Substructure Optimization and Bored Pile Group Performance of Cihanjuang Bridge on Steep Volcanic Slopes

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ABSTRACT

West Bandung Regency is focusing on infrastructure development, especially in mountainous areas like Parongpong, which struggles with steep terrain and uneven growth. The valley between Parongpong and Cisarua leads to traffic congestion and inconsistent land values. Surveys reveal untapped ecotourism potential in the area. A 200-metre-long, 10-metre-wide connecting bridge is planned between Cisarua KM 4+800 and Parongpong KM 5+500 at an elevation of 950 metres. Due to the landslide-prone zone, the project emphasizes the design of the bridge's foundation and piers. A1 measures 6 metres in height, 12 metres in width, and 14 metres in depth, with 10 bored piles for improved load capacity, shear, and overturning stability. A2 is 5 metres high, 12 metres wide, and 14 metres deep, with 8 bored piles providing similar stability. P1 and P2 are designed at 25 metres in height with 12 bored piles as deep foundations. The sub-structure, with a safety factor of 1.3, has sufficient stability against the loads. Increasing pile diameter enhances group capacity, while optimal pile spacing prevents stress interference and ensures better load distribution and efficiency.

Keywords: *Potential, Design, Substructure, Bridge, Foundation.*

1. INTRODUCTION

Bridges are essential for connecting roads separated by obstacles such as ravines, rivers, or railways. Beyond their structural function, bridges play a significant role in supporting transportation infrastructure, contributing to economic, social, and environmental development [1]. As regions develop, the need for bridges increases. A strong transportation system enhances access by boosting capacity, reducing travel time and costs, and supporting poverty reduction and economic development, especially in rural and hilly areas [2]. In these regions, bridges help transport people and goods using bicycles, livestock, and carts. West Bandung Regency faces similar challenges, with valleys and rivers obstructing direct access.

The local government has prioritized infrastructure development to drive economic growth, improve quality of life, and strengthen regional connectivity. This is particularly relevant in Parongpong District, located at 947 meters above sea level with slopes exceeding 40%. A valley separates Parongpong (east) and Cisarua (west). Currently, residents use a 7 km detour through Cimahi, taking 25–30 minutes depending on traffic. This roundabout route creates transportation inefficiency. A new bridge over the Cihanjuang Valley is needed to improve connectivity. According to the Cimahi City Transportation Agency, traffic volume reaches up to 1,000 vehicles per hour, with congestion and long queues during peak hours due to high traffic at intersections [3].

Based on survey results, a bridge location was selected between KM 4+800 in Cisarua and KM 5+500 in Parongpong. Soil investigations show slopes over 40%, requiring special treatment. Landslide risk mapping indicates the site is in a vulnerable zone. As such, this study focuses on designing the bridge's substructure and piers. The planning stages include selecting potential sites, determining the final location, mapping abutments and approach roads, conducting CPT tests, and performing structural analysis for foundations and piers. Explicitly formulates three core research questions: (1) how steep slope geometry (> 40%) and high seismic risk affect the ultimate bearing capacity and lateral stability of bridge substructures; (2) what foundation mechanism is required to restore geotechnical safety when shallow spread footings exhibit critical bearing capacity deficiencies; and (3) how variations in bored pile diameter (D = 0.5–1.0 m) and center-to-center spacing (S = 2.0–3.2 m) influence pile group efficiency and load distribution under rigid pile cap conditions in volcanic clay. Although the general mathematical relationship between pile spacing and group efficiency is well established, existing literature has rarely quantified the trade-offs among pile group efficiency reduction, seismic horizontal thrust, and slope-modified bearing capacity in soft-to-medium volcanic clay profiles. Accordingly, the novelty of this study lies in establishing an optimal design envelope for deep foundations on a 40% slope that minimizes unnecessary geotechnical overdesign while satisfying the 3S criteria (Stable, Stiff, and Strong) under loading conditions specified in SNI 1725:2016.2.

2. LITERATURE REVIEW

Previous studies related to the design of bridge substructures serve as essential references for this research. Reference [4] shows conducted a detailed design of the substructure for the Gunung Bentang–Padalarang Bridge, connecting Jayamekar Village and Bojonghaleuang Village in West Bandung Regency. The study aimed to produce a structural design that aligns with local soil conditions and traffic loads. Reference [5] performed a redesign of the abutment substructure using bored pile foundations on the Solo–Kertosono Toll Road Project, Section I, located in Karanganyar, Central Java. The main objective was to determine the foundation's bearing capacity as well as the optimal diameter and number of bored piles required. Reference [6] focused on designing abutments, foundations, and piers for a cable-stayed bridge located on the Semarang–Solo Toll Road, precisely in the Lemah Ireng section. The study emphasized the development of an efficient substructure design that fits the geotechnical and structural requirements of a cable-stayed system.This study highlight the importance of detailed geotechnical analysis, appropriate foundation selection, and structural efficiency in substructure design.

3. METHODOLOGY AND ANALYTICAL FORMULATIONS

3.1. CPT-Based Single-Pile Bearing Capacity

The ultimate axial capacity of a single bored pile is expressed as the sum of its base resistance and shaft resistance, as presented in Eq. (1). In Eq. (1) Q_u is the ultimate axial capacity of the pile, Q_b is the base resistance, Q_s is the total shaft resistance, q_b is the unit base resistance, A_b is the pile-base area, U is the pile perimeter, $f_{s,i}$ is the unit shaft resistance within soil layer i, and ΔL_i is the pile length within that layer. The unit base resistance is estimated from cone penetration test (CPT) data using the empirical approach associated with Meyerhof [7] and Schmertmann and Nottingham [8]. The representative cone resistance is evaluated over a specified influence zone above and below the pile tip, following the requirements of the selected CPT-based method.

$$Q_u = Q_b + Q_s = q_b A_b + U \sum_{i=1}^{n} f_{s,i} \Delta L_i \tag{1}$$

3.2. Pile-Group Efficiency

The axial capacity of a pile group depends on the capacity of individual piles and the interaction between adjacent piles. Group efficiency is evaluated using the Converse–Labarre method as presented in Eq. (2). In this formula, $E_g$is the pile-group efficiency factor, m is the number of pile rows, n is the number of piles per row, and θ is the interaction angle, expressed in degrees. The resulting group capacity is compared with the applied vertical load to assess the adequacy of each configuration. This comparison represents a capacity-to-demand ratio and should be distinguished from a factor of safety defined using a separate geotechnical design criterion.

$$E_g = 1 - \theta \frac{(n-1)m + (m-1)n}{90 \times m \times n} \tag{2}$$

3.3. Seismic Active Earth Pressure

Seismic active earth pressure is evaluated using the Mononobe–Okabe method, which extends the Coulomb earth-pressure framework to account for horizontal and vertical seismic effects. The total active earth force per unit length of retaining structure is expressed in Eq. (3). In Eq. (3), P_{AE} is the resultant seismic active earth force per unit length, $\gamma$is the soil unit weight, H is the retained height, k_v is the vertical seismic coefficient, and K_{AE} is the seismic active earth-pressure coefficient. The coefficient K_{AE} is calculated using the Mononobe–Okabe formulation, with the applicable soil friction angle, wall friction angle, backfill inclination, wall inclination, and seismic coefficients. The equation must be implemented using a single, internally consistent sign convention for all angles.

$$P_{AE} = \frac{1}{2}\gamma H^2 (1 - k_v) K_{AE} \tag{3}$$

3.4. Bridge Load Combinations

The foundation design assessment considers the load combinations specified in SNI 1725:2016 as follows.

