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Structural and Geotechnical Stability of a 120 m Suspension Footbridge in South Lampung, Indonesia

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ABSTRACT: Suspension footbridges provide essential pedestrian connectivity across rivers and other natural barriers. However, long-span configurations require integrated assessments of structural reactions, foundation stability, seismic site conditions, and approach-road performance. This study evaluates the structural and geotechnical stability of a proposed 120 m rigid, symmetrical suspension footbridge crossing the Way Sekampung River in South Lampung, Indonesia. The assessment integrates site surveys, standard penetration tests (SPTs) at four investigation points, cone penetration tests (CPTs) at four points, structural reaction analysis, foundation stability calculations, seismic site classification, and approach-road assessment. The proposed configuration comprises a 120 m main span, a 23.05 m backstay, a 35° cable inclination, and 15.75 m-high pylons. The corrected average SPT value was reported as 60.2, and the site was classified as Site Class SC in the project assessment. The reported seismic parameters include a peak ground acceleration (PGA) of 0.252 g, short-period spectral acceleration (SS) of 0.509 g, one-second spectral acceleration (S1) of 0.230 g, and design spectral acceleration (SD1) of 0.362 g. These values require verification against the original seismic design documentation. The calculated overturning safety factors ranged from 1.63 to 1.73 for the anchor blocks and from 5.62 to 5.86 for the pylons. The vertical and horizontal stability factors satisfied the adopted criteria. Calculated settlements were below 1.0 cm, although those for anchor block AB02 and pylon P2 approached the adopted limit. The maximum approach-road fill height was 2.29 m, compared with a calculated critical height of 5.74 m, while the reported slope safety factor was 9.70. Overall, the calculations indicate satisfactory structural and geotechnical performance under the investigated design conditions. The findings demonstrate the importance of integrating structural, foundation, seismic, and approach-road assessments while emphasizing the need to verify critical design inputs and monitor foundations with settlements approaching allowable limits.

KEYWORDS: Foundation stability; geotechnical assessment; pedestrian bridge; seismic site classification; suspension footbridge; structural stability

1. Introduction

Pedestrian suspension bridges provide essential access across rivers, valleys, and other physical barriers where conventional bridge systems may require substantial structural dimensions, intermediate supports, or extensive modifications to the surrounding terrain. Their structural efficiency is achieved through load transfer among cables, towers, and anchorages, making them particularly suitable for locations requiring long spans with limited intermediate supports. Nevertheless, the overall performance of a suspension footbridge depends on the interaction among the superstructure, supporting foundations, surrounding ground conditions, and access infrastructure.

Previous studies have investigated aerodynamic stability, nonlinear structural response, cable behaviour, tower flexibility, asymmetric loading, seismic performance, and reliability in suspension bridge systems [1–10]. Aerodynamic investigations have demonstrated that wind-induced responses depend on deck configuration, torsional stiffness, damping, and flutter characteristics [1–5]. Geometrically nonlinear analyses have highlighted the influence of tower–girder interaction and cable-force redistribution on overall structural behaviour [6, 9, 10]. Other studies have examined eccentric loading, asymmetric live loading, and ultimate structural capacity [7, 8]. Collectively, these investigations provide important insights into the structural performance of suspension bridges, particularly those with long spans and complex load-transfer mechanisms.

Dynamic interaction and anchorage behaviour are also important considerations in suspension bridge design. Studies of bridge systems subjected to moving loads and crosswinds have highlighted the potential coupling between external loading and structural response [11]. Anchorage investigations have emphasized the importance of safely transferring substantial cable forces to the supporting soil or rock mass [12]. Alternative stiffening systems, fatigue behaviour, and vibration performance have also received considerable attention [13, 14]. Furthermore, seismic investigations have examined the fragility and dynamic response of cable-supported bridges under earthquake excitation [15–17]. Recent developments have incorporated topology optimization, surrogate modelling, fire-related stability assessment, and reliability-based design approaches [16, 18–20].

Despite these advances, practical pedestrian suspension bridge projects often require assessments that differ from those commonly reported for long-span highway and railway bridges. Previous investigations focusing on aerodynamic flutter, nonlinear dynamics, or structural optimization have generally emphasized specific mechanisms governing superstructure performance. However, these approaches do not necessarily encompass the combined evaluation of foundation bearing capacity, overturning resistance, settlement, lateral deformation, seismic site classification, and approach-road stability under actual site conditions. For pedestrian suspension bridges, the functionality and safety of the complete crossing also depend on the stability and serviceability of the approach embankments and their interaction with the supporting ground.

Accordingly, the research gap addressed in this study concerns the limited integration of structural reactions, foundation stability, seismic site characterization, and approach-road performance within a comprehensive, site-specific assessment of relatively long pedestrian suspension bridges. The novelty of this study lies in integrating these complementary engineering assessments for a proposed 120 m rigid, symmetrical suspension footbridge crossing the Way Sekampung River. Rather than developing a new suspension bridge theory,

this study presents an integrated engineering evaluation that links bridge geometry and cable reactions with foundation loading, deformation limits, seismic site conditions, and approach-embankment requirements. This approach provides a practical framework for assessing the structural and geotechnical performance of pedestrian suspension bridges under site-specific conditions.

