4_32

Seed & Idriss (1971). Developed a predictive model to evaluate the potential for liquefaction and its effects on subterranean structures. They highlighted that significant increases in pore water pressure could sharply decrease soil-bearing capacity during an earthquake.

Tokimatsu & Seed (1985). Research about examined the effects of liquefaction on pile foundations, finding that liquefaction could cause notable lateral soil displacement, reducing the stability of pile foundations.

Boulanger & Idriss (2010). They devised a method to assess the risk of liquefaction and its impact on pile foundations. Their research indicated that loose sandy soils with high groundwater levels are highly susceptible to liquefaction, potentially leading to an 80% reduction in bearing capacity.

Ashford & Rollins (2000). This study demonstrated that pile groups in liquefaction-prone soils experienced significant reductions in bearing capacity. The authors emphasized the necessity of foundation designs that account for liquefaction risks.

The novelty of this study employs a quantitative approach, involving direct field soil exploration and sampling from the field at the study location in Bali-Indonesia, followed by calculations of the bearing capacity of pile group foundations under liquefaction conditions. By quantifying the reduction in bearing capacity, safety factors for building foundations can be determined, incorporating the potential effects of liquefaction.

2 Methodology

The research methodology for this study is outlined as follows: Conduct comprehensive soil testing, including cone penetration tests (CPT), standard penetration tests (SPT), and soil sampling, to determine the stratigraphy of soil layers and to obtain the physical and mechanical properties of the soil. Gather historical earthquake data for the Bali region and develop a designed earthquake scenario based on this data. Perform a susceptibility analysis of the site to assess the risk of soil liquefaction. Analyze the bearing capacity of pile group foundations under static conditions, without considering seismic loads and liquefaction, followed by an analysis that incorporates the effects of seismic loads and liquefaction on the bearing capacity.

3 Result and Discussion

3.1 Soil Test Result

Field soil testing involved a series of CPT (cone penetration test) and SPT (standard penetration test) evaluations. The study conducted CPT and SPT tests at three locations to characterize the soil conditions and stratigraphy. SPT tests were performed to an average depth of 6 meters, while CPT tests reached a depth of approximately 13 meters. The results indicated that the soil at the site consists of loose sand, with low SPT and CPT values. Hard soil was encountered at a depth of around 12 meters SPT values at depths ranging from 2 meters to 6 meters varied between 2 and 4.

Gradation tests conducted at each testing point identified the soil as predominantly sandy, with a clay content of 4% to 5%. The bulk density of the soil ranged from 16.17 to 18.37 kN/m³, with an assumed sand bulk density of 17.5 kN/m³ and a saturated bulk density of 18.5 kN/m³. For hard soil, the assumed saturated bulk density is 19 kN/m³. The groundwater table was observed at a depth of 1.5 meters. The following is a graph of the soil test results at the study site.