- Strength I : U=1.20MS+1.40MA+1.80TD+1.00TB+1.50TA
- Extreme I **:** U=1.00MS+1.00MA+0.50TD+1.00EQ+1.00TA

where U is the factored load effect; MS denotes the self-weight of structural components; MA denotes additional dead loads; TD denotes the applicable traffic load; TB denotes braking forces; TA denotes the relevant additional load component; and EQ denotes

earthquake effects. The definitions and load factors for each symbol must follow the applicable load-combination provisions. The combinations above should be checked against the exact load cases used in the structural analysis before being adopted as the final design combinations.

3.5. Distribution of Axial Loads among Piles

The axial load acting on an individual pile within a rigid pile cap is estimated using the elastic load-distribution approach. Assuming that the pile cap behaves as a rigid body and that the piles have comparable axial stiffness, the load on pile i is expressed in Eq. (4). where P_i is the axial load on pile i, P_{total} is the total vertical load transferred to the pile group, N is the total number of piles, M_x and M_y are the overturning moments about the respective axes, and x_i and y_i are the coordinates of pile i relative to the pile-cap centroid. The total vertical load includes the relevant structural reactions and foundation-related loads included in the design model. The signs of the moment contributions depend on the coordinate convention and the direction of the applied moments. The maximum calculated pile load is compared with the allowable axial capacity of a single pile. The assessment also considers settlement and lateral displacement to evaluate the serviceability performance of the proposed pile groups.

$$P_i = \frac{P_{total}}{N} + \frac{M_y x_i}{\sum_{j=1}^{N} x_j^2} + \frac{M_x y_i}{\sum_{j=1}^{N} y_j^2} \quad (4)$$

4. ANALYSIS AND DESIGN

The Cihanjuang Bridge is planned as a Class B upper-arch steel bridge at an elevation of 960 meters above sea level, with a total span of 200 meters supported by two abutments and two piers. This final project focuses on the substructure design, which includes the abutments, foundations, and piers. The design adheres to the 3S principles (stable, stiff, strong) and involves load analysis, abutment stability assessment based on soil bearing capacity, pile capacity calculations using soil investigation data, load distribution among piles, and pier design [9]. The bridge layout is illustrated through plan, longitudinal, and cross-sectional views (Figure 1).

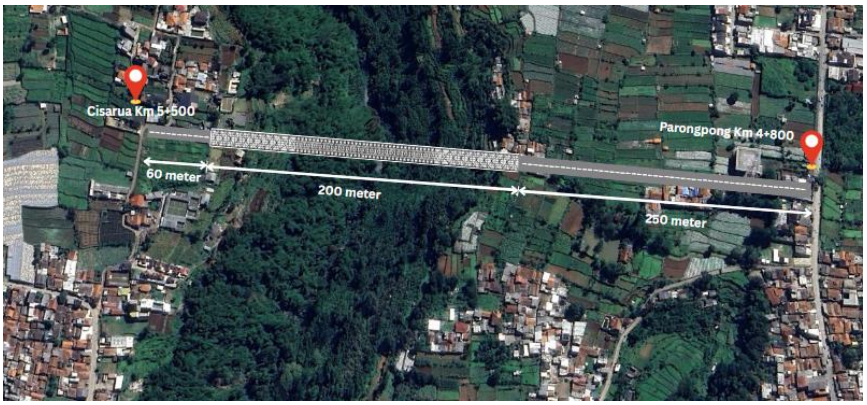


Figure 1. Plan view of the Cihanjuang bridge

4.1 Soil Data

Cone penetration tests (CPT) were conducted at two locations Cihanjuang and Padaasih with two test points at each site [10]. The testing was carried out on February 26, 2025, in accordance with SNI 2827:2008 standards [11]. The cone penetration test results from points S-01 and S-02 on the Cihanjuang and Padaasih slopes were used in this design. The soil types were classified based on cone resistance and friction ratio values, following J.E. Bowles' method [12]. The investigation revealed that the dominant soil type in the area is clay. Based on the cone penetration test results presented above, hard soil layers were identified at the following depths and cone resistance (q_c) values: at Point S1 Cihanjuang, hard soil was found at a depth of 16 meters with a q_c value of 147.7 kg/cm²; at Point S2 Cihanjuang, at 17 meters with a q_c of 146.7 kg/cm²; at Point S1 Padaasih, at 16 meters with a q_c of 152.9 kg/cm²; and at Point S2 Padaasih, at 17 meters with a q_c of 137.8 kg/cm². The soil stratigraphy of the Cihanjuang Valley is illustrated in Figure 2 and Soil Data Sample shown in Table 1.

4.2 Superstructure

Figure 3 shows the superstructure of the Cihanjuang Bridge, that is formed by an arch steel bridge. The length of the main span is 100 m, while the length of the supporting span on the left and right side of the bridge is 50 m each. The bridge is 12m in width, supported by bored pile foundation as its substructure. During the structural analysis, parameters used related to the material

properties were set to 2.4 T/m^3 for concrete unit weight (γ_c), 2.2 T/m^3 for asphalt unit weight (γ_{wc}), as well as 0.981 T/m^3 for water unit weight (γ_w). The superstructure load calculation for the Cihanjuang Bridge refers to SNI 1725:2016 regarding bridge loading. The loads acting on the short span are described in Table 2. Furthermore, the loads acting on the long span are presented in Table 3.