The proposed bridge crosses the Way Sekampung River, locally associated with the Way Jelujur crossing, between Relung Mulya and Batanghari Ogan villages in Natar Subdistrict, South Lampung Regency, Lampung Province, Indonesia. The existing emergency suspension footbridge consists of a timber deck approximately 1.5 m wide, while the surrounding access roads range from approximately 1.5 to 4.0 m in width. These conditions highlight the need to evaluate a wider, more permanent pedestrian crossing that can provide improved accessibility while maintaining structural stability and geotechnical safety.

This study aims to (1) evaluate the geometry and support reactions of the proposed 120 m suspension footbridge; (2) assess the stability, bearing capacity, settlement, and lateral deformation of the anchor-block and pylon foundations; (3) document the seismic site classification and evaluate its implications for structural and foundation design; and (4) assess the stability of the approach-road embankments. The findings are intended to determine whether the reported design calculations satisfy the adopted structural and geotechnical criteria and to identify critical considerations for construction, foundation monitoring, and long-term bridge performance.

2. Materials and Methods

2.1. Study Location and Bridge Configuration.

The study site is located at the Way Sekampung River crossing between Relung Mulya Village and Batanghari Ogan Village, Natar Subdistrict, South Lampung Regency, Lampung Province, Indonesia. The original coordinates recorded in the project documentation are 5°18′14″ S and 105°24′81″ E. Since the longitude contains 81 seconds, exceeding the conventional range of 0–59 seconds, it can be mathematically normalized to 105°25′21″ E. Accordingly, the normalized coordinates are 5°18′14″ S and 105°25′21″ E. However, this conversion represents a mathematical correction rather than independent verification of the surveyed location. The coordinates should therefore be confirmed against the original survey records before construction or publication.

The river is approximately 38.7 m wide under normal flow conditions, increasing to approximately 111 m during flooding. The existing emergency footbridge provides pedestrian access across the river but has a deck width of approximately 1.5 m. The access roads on both sides are relatively narrow, with reported widths ranging from 1.5 to 4.0 m. The proposed structure is a rigid, symmetrical suspension footbridge with a 120 m main span, a 23.05 m backstay, a cable inclination of 35°, and pylons measuring 15.75 m in height. These dimensions were obtained from the project design documentation and adopted as the basis for the structural reaction and foundation stability assessments.

The proposed 120 m span was compared with alternative main spans of 40, 60, 80, and 100 m to evaluate differences in bridge geometry and structural reaction forces. This comparison provides insight into the influence of increasing span length on the structural configuration and associated foundation demands. The 120 m span was adopted as the project

configuration for detailed design evaluation rather than selected through a documented multi-criteria optimization process. Therefore, the comparison is intended to illustrate the structural implications of different span lengths without implying that the 120 m configuration represents the most economical or structurally optimal alternative under all conditions.

2.2. Field Survey and Geotechnical Investigation.

The assessment incorporated field surveys, topographic observations, site-condition documentation, geotechnical investigation reports, structural design information, and detailed engineering design (DED) documentation. The geotechnical investigation included four boreholes with standard penetration tests (SPTs) and four cone penetration test (CPT) locations. The principal geotechnical reference was the soil investigation report prepared by the Soil Mechanics Laboratory of Universitas Lampung.

The investigation program was evaluated in relation to the principal foundation components, including the anchor blocks, pylons, and wind-guy foundation. Borehole and SPT data were used to characterize soil stratification and penetration resistance, while CPT data provided more continuous information on subsurface penetration resistance with depth. The combined investigation results supported the identification of potentially weak soil layers, assessment of subsurface variability, and estimation of foundation bearing capacity and settlement.

The available project information does not provide the coordinates, individual investigation depths, complete SPT profiles, CPT plots, or specific allocation of investigation points to individual foundation locations. Consequently, the spatial distribution of the eight investigation points could not be independently reconstructed from the available documentation. The geotechnical investigation is therefore described at the project level, with the reported average corrected SPT value of 60.2 retained as documented. This value represents the reported project-level result and should not be interpreted as evidence of uniform subsurface conditions across all foundation locations.

The geotechnical assessment considered soil stratification, penetration resistance, soil unit weight, shear strength parameters, bearing capacity, settlement, and foundation deformation. These parameters were used to evaluate the stability and performance of the anchor-block, pylon, and wind-guy foundations. The DED documentation provided the principal structural dimensions, foundation arrangements, loading information, and design configurations required for the integrated structural and geotechnical assessments.