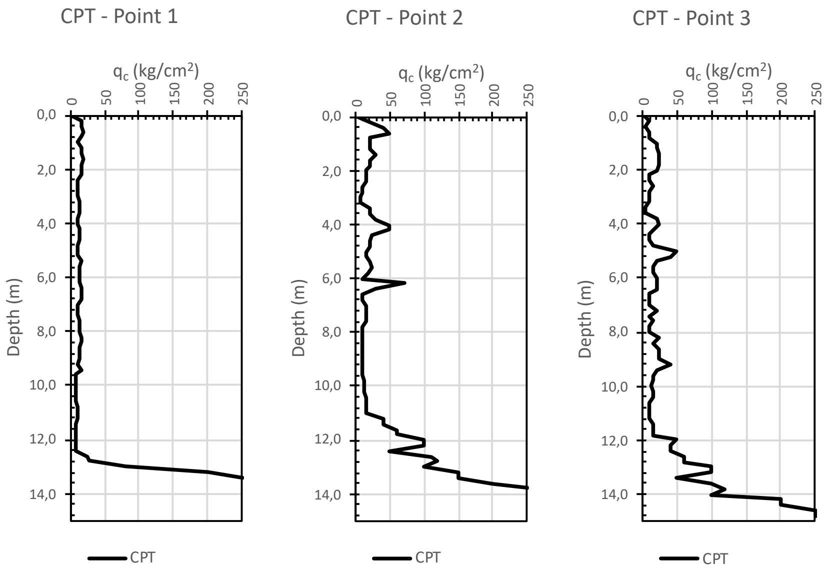

Figure 1. CPT test result

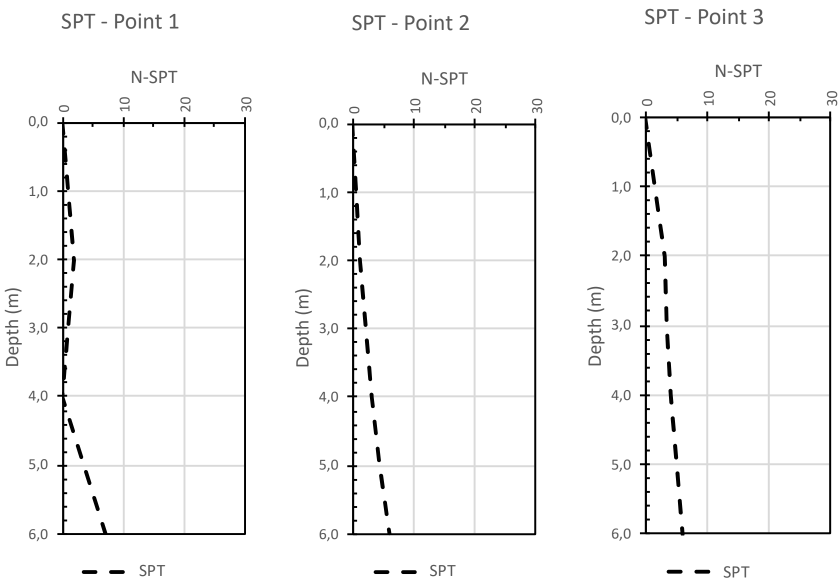

Figure 2. SPT test result

After conducting field tests, it was found that the average SPT value of 3 suggests, as per (Wood, 1990), that the soil's angle of internal friction (φ) at the research site, between depths of 0 to 12 meters, ranges from 25° to 30°. To facilitate calculations, a φ value of 27° is selected. Additionally, for hard soil with qc > 250 kg/m², a friction angle of 38° is applied."

3.2 Liquefaction Susceptible Analysis

The safety analysis of the soil layers against liquefaction is based on planned earthquake magnitude (Kramer & Seed, 1988; Seed et al., 1983), in this study use magnitudes (Mw) of 4, 5, and 6. According to the calculations, the soil layers are stable and will not liquefy under a seismic load of Mw = 4. Liquefaction of the soil layers begins to occur when the seismic load reaches Mw > 5.

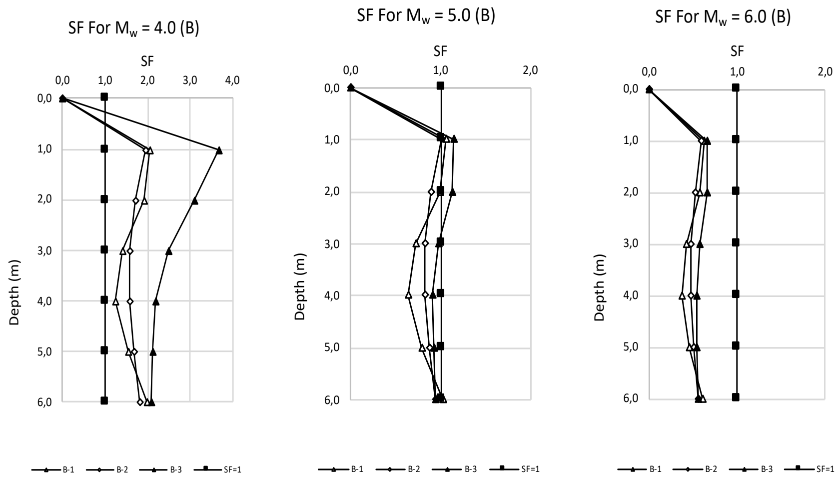

Figure 3. Liquefaction factor of safety vs depth graph

3.3 Bearing Capacity of Pile Group Before Liquefaction

In the analysis of pile group foundations bearing capacity use the method by (Das, 2017), it will model two types of pile group foundations: one where the bottom of the piles rests on hard soil, with 4 piles for type 1 and 9 piles for type 2. Input parameter and configuration of the pile group are shown in Table 1.

Table 1. Specification and configuration of the pile group

Foundation type	Pile diameter	Length	Number of pile	Distance between pile
1	30 cm	15 m	4	70 cm
2	30 cm	15 m	9	70 cm

Bearing capacity calculation

End Bearing. Effective Vertical stress calculation as stated in Equation (1):

$$\sigma'_b = \sigma_b - u \tag{1}$$

$$\sigma'_b = (17.5 \times 1.5) + (18.5 \times 11.5) + (19 \times 2) - (10 \times 13.5) = 142\ KN/m^2$$

Internal angle of friction in this calculation is assumed 36°.
End bearing capacity of the single pile as stated in Equations (2)

$$Q_b = A_b\sigma_b(N_q - 1) \qquad (2)$$

$$Q_b = 0.07\ x\ 142\ x\ (70 - 1) = 1371\ KN$$

Skin Resistance. The angle of friction between soil and pile is assumed ¾ of the soil internal friction. Shear stress at any distance on the soil is calculated as stated in Equations (3):

$$\tau_s = K_s\ \sigma_v\ tan\,\delta_{cv} \qquad (3)$$

$$\tau_s = K_s\ \sigma_v\ tan\,\delta_{cv} = 1\ \times\ 17.5z\ tan\,21 = \ 6.71\ z$$

$$\tau_s = K_s\ \sigma_v\ tan\,\delta_{cv} = 1\ \times\ (18.5 - 10)z\ tan\,21 = 3.26\ z$$

$$\tau_s = K_s\ \sigma_v\ tan\,\delta_{cv} = 2\ \times\ (19 - 10)z\ tan\,27 = 9.17\ z$$