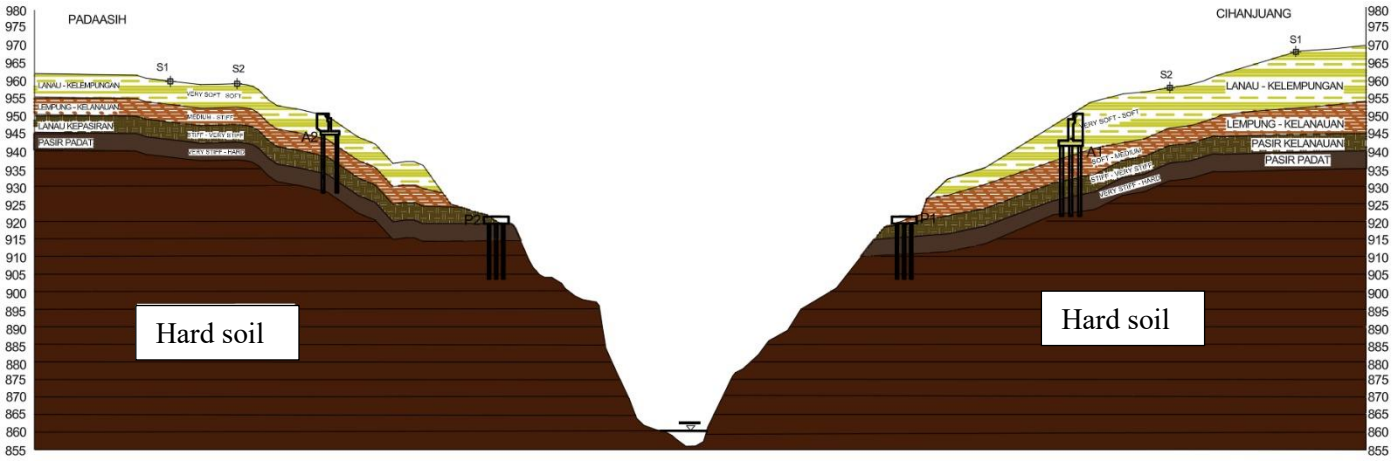


Figure 2. Geological stratigraphy profile

Table 1. Soil data sample

Soil Sample 1			Soil Sample 2		
Parameter	Value	Unit	Parameter	Value	Unit
γ_1	1.86	T/m^3	γ_1	1.74	T/m^3
C_1	0.41		C_1	0.24	
ϕ_1	15.48	°	ϕ_1	26.57	°

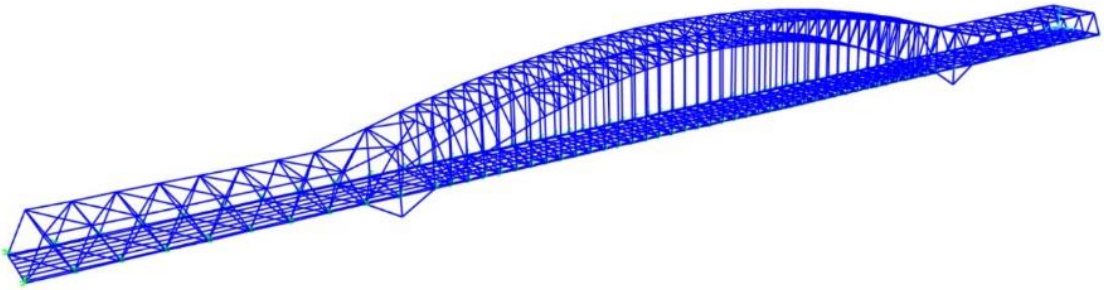


Figure 3. 3D View of the Cihanjuang bridge

table 2. Total load for short span.

Load Type	Vertical	Horizontal	
	P (Ton)	Tx (Ton)	Ty (Ton)
Superstructure Self Weight (SW)	114.62		
Dead Load (DL)	28.89		
Live Load (LL)	10.29		
Braking Force (TB)		10.6	
Wind load on structure (E_{ws})			5.63
Wind Load on Vehicle (E_{WI})		1.8	
Total	153.80	12.4	5.63

Table 3. Total load for long span

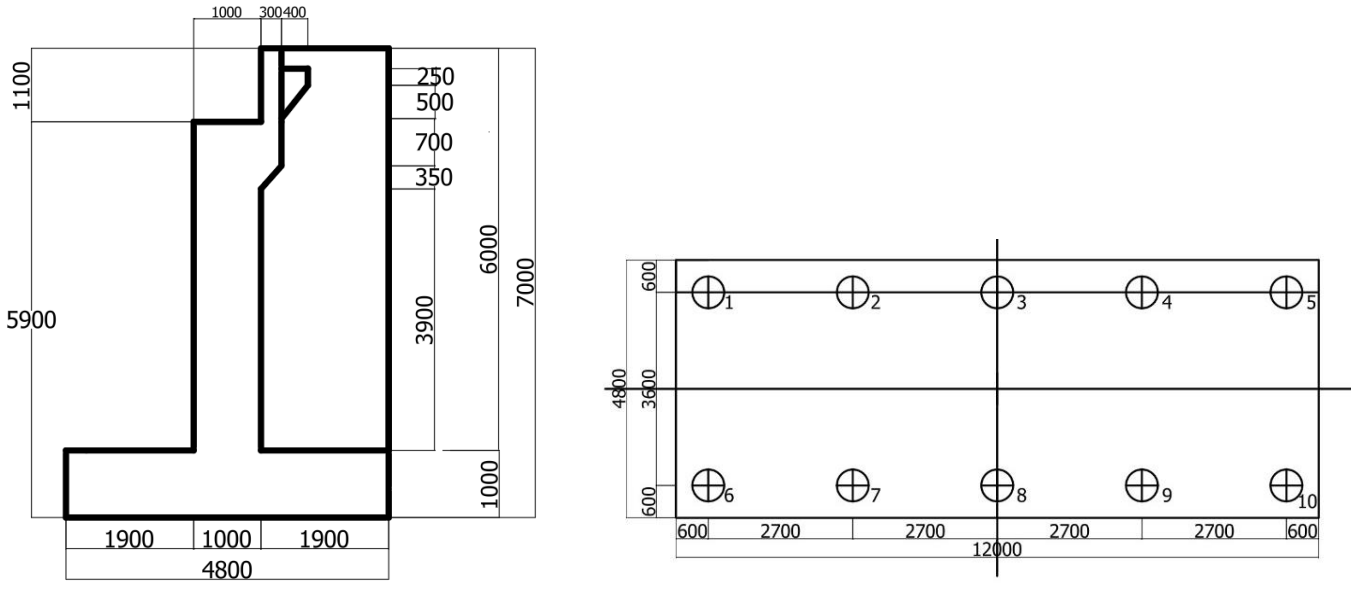
Load Type	Vertical	Horizontal	
	P (Ton)	Tx (Ton)	Ty (Ton)
Superstructure Self Weight (SW)	182.2		
Dead Load (DL)	49.56		
Live Load (LL)	10.29		
Braking Force (TB)		10.6	
Wind load on structure (Ews)			7.03
Wind Load on Vehicle (EWI)		1.8	
Total	242.05	12.4	7.03

4.2.1 Abutment A2

Based on the data above, a preliminary design was carried out for Abutment A2. At Abutment A2, checks were performed for bearing capacity stability of 3.67 and > 3, overturning stability of 3.17 and > 2 , and sliding stability of 1.03 and < 1.5; which not safe enough and needed depth foundation. Subsequently, the bored pile foundation design is initiated by selecting a pile diameter of 60 cm and a depth of 14 meters, after which both the single pile capacity and group capacity are calculated to evaluate their adequacy in supporting the applied structural loads. After calculating the pile capacity, which is 141.7 Tons, it was determined that a total of 8 piles are required, arranged in a 2 × 4 configuration with a spacing of 6D between piles. The configuration is shown in Figure 5 (a) and (b).