2.3. Design Standards and Loading Criteria.

The structural and geotechnical assessments were conducted with reference to relevant Indonesian bridge design standards and technical provisions. General design requirements for pedestrian suspension bridges were based on Circular Letter No. 02/SE/M/2010 [21]. Bridge loading requirements were referenced to SNI 1725:2016 [22], while reinforced-concrete bridge components were assessed in accordance with RSNI T-12-2004 [23]. Seismic design considerations were based on SNI 2833:2016 [24], and geotechnical assessments referred to SNI 8460:2017 [25]. Additional provisions concerning shallow foundations, caisson foundations, and pile foundations were considered in the foundation assessments [26–28]. The approach-road and embankment stability evaluations referred to Pd T-11-2003 [29]. The project-specific structural arrangement, foundation configurations, and design parameters were

obtained from the bridge construction documentation and detailed engineering design (DED) reports [30, 31].

The adopted minimum safety factors were 1.50 for overturning stability, 1.50 for horizontal stability, and 2.00 for vertical stability. The allowable deformation limits were specified as 1.00 cm for settlement and 2.54 cm for lateral displacement. These values represent the design criteria adopted in the project calculations. Their applicability to individual foundation types, governing load combinations, serviceability requirements, and relevant design standards should be confirmed against the original design documentation.

The structural and foundation assessments considered the reaction forces associated with the governing design conditions. In principle, the applicable load combinations should account for dead loads, pedestrian live loads, wind loads, and seismic effects, as required by the relevant standards. However, the available project documentation does not provide a complete schedule of load combinations or a detailed breakdown of individual load components. Consequently, the reaction forces and safety factors reported in the original calculations were retained for the present assessment without reconstructing undocumented loading conditions. This approach allows the reported structural and geotechnical performance to be evaluated against the adopted design criteria while maintaining consistency with the available project information.

2.4. Geometric and Reaction-Force Analysis.

The geometric assessment considered five alternative main-span lengths of 40, 60, 80, 100, and 120 m to examine the influence of span length on bridge configuration and support reactions. The evaluated parameters included main-span length, cable inclination, backstay length, pylon height, and the corresponding horizontal, vertical, and resultant support reactions. A constant cable inclination of 35° was adopted for all configurations, while the backstay length and pylon height varied according to the respective span arrangements.

The resultant support reaction was calculated from the horizontal and vertical reaction components using Equation (1):

$$R = \sqrt{H^2 + V^2} \tag{1}$$

where R is the resultant reaction kN, H is the horizontal reaction kN, and V is the vertical reaction kN.

For the selected 120 m span configuration, the reported horizontal and vertical reactions were 723.20 kN and 521.85 kN, respectively. Substituting these values into Equation (1) gives a resultant reaction of approximately 891.8 kN. These reaction components were subsequently used as inputs for the foundation stability and bearing-capacity assessments, providing a connection between the bridge's structural configuration and the corresponding foundation demands.

Table 1 summarizes the geometric parameters and reported support reactions for the five span alternatives. The results indicate that increasing the main-span length is accompanied by increases in pylon height, backstay length, and support reaction forces. The resultant reaction increases from approximately 180.6 kN for the 40 m configuration to 891.8 kN for the 120 m configuration. This trend demonstrates the increasing foundation demands associated with longer spans under the investigated design arrangements. The 120 m configuration was adopted for the detailed structural and geotechnical assessments because it represents the proposed

project design. The comparison among alternative spans is therefore intended to illustrate the relationship between bridge geometry and support reactions rather than establish an optimized span length. The reported reaction forces provide the basis for evaluating the overturning resistance, horizontal and vertical stability, bearing capacity, and deformation performance of the proposed foundation systems.

Table 1. Bridge geometry and reported anchor-block reactions for different main-span lengths.

Main span (m)	Cable angle (°)	Backstay (m)	Pylon height (m)	Horizontal reaction, H (kN)	Vertical reaction, V (kN)	Resultant reaction, R (kN)
40	35	9.41	6.2	149.99	100.6	180.6
60	35	11.87	8	385.94	250.68	460.2
80	35	16.26	11	546.47	363.56	656.4
100	35	19.82	13.5	683.27	456.73	821.9
120	35	23.05	15.75	723.2	521.85	891.8

Note: Values are reproduced from the project data provided for this study. The original calculation sheets should be consulted to confirm the precise reaction definitions and load cases.