Skin Resistance can be calculated as stated in Equations (4):

$$Q_s = \pi \text{d}\ \times\ \left(\int_0^{1,5} 6.71\ z\ dz + \int_{1.5}^{13} 3.26\ z\ dz + \ \int_{13}^{15} 9.17\ z\ dz\ \right) \qquad (4)$$

$$Q_s = 504.7\ KN$$

Thus, the total of the bearing capacity og the single pile as stated in Equations (5):

$$Q_u = Q_b +\ Q_s \qquad (5)$$

$$Q_u = 1371 + 504.7\ = 1875\ KN$$

End bearing capacity of the pile group before liquefaction can be calculated as follows:

Type 1 Pile Group Foundation: (four pile configuration)
Efficiency of the pile group as stated in Equations (6) and (7):

$$E_g = 1 -\ \theta\,\frac{(n-1)m+(m-1)n}{90mn} \qquad (6)$$

$$E_g = 1 -\ 23.2\ \frac{(2-1)2 + (2-1)2}{90 \times 2 \times 2}$$

$$E_g = 0.742$$

$$Q_{u(gab)} = E_g\ \times\ 4\ \times\ Q_u \qquad (7)$$

$$Q_{u(gab)} = 0.742\ \times\ 4\ \times\ 1875\ KN$$

$Q_{u(gab)} = 5.565\ KN$

Type 2 Pile Group Foundation: (nine pile configuration)
Efficiency of the pile group:

$$E_g = 1 - \theta \frac{(n-1)m + (m-1)n}{90mn}$$

$$E_g = 1 - 23.2 \frac{(3-1)3 + (3-1)3}{90 \times 3 \times 3}$$

$$E_g = 0.656$$

$$Q_{u(gab)} = E_g \times 4 \times Q_u$$

$$Q_{u(gab)} = 0.656 \times 9 \times 1875\ KN$$

$$Q_{u(gab)} = 11.070\ KN$$

The tabulation calculation of the pile group foundation before and during liquefaction is shown in Table 2.

Table 2. Tabulation of bearing capacity of pile group foundation

Pile Bearing Capacity	Type 1 Foundation (four piles)	Type 2 foundation (nine piles)
Before Liquefaction	5.565 KN	11.070 KN
During Liquefaction	1.158 KN	2.303 KN
Decrease	4.407 KN (79%)	8.767 KN (79%)

3.4 Bearing Capacity of Pile Group Before Liquefaction

Table 2. Bearing capacity calculation

Pile bearing capacity	Type 1 foundation (four piles)	Type 2 foundation (nine piles)
Before Liquefaction	5.565 KN	11.070 KN
During Liquefaction	1.158 KN	2.303 KN
Decrease	4.407 KN (79%)	8.767 KN (79%)

3.5 Discussion

Based on the calculations above for the pile foundation's bearing capacity, it was determined that pile group type 1 experiences a 79% decrease in bearing capacity. Before liquefaction, its capacity is 5.565 kN, reducing to 1.158 kN during liquefaction. Similarly, pile group type 2 also sees a 79% reduction, with capacities of 11.070 kN and 2.303 kN respectively, before and after liquefaction.

These calculations show that varying the number of piles with the same diameter, depth, and spacing results in different efficiency values. However, the percentage decrease in bearing capacity for both types 1 and 2 pile groups remains consistent during liquefaction events. This is because the pile depths in the calculations are uniform, leading liquefaction to reduce the pile strength by the same proportion.

4 Conclusion

According to the result and discussion above, it can be concluded as follows: The soil layers at the test site are predicted to experience liquefaction during earthquakes with magnitudes starting from Mw = 5, at depths ranging from -1.5 meters to -13 meters. The axial bearing capacities of pile group foundations, Type 1 and Type 2, before liquefaction, are 5.565 kN and 11.070 kN, respectively. During liquefaction, the bearing capacities of pile group foundations, Type 1 and Type 2, are reduced to 1.158 kN and 2.303 kN, respectively. The reduction in bearing capacity for pile group foundations, Type 1 and Type 2, during liquefaction, is approximately 79%.

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Acknowledgment

I am deeply grateful to Politeknik Negeri Bali for sponsoring my research. Their financial support and resources have been invaluable. Special thanks to civil engineering department for their encouragement and guidance. This work would not have been possible without their generous support.

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