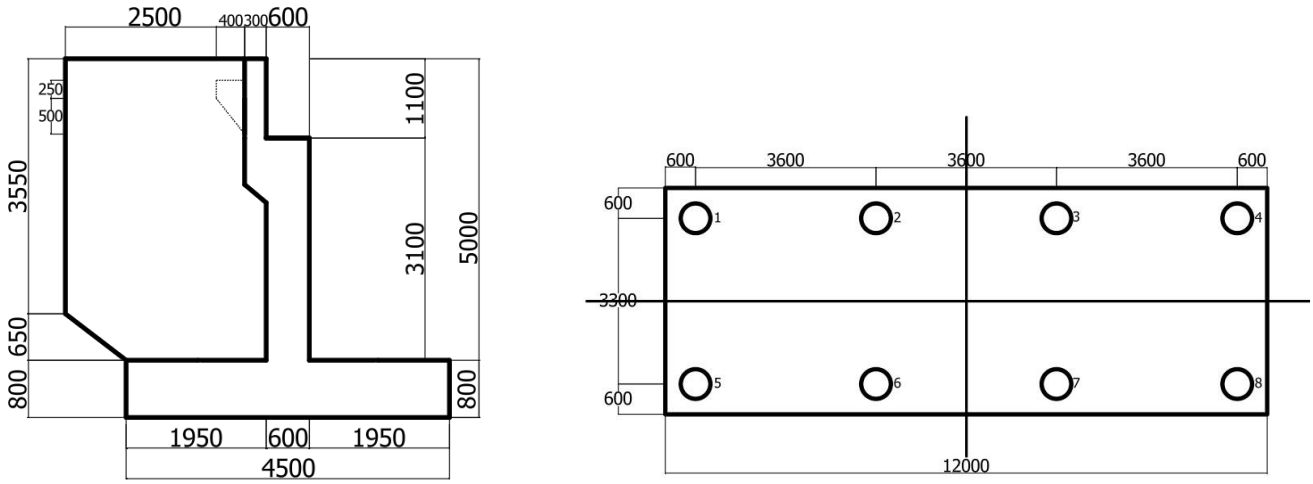
4.2.2 Pier P1 and Pier P2

The pier structures at locations P1 and P2 utilize identical dimensions with roller-type support. The foundation design for P1 and P2 is adapted according to the respective soil data. From the final load combination analysis, the maximum load is selected and applied in the foundation calculations. At Pier P1 dan P2, checks were performed for bearing capacity stability of 0.01 and < 3, overturning stability of 20.46 and > 2 , and sliding stability of 18.48 and > 1.5; which not safe enough and needed depth foundation. Subsequently, the bored pile foundation design is initiated by selecting a pile diameter of 100 cm and a depth of 14 meters, after which both the single pile capacity and group capacity are calculated to evaluate their adequacy in supporting the applied structural loads. After calculating the pile capacity, which is 367.3 Tons, it was determined that a total of 12 piles are required, arranged in a 3 × 4 configuration with a spacing of 3.6D between piles. The dimension of the pier and the pile configuration is shown in Figure 6 (a) and (b).



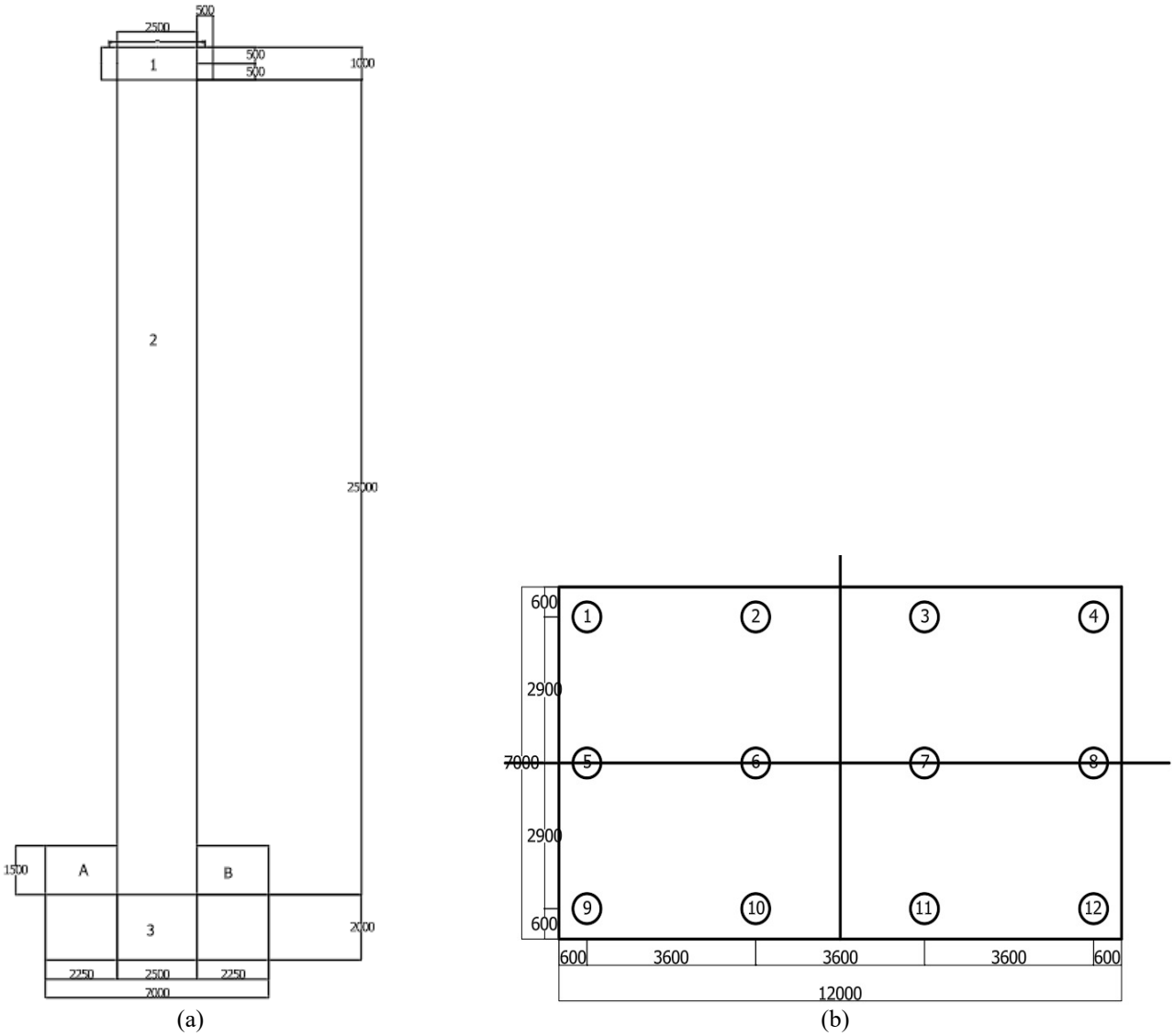
(a) (b)