2.5. Foundation Stability Assessment.

The anchor-block, pylon, and wind-guy foundations were evaluated for applicable stability and serviceability conditions. The principal checks comprised overturning, vertical stability, horizontal stability, bearing capacity, settlement, and lateral deformation.The factor of safety against overturning (FS_{OT}) was determined using Equation (2):

$$FS_{OT} = \frac{\sum M_{res}}{\sum M_{over}} \tag{2}$$

where $\sum M_{res}$ is the sum of resisting moments and $\sum M_{over}$ is the sum of overturning moments about the relevant reference point.Similarly, the vertical stability factor FS_v was evaluated using Equation (3):

$$FS_V = \frac{Q_{ult}}{Q_{aplied}} \tag{3}$$

where Q_{ult} is the applicable vertical resistance or capacity and Q_{aplied} is the corresponding vertical demand. The precise definition of resistance must follow the foundation model used in the original calculation.Horizontal stability was assessed based on Equation (4):

$$FS_H = \frac{R_H}{D_H} \tag{4}$$

where R_H is the available horizontal resistance and D_H is the applied horizontal demand. The resistance may include applicable contributions from base friction, passive resistance, or other mechanisms permitted by the adopted design method.For shallow-foundation bearing capacity, the general theoretical framework is given by Equation (5):

$$q_{ult} = c'N_c + q'N_q + \frac{1}{2}\gamma B N_\gamma \tag{5}$$

where c' is the effective cohesion, q' is the effective overburden pressure at foundation level, γ is the soil unit weight, B is the applicable foundation width, and N_c, N_q, and N_γ are bearing-capacity factors. Shape, depth, inclination, groundwater, and load-eccentricity corrections must be applied where required by the selected method. Equation (5) is presented as the general

theoretical framework and does not replace project-specific caisson calculations.The corresponding factor of safety for bearing capacity FS_B is defined in Equation (6):

$$FS_B = \frac{Q_{ult}}{Q_{aplied}} \tag{6}$$

where the numerator and denominator represent compatible resistance and demand quantities. Furthermore, the contact stress criterion is expressed as Equation (7):

$$q_{contact} < q_{allow} \tag{7}$$

where $q_{contact}$ is the calculated foundation contact stress and q_{allow} is the adopted allowable bearing pressure.

Settlement was evaluated against the project-adopted serviceability limit. The reported settlement values are retained because detailed soil compressibility parameters and layer-by-layer settlement calculations are not available in the supplied material. A complete independent settlement calculation would require relevant compressibility parameters, foundation dimensions, applied stress distribution, soil-layer thicknesses, and groundwater conditions.The anchor-block system comprises an approximately 8.0 m x 8.0 m x 3.0 m pile-cap arrangement supported by a caisson approximately 4.0 m in diameter and 3.0 m deep. Bearing capacity was assessed using the Meyerhof approach [32]. The reported caisson bearing capacity was approximately 5,191.5 tonnes, compared with a factored axial demand of approximately 541.1 tonnes.The pylon foundation comprises a 6.0 m x 7.5 m x 1.5 m pile cap supported by a double-ring caisson. The outer and inner caisson diameters are approximately 4.0 m and 3.2 m, respectively, with a foundation depth of approximately 4.0 m. The reported axial-capacity estimates were approximately 73,214.2 kN based on material capacity, 29,487.9 kN based on laboratory soil parameters, and 163,362.8 kN based on the reported SPT-based assessment. The laboratory-based estimate was adopted as the governing value in the project assessment because it was the lowest of the three reported values. These capacities must be interpreted according to their original calculation definitions and should not be combined unless they represent compatible resistance mechanisms and limit states [32, 33].

2.6. Seismic Site Classification.

The seismic site assessment was based on the reported geotechnical investigation and the provisions referenced in SNI 2833:2016. The project documentation classified the site as Site Class SC and reported a corrected average SPT value of 60.2 [34]. The seismic parameters reported in the project data were PGA = 0.252 g, S_S= 0.509 g, (S_1=0.230 g), and (SD_1=0.362 g). These values are retained as reported rather than recalculated because the original seismic hazard output, site coefficients, and response-spectrum calculations were not supplied.

The relationship between the site class and the seismic design spectrum is important because local ground conditions influence the spectral demand imposed on the bridge. Site classification, design spectral parameters, structural period, damping, and the applicable response-modification provisions collectively influence the calculated seismic forces. These forces may alter tower reactions, cable forces, anchorage demand, and foundation deformation.

The reported value of (SD_1) is greater than the reported value of (S_1). Because the parameters represent different quantities in the seismic design process, this relationship cannot be judged solely by comparing their numerical magnitudes; however, the values must be

checked against the original hazard and response-spectrum calculations before final submission. A static stability assessment alone does not establish seismic adequacy unless the earthquake-induced actions have been included in the relevant load combinations and the applicable seismic design checks have been completed.

2.7. Approach-Road and Embankment Assessment.

The approach-road assessment considered the geometry of the access roads, embankment height, soil strength parameters, allowable bearing pressure, and reported slope stability [35]. The approach grades were approximately 17% on the Relung Mulya side and 14% on the Batanghari Ogan side. The reported fill material had a unit weight of approximately 1.72 t/m³ and a friction angle of 35°. The native soil at Batanghari Ogan was characterized by a unit weight of approximately 1.77 t/m³, a friction angle of 32.2°, and cohesion of approximately 0.37 t/m².

The reported ultimate bearing capacity was approximately 29.62 t/m², with an allowable bearing pressure of approximately 9.88 t/m². The critical fill height was estimated at 5.74 m, while the maximum proposed fill height was approximately 2.29 m. The reported finite-element slope analysis produced a safety factor of approximately 9.70. The approach embankment was assessed against the project-reported critical height and slope stability results. The margin between the proposed fill height and the reported critical height provides an initial indication of geometric adequacy, but it does not independently establish stability under groundwater changes, heavy rainfall, erosion, traffic loading, or earthquake loading. The relatively high reported slope safety factor should therefore be interpreted in the context of the model geometry, boundary conditions, soil parameters, groundwater assumptions, and the failure mechanism considered.

3. Results and Discussion

3.1. Bridge Geometry and Cable Configuration.

Table 1 shows that the backstay length, pylon height, and support reactions increase with the main span. For the 120 m configuration, the backstay length and pylon height are 23.05 m and 15.75 m, respectively. The horizontal and vertical anchor-block reactions are 723.20 kN and 521.85 kN, producing a resultant reaction of approximately 891.8 kN according to Equation (1). As the span increases from 40 to 120 m, the horizontal reaction rises from 149.99 to 723.20 kN (382.2%), while the resultant reaction increases from 180.6 to 891.8 kN (393.8%). These changes demonstrate the increasing demands on the anchorage system and supporting ground. However, the relationship between span length and reaction forces is not necessarily linear because cable sag, tension, deck stiffness, load distribution, and support conditions influence structural equilibrium. Therefore, the values in Table 1 represent project-specific calculations rather than a general relationship applicable to all suspension bridges.

The 120 m span was adopted based on the proposed project configuration rather than a comprehensive economic optimization. Although shorter spans produce lower reactions, their relative suitability cannot be established without considering construction costs, material quantities, foundation requirements, hydraulic clearance, and lifecycle performance. Previous studies have highlighted the influence of geometric nonlinearity, tower flexibility, asymmetric loading, and cable behaviour on suspension bridge performance [6–10]. These findings provide

a theoretical basis for interpreting the present results, although direct quantitative comparisons require consistent structural configurations and loading conditions. The rigid, symmetrical arrangement provides a nominally balanced structural system. Nevertheless, unequal pedestrian loading, construction tolerances, cable adjustments, support displacement, and variations in foundation stiffness may introduce asymmetric responses. Accordingly, anchorage and pylon stability should be evaluated using the governing design load combinations rather than relying solely on geometric symmetry.

3.2. Geotechnical Characteristics and Seismic Classification.

The reported corrected average standard penetration test (SPT) value of 60.2 indicates relatively high penetration resistance within the investigated subsurface materials. However, this average does not fully represent vertical soil stratification or potential variations in foundation conditions. The four SPT and four cone penetration test (CPT) investigation points provide complementary geotechnical information, although their individual profiles, depths, and locations are necessary to establish spatial variability and identify potentially weaker layers. The project assessment classified the site as Site Class SC, with reported seismic parameters comprising a peak ground acceleration (PGA) of 0.252 g, short-period spectral acceleration (S_S) of 0.509 g, one-second spectral acceleration (S_1) of 0.230 g, and design spectral acceleration (SD_1) of 0.362 g. These parameters indicate that seismic considerations were incorporated into the project documentation. However, their derivation and consistency with the applicable seismic classification criteria and design response spectrum should be verified against the original calculations.

Seismic loading may influence tower base moments, cable tension, anchorage reactions, and foundation stability. The magnitude of these effects depends on structural flexibility, geometric nonlinearity, dynamic characteristics, and interactions among the deck, towers, and supporting foundations. Previous investigations have demonstrated the importance of earthquake excitation and nonlinear structural behaviour in cable-supported bridge systems [15, 17]. The present assessment focuses primarily on static stability and geotechnical performance. Therefore, its findings should be interpreted within the investigated design conditions rather than as confirmation of comprehensive seismic adequacy. Verification of the governing seismic load combinations, design response spectrum, and relevant dynamic responses remains important for establishing the overall seismic performance of the proposed bridge.

3.3. Anchor-Block Stability.

Table 2 summarizes the stability and deformation assessments of anchor blocks AB01 and AB02, including the calculated safety factors, adopted acceptance criteria, and corresponding safety margins. Positive margins indicate that the calculated safety factors exceed the minimum requirements or that the predicted deformations remain below the allowable limits. Both anchor blocks satisfy the adopted stability criteria. The overturning safety factors of 1.73 for AB01 and 1.63 for AB02 exceed the minimum requirement of 1.50, although AB02 has a smaller safety margin. The corresponding horizontal stability factors are 1.75 and 1.86, while the vertical safety factors reach 6.15 and 6.80, respectively. These results indicate satisfactory resistance to overturning, sliding, and vertical loading under the investigated design conditions.

Table 2. Stability Assessment of Anchor Blocks.