Figure 4. (a). Dimensions of abutment A1, (b) Foundation pile layout with 10 piles



(a) (b)

Figure 5. (a). Dimensions of abutment A2, (b) Foundation pile layout with 8 piles



(a) (b)

Figure 6. (a) Dimensions of pier, (b). Foundation pile layout with 12 piles

4.2.3 Comparison of Diameter and Spacing Configurations of Bored Pile

The diameter of bored piles and the spacing between piles are critical parameters in the design of pile group foundations. Pile spacing plays a critical role in affecting the dynamic performance of coastal bridge foundations, particularly under multi-directional or combined loading scenarios [13]. These variables directly influence the group efficiency and the total bearing capacity of the foundation system. An optimal combination of pile diameter and spacing enhances the performance of the substructure by ensuring efficient load distribution to the underlying soil. The following Table 4 show the impact of varying pile diameters and spacing on pile group efficiency and bearing strength. Figure 7 show graph between group efficiency and group capacity. The comparison illustrates the relationship between pile group efficiency and group capacity across each substructure. With the same number of piles, bored piles with larger diameters result in higher group capacity.

Table 4. Comparison of diameter and spacing bored pile.

Foundation	Diameter (m)	Spacing (m)	Configuration	Group Efficiency	Q_{ag} (Ton)
A1	0.5	2.7	2 x 5	0.848	678.81
	0.5	2.0	2 x 5	0.797	637.84
	0.6	2.0	2 x 5	0.759	849.32
	0.6	2.7	2 x 5	0.819	916.75
A2	0.5	3.0	2 x 4	0.869	698.96
	0.5	2.5	2 x 4	0.843	678.31
	0.6	3.2	2 x 4	0.853	966.98
	0.6	2.4	2 x 4	0.805	913.15
P1 & P2	0.8	2.5	3 x 5	0.711	2556.34
	0.8	3.0	3 x 5	0.757	2721.21
	1.0	2.5	3 x 5	0.645	3552.52
	1.0	3.0	3 x 5	0.700	3854.81

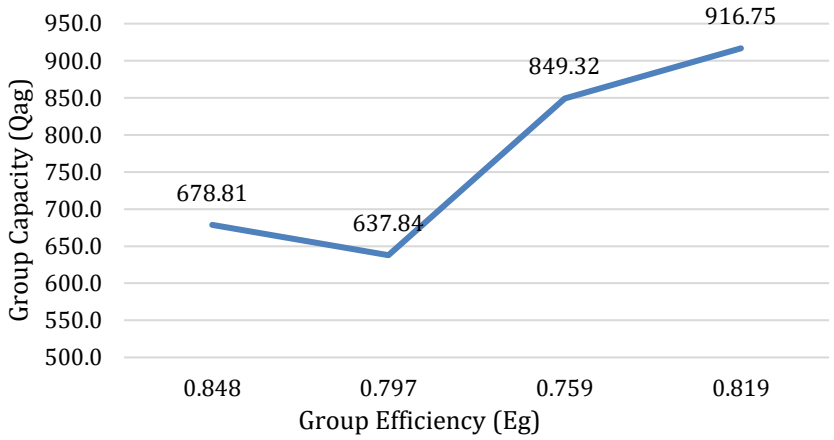


Figure 7. Comparison between group efficiency (Q_{ag}) and group capacity (E_g)

4.4 Subsurface Conditions and Natural Slope Stability

The subsurface conditions were characterized using cone penetration test (CPT) data and laboratory soil tests, including triaxial and direct shear tests. The investigation identified predominantly clayey volcanic soils, with cone resistance and interpreted soil stiffness generally increasing with depth.The interpreted subsurface profile is summarized as follows:

- **0–6 m:** Soft to medium-stiff silty volcanic clay, with CPT cone resistance (q_c) ranging from 8.09 to 16.94 kg/cm², cohesion (c) ranging from 0.24 to 0.41 kg/cm², friction angle (ϕ) ranging from 15.48° to 26.57°, and unit weight (γ) ranging from 1.74 to 1.86 T/m³.
- **6–10 m:** Medium-stiff clay, with (q_c) ranging from 15.71 to 56.71 kg/cm².
- **10–14 m:** Stiff clay, with (q_c) ranging from 63.20 to 108.70 kg/cm².
- **14–16 m:** A transitional layer, with (q_c) ranging from approximately 101 to 134 kg/cm².
- **16–17 m:** A very stiff to hard layer associated with very high CPT resistance or possible cone refusal, with reported q_c 150 kg/cm².

The initial pile design adopts a nominal length of 14 m. Based on the interpreted soil profile, the pile tips terminate approximately 2–3 m above the identified hard layer. Consequently, the contribution of the deeper, stronger stratum to pile-end resistance cannot be assumed without further investigation or a revised pile-length assessment. The axial resistance of the proposed piles must therefore be evaluated using the soil properties along the actual pile length and a CPT-based method appropriate for bored piles. Slope stability was evaluated using limit-equilibrium analysis in GeoStudio with the Morgenstern–Price method. The results for the natural slope conditions are presented in Table 5. Both locations exhibit calculated factors of safety below the adopted target of 1.50 for permanent slope conditions. The results indicate that the natural slopes have limited stability margins and may be vulnerable to additional loading or changes in groundwater conditions and stress distribution.These findings demonstrate the need to assess the bridge foundations and surrounding slopes as an interacting geotechnical system. Foundation selection should not be based solely on vertical bearing capacity; the effects of foundation loading, lateral forces, seismic actions, and potential slope deformation must also be considered.