Stability parameter	AB01	AB02	Requirement	Margin: AB01	Margin: AB02
Overturning safety factor (–)	1.73	1.63	≥ 1.50	0.23	0.13
Vertical safety factor (–)	6.15	6.8	≥ 2.00	4.15	4.8
Horizontal safety factor (–)	1.75	1.86	≥ 1.50	0.25	0.36
Settlement (cm)	0.87	0.984	≤ 1.000	0.130 cm	0.016 cm
Lateral deformation (cm)	0.003	0.007	≤ 2.540	2.537 cm	2.533 cm

Note: Safety-factor margins are calculated as the reported factor minus the minimum criterion. Deformation margins are calculated as the allowable value minus the reported deformation. Values are based on the supplied project data.

The reported caisson bearing capacity of approximately 5,191.5 tonnes exceeds the factored axial demand of approximately 541.1 tonnes, indicating a substantial nominal capacity margin. However, this comparison requires confirmation that both quantities are based on compatible loading and resistance definitions. In particular, the original calculations should establish whether the reported bearing capacity represents an ultimate or allowable value and whether gross or net foundation resistance was considered. The capacity-to-demand ratio should not be interpreted as a formal safety factor without consistent definitions.

Settlement represents the principal serviceability consideration, particularly for AB02. As shown in Table 2, the calculated settlements are 0.870 cm for AB01 and 0.984 cm for AB02, both below the adopted limit of 1.000 cm. However, AB02 has a remaining margin of only 0.016 cm, indicating sensitivity to uncertainties in soil stiffness, groundwater conditions, foundation geometry, and construction disturbance. The calculated lateral deformations of 0.003 and 0.007 cm are substantially below the allowable limit of 2.540 cm, although these small values should be interpreted as model predictions rather than guaranteed field displacements. These findings reinforce the importance of anchorage stability in suspension bridge performance [12]. During construction, particular attention should be given to caisson dimensions, excavation conditions, concrete quality, reinforcement placement, and foundation alignment. Settlement monitoring is especially recommended for AB02 because its predicted deformation approaches the adopted serviceability limit.

3.4. Pylon Stability and Structural Capacity.

Table 3 presents the stability and deformation results for pylon foundations P1 and P2. Both foundations satisfy the adopted overturning, vertical, and horizontal stability requirements, although their settlement margins differ considerably. The overturning safety factors are 5.86 for P1 and 5.62 for P2, considerably exceeding the minimum requirement of 1.50. The vertical safety factors of 3.46 and 3.93 also satisfy the adopted criterion of 2.00, while the horizontal stability factors of 6.75 and 7.56 indicate substantial resistance to sliding. These results suggest adequate foundation stability under the evaluated loading conditions. The overturning safety factors are 5.86 for P1 and 5.62 for P2, considerably exceeding the minimum requirement of 1.50. The vertical safety factors of 3.46 and 3.93 also satisfy the adopted criterion of 2.00, while the horizontal stability factors of 6.75 and 7.56 indicate substantial resistance to sliding. These results suggest adequate foundation stability under the evaluated loading conditions.

Despite these favorable stability factors, settlement remains an important serviceability consideration. The predicted settlements are 0.930 cm for P1 and 0.995 cm for P2, compared with the allowable limit of 1.000 cm. The remaining margin for P2 is only 0.005 cm, making

its predicted performance particularly sensitive to uncertainties in subsurface conditions, soil stiffness, groundwater fluctuations, and construction execution. Meanwhile, the calculated lateral deformations of 0.051 and 0.086 cm remain substantially below the adopted limit of 2.540 cm.

Table 3. Stability Assessment of Pylon Foundations.

Stability parameter	P1	P2	Requirement	Margin: P1	Margin: P2
Overturning safety factor (–)	5.86	5.62	≥ 1.50	4.36	4.12
Vertical safety factor (–)	3.46	3.93	≥ 2.00	1.46	1.93
Horizontal safety factor (–)	6.75	7.56	≥ 1.50	5.25	6.06
Settlement (cm)	0.93	0.995	≤ 1.000	0.070 cm	0.005 cm
Lateral deformation (cm)	0.051	0.086	≤ 2.540	2.489 cm	2.454 cm

Note: Values are rounded to three decimal places where appropriate. Margins are calculated from the reported project results and adopted criteria.

The reported axial-capacity estimates for the pylon foundation are approximately 73,214.2 kN based on material capacity, 29,487.9 kN based on laboratory-derived soil parameters, and 163,362.8 kN based on SPT data. The laboratory-based estimate was adopted as the governing value because it represents the lowest reported capacity. However, the original calculations should be consulted to verify the comparability of these estimates and confirm that the governing resistance was determined using the appropriate design methodology.

The reported reinforcement arrangement includes D22 and D19 longitudinal bars with D13 transverse reinforcement for the anchor-block components, while the caisson rings incorporate D19 reinforcement with spiral reinforcement. The corresponding demand-to-capacity ratios are 0.464 for anchor-block axial loading, 0.474 for anchor-block shear, and 0.415 for pylon axial loading. These ratios indicate that the reported demands remain below their respective calculated capacities, provided that the applicable design resistances and governing load combinations were consistently applied. Confirmation of reinforcement adequacy nevertheless requires verification of the original section calculations and detailing requirements.

Overall, the results demonstrate that satisfactory ultimate stability does not necessarily guarantee adequate serviceability performance. Although both pylon foundations exhibit substantial resistance to overturning and horizontal loading, the limited settlement margin at P2 warrants particular attention. Together with AB02, this foundation should be prioritized for construction quality control and settlement monitoring to maintain the long-term structural and geotechnical performance of the proposed suspension footbridge.

3.5. Wind-Guy Foundation and Approach-Road Stability.

The wind-guy foundation has reported plan dimensions of 4.0 × 2.5 m and a depth of 2.0 m, with a calculated settlement of 0.36 cm. The maximum contact stress of 175.83 kN/m² represents approximately 35.9% of the allowable bearing pressure of 490.3 kN/m², providing a margin of 314.47 kN/m². This result indicates satisfactory bearing performance under the reported loading conditions. However, confirmation of overall foundation stability requires verification of the governing horizontal forces, potential uplift, overturning resistance, and applicable bearing-capacity calculations.

The approach roads have reported longitudinal gradients of approximately 17% on the Relung Mulya side and 14% on the Batanghari Ogan side. These relatively steep gradients may affect pedestrian accessibility, surface drainage, erosion control, and maintenance. Therefore, their operational suitability should be evaluated independently of embankment stability, particularly regarding pedestrian safety and accessibility requirements.

The proposed embankment fill has a unit weight of 1.72 t/m³ and an internal friction angle of 35°. The underlying soil at Batanghari Ogan has a unit weight of 1.77 t/m³, a friction angle of 32.2°, and cohesion of 0.37 t/m². The reported ultimate and allowable bearing pressures are 29.62 and 9.88 t/m², respectively, corresponding to a ratio of approximately 3.00. This ratio suggests an implied reduction factor of 3.00, subject to confirmation that both values were obtained using consistent calculation assumptions.

Table 4 summarizes the reported foundation performance and adopted acceptance criteria for the anchor blocks, pylons, and wind-guy foundation. The results indicate that the anchor blocks and pylons satisfy the specified stability requirements, although settlement margins are limited at AB02 and P2. The wind-guy foundation also satisfies the reported bearing-pressure comparison, but complete stability verification requires additional information from the original calculations.

Table 4. Summary of Foundation Performance and Adopted Acceptance Criteria.

Foundation component	Reported load/capacity information	Stability results	Settlement	Lateral deformation	Adopted criteria	Assessment status
Anchor blocks AB01–AB02	Caisson capacity: approximately 5,191.5 tonnes; reported factored axial demand: approximately 541.1 tonnes	Overturning FS: 1.73 and 1.63; vertical FS: 6.15 and 6.80; horizontal FS: 1.75 and 1.86	0.870 and 0.984 cm	0.003 and 0.007 cm	Overturning FS ≥ 1.50; vertical FS ≥ 2.00; horizontal FS ≥ 1.50; settlement ≤ 1.00 cm; lateral deformation ≤ 2.54 cm	Reported criteria satisfied; AB02 settlement margin is small
Pylon foundations P1–P2	Reported axial-capacity estimates: 73,214.2, 29,487.9, and 163,362.8 kN; governing value reported as 29,487.9 kN	Overturning FS: 5.86 and 5.62; vertical FS: 3.46 and 3.93; horizontal FS: 6.75 and 7.56	0.930 and 0.995 cm	0.051 and 0.086 cm	Overturning FS ≥ 1.50; vertical FS ≥ 2.00; horizontal FS ≥ 1.50; settlement ≤ 1.00 cm; lateral deformation ≤ 2.54 cm	Reported criteria satisfied; P2 settlement margin is very small
Wind-guy foundation	Allowable bearing pressure: 490.3 kN/m²; maximum contact stress: 175.83 kN/m²	Bearing stress below allowable pressure; complete stability factors not available	0.360 cm	Not reported	Contact stress ≤ allowable bearing pressure; settlement criterion subject to confirmation of project specification	Reported bearing and settlement results are within the stated values; full stability verification requires the original calculations

Note: FS denotes factor of safety. “Not reported” means that the relevant value was not included in the supplied material; it does not mean that the check failed or was necessarily omitted from the original design.

3.6. Integrated Structural and Geotechnical Discussion.

The findings highlight the importance of evaluating pedestrian suspension bridges as integrated structural and geotechnical systems. Cable forces are transmitted through the towers and anchorages to the foundations and surrounding soil, making structural stability dependent on foundation resistance, ground deformation, seismic conditions, and approach-road performance. For the proposed 120 m bridge, the calculated reactions and foundation safety factors indicate satisfactory performance under the investigated design conditions.