Table 5. Natural slope stability results

Bridge location	Slope angle	Critical slip depth (m)	Factor of safety	Assessment relative to (FS=1.50)
Cihanjuang, right side	21.8°	14	1.212	Below target
Padaasih, left side	21.8°	14	1.286	Below target

4.5. Stability Assessment of Existing Shallow Foundations

The existing shallow foundations were assessed in terms of bearing resistance, sliding, overturning, and load eccentricity. The assessment considered the reported foundation geometry, vertical and horizontal loads, and overturning moments under the specified loading conditions.

4.5.1. Abutment A1

Abutment A1, located on the right side of the bridge, has a reported foundation height of 6 m and a footing width of 12 m. The applied vertical load is 508.74 Tons, and the overturning moment is 1,055.72 Ton.m. The load eccentricity is using Eq. (5), where (e) is the eccentricity of the resultant vertical load, (B) is the footing dimension in the direction of eccentricity, (M_y) is the corresponding overturning moment, and (V) is the total vertical load.

$$e = \left| \frac{B}{2} - \frac{M_y}{V} \right| \tag{5}$$

This result indicates that the assumption of full compressive contact beneath the footing requires further examination. The reported maximum bearing pressure of approximately 81.72 T/m² exceeds the reported ultimate bearing resistance of 81.13 T/m², giving a reported bearing-capacity factor of safety of approximately 0.99. This value is below the adopted criterion of 3.00. The reported sliding factor of safety is 2.36, exceeding the criterion of 1.50, while the overturning factor of safety is 2.90, exceeding the criterion of 2.00. Nevertheless, the reported bearing-capacity result indicates that the existing shallow foundation does not satisfy all the evaluated stability criteria. Because the resultant lies outside the middle third of the footing, the bearing-pressure distribution should be recalculated using a contact model consistent with the actual footing geometry and loading. The reported bearing pressure and associated factor of safety should be treated as provisional until this verification is completed.

4.5.2. Abutment A2

Abutment A2, located on the left side of the bridge, has a reported foundation height of 5 m and a footing width of 12 m. The applied vertical load is 361.04 Tons, and the horizontal load is 174.90 Tons. The reported bearing-capacity factor of safety is 3.67, exceeding the adopted criterion of 3.00. The overturning factor of safety is 3.17, which also exceeds the criterion of 2.00. However, the reported sliding factor of safety is only 1.03, below the required value of 1.50.Sliding therefore governs the reported stability assessment for A2. Under the evaluated loading condition, the existing shallow foundation cannot be considered adequate unless appropriate modification or strengthening measures are implemented.

4.5.3. Pier Foundations P1 and P2

The existing pier foundations support columns approximately 25 m high. The reported vertical load is 3,309.99 Tons, equivalent to approximately 32,471 kN. The footing width is reported as 7 m, and the calculated load eccentricity is 5.96 m. The reported eccentricity substantially exceeds this limit.This indicates that the resultant lies outside the footing base in the direction of eccentricity. Under these conditions, a conventional compression-only contact-pressure distribution cannot establish equilibrium without additional resisting mechanisms or a revised interpretation of the applied loads and geometry.The reported bearing-pressure calculation uses a dimension of 12 m, although the footing width is stated to be 7 m. Consequently, the reported maximum pressure of 5,183.98 T/m² and factor of safety of approximately 0.01 cannot be considered validated results.The pier foundation assessment

must therefore be revised using verified footing dimensions, load directions, applied moments, and an appropriate contact-pressure model. Although the reported eccentricity indicates a potentially serious design concern, a definitive bearing-capacity conclusion should be drawn only after these inconsistencies have been resolved.

4.6. Performance of the Proposed Bored Pile Foundations

Alternative bored pile configurations were evaluated for abutments A1 and A2 and pier foundations P1 and P2. The assessment considered pile diameter, length, arrangement, center-to-center spacing, group efficiency, axial capacity, maximum individual pile load, settlement, and lateral displacement. The resulting capacity-to-demand ratio was calculated by dividing the reported group capacity by the applied vertical load. This ratio is used here as a preliminary comparison measure and should not be interpreted as a complete demonstration of foundation safety.

4.6.1. Abutment A1

The proposed foundation for A1 consists of ten bored piles, each 0.60 m in diameter and 14 m long, arranged in two rows of five piles. The center-to-center spacing is 2.7 m, equivalent to 4.5 times the pile diameter. The reported allowable axial capacity of an individual pile is 111.93 Tons. Using a reported group-efficiency factor of 0.819, the group capacity is 916.75 Tons. The reported group capacity exceeds the applied vertical load of 508.74 Tons, giving a capacity-to-demand ratio of approximately 1.80. The maximum individual pile load is reported as 109.52 Tons, which is below the allowable single-pile capacity of 111.93 Tons. However, the difference is relatively small, so the maximum pile load should be verified against the governing load combinations and the adopted capacity criteria. The estimated settlement is 7.060 cm, compared with a stated allowable value of 16.80 cm. The estimated lateral displacement is 0.482 cm, below the stated limit of 2.54 cm. These results suggest that the proposed A1 pile group satisfies the reported axial-capacity and serviceability checks. Final acceptance remains conditional on verification of the pile-group efficiency, individual pile load distribution, settlement criterion, and lateral-response model.

4.6.2. Abutment A2

The proposed foundation for A2 consists of eight bored piles, each 0.60 m in diameter and 14 m long, arranged in two rows of four piles. The center-to-center spacing is 3.2 m, equivalent to approximately 5.33 times the pile diameter. The reported allowable axial capacity of an individual pile is 141.78 Tons. Using a reported group-efficiency factor of 0.852, the group capacity is 966.98 Tons. The reported group capacity is approximately 2.68 times the applied vertical load of 361.04 Tons. The maximum individual pile load is reported as 86.20 Tons, below the allowable single-pile capacity of 141.78 Tons. The estimated settlement is 7.341 cm, compared with a stated allowable value of 16.80 cm, while the estimated lateral displacement is 0.30 cm, below the stated limit of 2.54 cm. The reported results indicate adequate axial capacity and serviceability performance for the proposed A2 pile group. However, the conclusion remains conditional on verification of the adopted load combinations, group interaction, lateral resistance, and displacement criteria.

4.6.3. Pier Foundations P1 and P2

The proposed pier foundations consist of fifteen bored piles, each 1.00 m in diameter and 14 m long, arranged in three rows of five piles. The center-to-center spacing is 2.5 m, equivalent to 2.5 times the pile diameter. The reported allowable axial capacity of an individual pile is 367.35 Tons. Using the stated group-efficiency factor of 0.625 gives 3443.91 Tons. The original manuscript reports a group capacity of 3,441.29 Tons. This discrepancy should be reconciled using the unrounded individual pile capacity and group-efficiency factor from the original calculation. Using the originally reported group capacity of 3,441.29 Tons, the capacity-to-demand ratio relative to the applied vertical load of 3,309.99 Tons is approximately 1.04. The maximum individual pile load is reported as 228.38 Tons, below the allowable single-pile capacity of 367.35 Tons. The estimated settlement is 11.419 cm, compared with a stated allowable value of 16.83 cm. The estimated lateral displacement is 0.010 cm, below the stated limit of 2.54 cm. Although the reported group capacity exceeds the applied vertical load, the capacity-to-demand margin is relatively small. The pier foundation therefore requires particular attention to the governing load combinations, group capacity, individual pile loads, settlement, lateral response, and possible slope deformation.

4.6.4. Comparison of Existing Shallow Foundations and Proposed Pile Foundations

Table 6 summarizes the reported results for the existing shallow foundations and proposed bored pile groups. For the pile foundations, the capacity-to-demand ratio is defined as the reported allowable group capacity divided by the applied vertical load. The shallow-foundation bearing factors of safety are reported separately because they represent a different assessment measure.

Table 6. Comparison of existing shallow foundations and proposed bored pile foundations

Foundation location	Foundation type	Configuration	Reported bearing result	Sliding FS	Overturning FS	Assessment
A1	Shallow footing	Existing	0.99; target 3.00	2.36	2.9	Reported bearing resistance inadequate; pressure calculation requires verification
A1	Bored pile group	10 piles, (D=0.60) m, (L=14) m	Capacity/demand = 1.80	Not independently verified	Not independently verified	Preliminary axial and serviceability results are favorable
A2	Shallow footing	Existing	3.67; target 3.00	1.03	3.17	Sliding resistance inadequate
A2	Bored pile group	8 piles, (D=0.60) m, (L=14) m	Capacity/demand = 2.68	Not independently verified	Not independently verified	Preliminary axial and serviceability results are favorable
P1/P2	Shallow footing	Existing	Approximately 0.01, provisional	Requires verification	Requires verification	Calculation must be revised because of geometric inconsistency and excessive eccentricity
P1/P2	Bored pile group	15 piles, (D=1.00) m, (L=14) m	Capacity/demand ≈ 1.04	Not independently verified	Not independently verified	Relatively limited axial capacity margin

The preliminary results suggest that bored pile foundations may provide a more suitable load-transfer mechanism than the existing shallow footings, particularly where near-surface bearing resistance or load eccentricity is problematic. Nevertheless, their adequacy must be demonstrated through consistent axial, lateral, settlement, and global slope-stability assessments.

4.7. Load Transfer and the Relationship between Piles and the Critical Slip Surface

The CPT profile indicates a hard layer at approximately 16–17 m depth, where the reported cone resistance reaches or exceeds 150 kg/cm². The proposed piles terminate at 14 m, approximately 2–3 m above this layer. Based on the interpreted soil profile, the proposed piles may be classified as predominantly friction-resisting piles, although their actual behavior depends on the distribution of shaft resistance and the contribution of end bearing at the pile tips. The relative contributions of these mechanisms depend on the soil layers along the pile length, the CPT-based correlations adopted, and the effects of bored-pile construction. The reported shaft-resistance contributions are 308.57 kN for A1, 706.38 kN for A2, and 379.21 kN for the pier foundations. These values should be checked against the sum of the shaft-resistance contributions over the full embedded length of the relevant piles. It should also be clarified whether the values represent the resistance of an individual pile or the total resistance of the pile group. The slope-stability analysis identifies a critical slip depth of approximately 14 m, which coincides with the proposed pile length. Consequently, the current geometry does not demonstrate that the pile tips extend below the critical slip surface. Although the piles may intersect the potential sliding zone, their contribution to global slope stability cannot be established from axial-capacity calculations alone.

A separate soil–pile interaction analysis is required to assess the effects of pile location, pile stiffness, bending resistance, soil movement, and the potential development of lateral loads along the piles. If slope stabilization is an explicit design objective, the analysis should demonstrate that the pile system provides a sufficient increase in the global factor of safety. Until such an assessment is completed, the proposed bored pile system should be described as a foundation-strengthening alternative rather than a verified slope-stabilization measure.

5. COMPARISON OF PILE DIAMETER AND SPACING CONFIGURATIONS

A parametric assessment was conducted to investigate the effects of pile diameter and center-to-center spacing on the calculated axial capacity of the pile groups supporting abutments A1 and A2 and pier foundations P1 and P2. The assessment compares alternative configurations using the reported individual pile capacities and group-efficiency factors. The objective is to identify configurations that improve axial capacity while remaining practical in terms of pile arrangement, pile-cap dimensions, and construction requirements. The results should be interpreted as a preliminary capacity-based comparison. The final design selection must also account for individual pile loads, settlement, lateral resistance, pile-group interaction, seismic loading, and the relationship between the foundation system and the surrounding slope.

5.1. Abutment A1

Table 7 presents the reported parametric results for abutment A1. The baseline configuration consists of ten piles with a diameter of 0.50 m and a center-to-center spacing of 2.0 m, arranged in two rows of five piles. The reported results indicate that increasing the pile diameter from 0.50 to 0.60 m produces a greater increase in group capacity than increasing the spacing alone. The configuration

with 0.60 m diameter piles and 2.7 m spacing has the highest reported group capacity among the evaluated alternatives. However, the group-efficiency factors must be verified using a consistent formulation and the correct pile arrangement. The selected configuration should also be assessed against individual pile capacity, maximum pile load, pile-cap geometry, settlement, lateral displacement, and construction tolerances.

Table 7. Parametric assessment of pile diameter and spacing for abutment A1

Pile diameter, D (m)	Spacing, (S) (m)	Arrangement	Group efficiency	Reported group capacity, (Q_{ag}) (Tons)	Increase from baseline
0.5	2.0	2x5	0.797	637.84	Baseline
0.5	2.7	2x5	0.848	678.81	6.40%
0.6	2.0	2x5	0.759	849.32	33.20%
0.6	2.7	2x5	0.819	916.75	43.70%

5.2. Abutment A2

The parametric assessment for A2 considers eight-pile groups arranged in two rows of four piles. The baseline configuration uses piles with a diameter of 0.50 m and a center-to-center spacing of 2.5 m. The reported results suggest that increasing pile diameter is the principal contributor to the increase in calculated axial group capacity. Increasing the spacing is also associated with higher reported group efficiency for the configurations considered. Among the evaluated alternatives, the configuration with 0.60 m diameter piles and 3.2 m spacing provides the highest reported group capacity. Nevertheless, the final configuration should be selected only after verifying group efficiency, individual pile loads, pile-cap dimensions, settlement, lateral response, and the applicable design criteria.