Although the anchor-block and pylon foundations satisfy the adopted stability requirements, settlement remains a critical serviceability consideration. As summarized in Table 4, AB02 and P2 have calculated settlements of 0.984 and 0.995 cm, respectively, approaching the allowable limit of 1.000 cm. Their remaining margins of 0.016 and 0.005 cm indicate sensitivity to uncertainties in soil properties, groundwater conditions, construction methods, and load distribution. These results emphasize the importance of verifying design assumptions and implementing settlement monitoring during construction and operation.

The geotechnical investigation reported relatively high penetration resistance and a seismic classification of Site Class SC. However, the reliability of the seismic assessment depends on the accuracy of the subsurface characterization and the derivation of the design-spectrum parameters. Earthquake excitation may alter cable tensions, tower base moments, anchorage reactions, and foundation deformations. Previous investigations have demonstrated the influence of seismic loading and nonlinear structural behaviour on cable-supported bridges [15,17]. Accordingly, the reported static stability results should be considered alongside the seismic assessments required by the applicable design standards.

The approach-road assessment further demonstrates that geotechnical stability and operational suitability must be considered separately. Although the proposed fill height remains below the calculated critical height and the reported slope safety factor is 9.70, the relatively steep approach gradients may affect pedestrian accessibility, drainage, erosion, and long-term maintenance. Verification of the slope stability model and evaluation of less favorable groundwater and loading conditions would strengthen confidence in the embankment design.

Unlike previous investigations primarily addressing aerodynamic stability, nonlinear structural response, and dynamic loading [1–10], this study integrates structural reactions, foundation stability, deformation performance, seismic site characterization, and approach-road conditions within a single project assessment. Its contribution lies in demonstrating how these interconnected engineering considerations influence the feasibility and safety of a rural pedestrian suspension bridge rather than developing a new structural analysis method.

Further assessment should prioritize verification of the original structural and foundation calculations, confirmation of seismic design parameters, and sensitivity analysis of soil strength and groundwater conditions. Where required by the governing standards, dynamic structural analysis and seismic load-combination checks should also be undertaken. Particular attention should be given to construction quality control and settlement monitoring at AB02 and P2 to support the bridge's long-term serviceability and structural performance.

4. Conclusions

This study evaluated the structural and geotechnical performance of a proposed 120 m rigid, symmetrical suspension footbridge crossing the Way Sekampung River in South Lampung,

Indonesia. The assessment integrated bridge geometry, support reactions, foundation stability, seismic site classification, and approach-road conditions. The proposed configuration comprises a 120 m main span, a 23.05 m backstay, a 35° cable inclination, and 15.75 m-high pylons. The calculated horizontal and vertical anchor-block reactions were 723.20 and 521.85 kN, respectively, yielding a resultant reaction of approximately 891.8 kN. The reported corrected average SPT value was 60.2, with the site classified as SC and seismic parameters of PGA = 0.252 g, S_S = 0.509 g, S_1 = 0.230 g, and S_D1 = 0.362 g. Both anchor-block and pylon foundations satisfied the adopted stability criteria. Overturning, vertical, and horizontal safety factors ranged from 1.63 to 1.73, 6.15 to 6.80, and 1.75 to 1.86 for the anchor blocks, compared with 5.62 to 5.86, 3.46 to 3.93, and 6.75 to 7.56 for the pylons. However, settlements at AB02 (0.984 cm) and P2 (0.995 cm) approached the allowable limit of 1.000 cm. The wind-guy foundation exhibited a settlement of 0.36 cm and a contact stress of 175.83 kN/m², below the allowable bearing pressure of 490.3 kN/m². The proposed embankment height of 2.29 m remained below the critical height of 5.74 m, with a reported slope safety factor of 9.70. Overall, the results indicate satisfactory performance under the investigated conditions, subject to verification of seismic parameters, loading assumptions, and foundation calculations. Construction quality control, settlement monitoring, and evaluation of approach-road accessibility and drainage are recommended to support long-term safety and serviceability.

Author Contribution

Conceptualization: B. Eko Septiyanto, Yoga Marta Agustiawan, Sari Utama Dewi. Methodology: B. Eko Septiyanto, Yoga Marta Agustiawan. Data Collection: Yoga Marta Agustiawan, Sari Utama Dewi. Data Analysis: B. Eko Septiyanto, Yoga Marta Agustiawan. Writing – Original Draft: B. Eko Septiyanto. Writing – Review and Editing: B. Eko Septiyanto, Yoga Marta Agustiawan, Sari Utama Dewi.

Competing Interest

The authors declare that they have no competing interests related to this study.

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Declaration of Generative AI Use

Generative AI tools (Gramarly/Quilbot/etc) were used solely to assist with language editing, grammar correction, and improving manuscript readability. The authors reviewed and verified the final content and take full responsibility for the accuracy, originality, and integrity of the manuscript.

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