5.3. Pier Foundations P1 and P2

The parametric assessment for the pier foundations considers fifteen-pile groups arranged in three rows of five piles. The baseline configuration consists of 0.80 m diameter piles with a center-to-center spacing of 2.5 m. The reported results indicate that the configuration with 1.00 m diameter piles and 3.0 m spacing provides the highest calculated axial group capacity among the evaluated alternatives. Relative to the baseline configuration, the reported capacity increases by approximately 50.8%. However, the pier foundations carry substantially larger loads than the abutment foundations. The previously reported capacity-to-demand ratio for the selected 1.00 m diameter configuration is also relatively close to unity. Therefore, the configuration with the highest calculated group capacity should not automatically be regarded as the optimum design. The final selection must account for the governing load combinations, verified individual pile capacities, maximum pile loads, settlement, lateral resistance, pile-cap dimensions, constructability, and the required design margin.

Table 8. Parametric assessment of pile diameter and spacing for abutment A2

Pile diameter, D (m)	Spacing, (S) (m)	Arrangement	Group efficiency	Reported group capacity, (Q_{ag}) (Tons)	Increase from baseline
0.5	2.0	2x5	0.797	637.84	Baseline
0.5	2.7	2x5	0.848	678.81	6.40%
0.6	2.0	2x5	0.759	849.32	33.20%
0.6	2.7	2x5	0.819	916.75	43.70%

Table 9. Parametric assessment of pile diameter and spacing for pier foundations

Pile diameter, D (m)	Spacing, S (m)	Arrangement	Group efficiency	Reported group capacity, Q_{ag} (Tons)	Increase from baseline
0.8	2.5	3x5	0.711	2,556.34	Baseline
0.8	3.0	3x5	0.757	2,721.21	6.40%
1	2.5	3x5	0.645	3,552.52	39.00%
1	3.0	3x5	0.7	3,854.81	50.80%

5.4. Overall Interpretation of the Parametric Results

Across the three foundation locations, the reported parametric results indicate that increasing pile diameter generally produces a larger increase in calculated axial group capacity than increasing pile spacing alone. Increasing the spacing is associated with improved group efficiency in the reported configurations, although the magnitude of this effect depends on the adopted interaction model and the pile arrangement. For abutment A1, the configuration with 0.60 m diameter piles and 2.7 m spacing provides the highest reported group capacity. For abutment A2, the corresponding configuration uses 0.60 m diameter piles with 3.2 m spacing. For the pier foundations, the configuration with 1.00 m diameter piles and 3.0 m spacing produces the highest reported group capacity in the parametric assessment. These findings identify pile diameter and spacing as important design variables for foundation optimization. Nevertheless, the results represent a preliminary comparison based on calculated axial capacity rather than a complete optimization outcome. Before the preferred configurations can be recommended, the group-efficiency calculations, individual pile capacities, load distribution, serviceability criteria, and foundation–slope interaction must be verified.

The proposed bored pile systems may provide a more appropriate load-transfer mechanism than the existing shallow foundations under the investigated conditions. Their overall suitability for the Cihanjuang Bridge, however, must be established through an integrated assessment of axial and lateral resistance, settlement, seismic loading, construction feasibility, and global slope stability.

8. CONCLUSION

The Cihanjuang Bridge is planned to be located in Cihanjuang–Padaasih, West Bandung Regency. Based on the identified issues, the design of the substructure of the Cihanjuang Bridge includes the foundation, abutments, piers, and their supporting elements.Based on the results of the substructure analysis and design, the right abutment is planned to be constructed using the cast-in-situ method with a height of 7 meters. However, the stability analysis of the abutment in terms of bearing capacity and sliding resistance does not meet the required safety factors. Therefore, a pile foundation is needed to enhance the strength of the substructure. A bored pile foundation consisting of 10 cast-in-place piles with a length of 14 meters is designed, in accordance with the analysis of single pile capacity, group pile capacity, settlement, and displacement.On the left side of the bridge, the left abutment is also designed using the cast-in-situ construction method with a height of 5 meters and supported by a deep foundation (bored pile) to satisfy stability requirements. For the pier foundation design on both the left and right sides, a 25-meter-high column is supported by 12 bored piles with a diameter of 100 cm.

The comparison of diameter and the spacing between piles show, the bigger diameter could increase group capacity. Pile spacing has a significant impact on how efficient the pile group performs. Closer spacing tends to reduce efficiency due to overlapping stress zones, while wider spacing improves load distribution and overall group performance. The 40% slope gradient (21.8º) affects the bearing performance and stability of the bridge foundations. The reported slope correction factor ($g_c = 0.852$) corresponds to a 14.8% reduction in the calculated ultimate soil bearing capacity. This reduction contributes to the inadequate performance of the existing shallow foundations, with Abutment A1 exhibiting a reported bearing-capacity factor of safety of 0.99 and Abutment A2 exhibiting a sliding factor of safety of 1.03. These results indicate that the existing shallow foundations do not satisfy the adopted safety criteria for bearing capacity and sliding, respectively.

The central piers P1 and P2 carry a reported vertical load of 3,309.99 Tons, while the preliminary assessment of their existing shallow foundations indicates a bearing-capacity factor of safety of approximately 0.01. However, this value requires verification because of inconsistencies in the reported footing dimensions and eccentricity. The proposed 14 m bored pile foundations provide higher axial load capacity, with reported capacity-to-demand ratios ranging from 1.04 to 2.68, lateral displacements below 0.48 cm, and total group settlements below 11.42 cm, compared with the stated allowable settlement of 16.8 cm. These results indicate improved foundation performance, although final verification of pile-group capacity, settlement, lateral response, and overall slope stability remains necessary.

At a length of 14 m, the proposed bored piles are expected to derive a substantial portion of their axial resistance from shaft friction in the stiff clay layers, although the relative contributions of shaft resistance and end bearing must be confirmed through the detailed capacity calculations. Increasing pile diameter produces a reported group-capacity increase of up to 50.8%, while increasing the center-to-center spacing can improve the Converse–Labarre group efficiency factor to approximately for the evaluated configurations. The results support the use of diameter and spacing as important design variables for optimizing the Cihanjuang Bridge substructure. Nevertheless, the final configuration should be selected based on verified axial capacity, settlement, lateral response, structural requirements, construction feasibility, and global slope stability rather than capacity alone.

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