

Article

Calculation of End Bearing Capacity and Shaft Resistance of Cast-in-Place Pile in Coral Reef Formations Considering Grout Penetration and Cementation

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Abstract

Existing methods for calculating the bearing capacity of cast-in-place piles in coral reef formations usually treat coral reef strata as ordinary sandy soil or conventional rock–soil media, without explicitly considering grout penetration and cementation in highly porous coral reef rock. This may lead to an incomplete evaluation of pile end bearing capacity and shaft resistance. To address this limitation, this study proposes a semi-empirical calculation framework that incorporates the contribution of the grout-induced cementation-enhanced zone. In the proposed model, the pile end bearing capacity is divided into three components: the intact coral reef rock contribution, the grout–coral reef rock cemented interface contribution, and the vertical effective stress term. The shaft resistance is divided into interface friction resistance and additional cementation-induced friction resistance. Key parameters were determined through an integrated procedure combining established laboratory and field techniques, including saturated weighing, mercury intrusion porosimetry, CT scanning, field coring, tracer observation, CT-based back-analysis, unconfined compression tests, and interface shear tests. For the investigated engineering case, the comprehensive porosity was 35%, the effective grout penetration radius was 0.118 m, the shear strength of intact coral reef rock was 2.5 MPa, and the shear strength of the cemented interface was 3.0 MPa. The calculated pile end bearing capacity was 1.76 MN, close to the field static load test value of 1.72 MN, with a relative difference of about 3%. The calculated total shaft resistance was 2.59 MN, compared with the measured value of 2.63 MN, with a relative difference of about 1.5%. The results suggest that considering the cementation-enhanced zone can better reflect the bearing response of cast-in-place piles in the investigated coral reef project. However, because the assessment is based on a single project and several parameters were obtained from local tests or back-analysis, the proposed method should be regarded as a site-specific semi-empirical framework. Further independent field tests are needed to examine its transferability and statistical reliability.

Keywords: coral sand geology; cast-in-place pile; grout cementation; end bearing capacity; shaft resistance



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1. Introduction

Coral islands and reefs are widely distributed in the offshore regions along the Maritime Silk Road, with great potential for infrastructure development in cross-sea bridges, ports and navigation channels, as well as industrial and civil buildings. However, key technical challenges in pile foundation design under coral reef geological conditions must first be solved. Coral reef sediments are predominantly composed of reef limestone, whose diagenetic process and post-depositional evolution are unique and fundamentally different from those of conventional terrigenous sediments, resulting in complex engineering properties [1–3]. Experimental tests demonstrate that coral sand primarily consists of biogenic carbonate minerals, with calcium carbonate constituting approximately 90% of its chemical composition. Minor terrigenous minerals, such as feldspar or quartz, may occur locally as detrital impurities [4]. Unlike ordinary terrigenous soils, coral reef formations are biogenic carbonate deposits characterized by weak cementation, high porosity, strong pore connectivity, and pronounced particle breakage. These characteristics make their pile-bearing behavior different from that of conventional sandy soils, clays, or intact rock masses [5,6].

Pile foundations are widely used in island-reef engineering because of their high bearing capacity, adaptability to complex geological conditions, and small post-construction settlement [7]. Cast-in-place piles are widely used in practice owing to their ease of construction and favorable deformation performance [8]. Based on full-scale static load tests and three-dimensional numerical analyses of SDCM piles in silty soil foundations, Wen et al. [9] reported that the ultimate compressive bearing capacity of SDCM piles can be significantly higher than the sum of the capacities of the cement mixing pile and the core pile considered separately. Based on back-analysis of the results of lateral static load tests on six cast-in-place piles, Wang et al. [10] found that increasing the pile diameter enhances the pile lateral bearing capacity, as a larger diameter mobilizes more surrounding soil to resist the load. Li et al. [11] applied Terzaghi's classical bearing-capacity equation to coral reef sandy soil foundations in an East African port project and compared the calculated results with field plate loading test results. Their study showed that the unmodified Terzaghi equation produced obvious deviations from the measured bearing capacity, and a modified Terzaghi equation was therefore proposed for coral reef sandy soil foundations. Similarly, Shen et al. [12] investigated the bearing capacity of coral sand foundations in the South China Sea through model plate loading tests and proposed a modified bearing-capacity formula for coral sand based on Meyerhof's formula. These studies demonstrate that conventional sand-based bearing-capacity theories have been used as starting points for coral sand foundation analysis, but they also show that coral sand cannot be simply treated as ordinary terrigenous sand because of its high carbonate content, irregular particle shape, high porosity, particle breakage, and strong compressibility.

Research on the bearing capacity of pile foundations has developed from classical bearing-capacity theory to load-transfer analysis and empirical design methods. Classical limit-equilibrium theories proposed by Meyerhof [13], Vesic [14], Terzaghi [15], established the basic framework for evaluating pile resistance by separating the total axial resistance into end bearing capacity and shaft resistance. Based on elastic continuum theory, Randolph and Wroth [16] further explained the load-transfer mechanism of vertically loaded piles, showing that the pile-head load is progressively transferred to the surrounding ground through shaft resistance, while the remaining load is carried by the pile base. Later, CPT-based and empirical design methods were developed. Schmertmann [17] showed that cone tip resistance and sleeve friction obtained from CPT tests can be used to characterize soil stratification, while Poulos and Davis [18] emphasized that pile settlement and internal force distribution are governed by pile–soil relative stiffness

and soil layering. Jardine et al. [19] further demonstrated that the axial capacity of piles in sand and clay is controlled by CPT indices and stress-path effects. More recent studies have extended load-transfer and simplified analytical methods to pile systems subjected to coupled mechanical and thermomechanical actions. Liu et al. [20] developed a simplified method for energy piles under combined horizontal and vertical loading. Song et al. [21] established a load-transfer method for dissimilar energy pile groups by considering pile–pile interaction, while Song et al. [22] investigated the long-term thermomechanical response of energy piles under inclined loading in saturated clay. These studies demonstrate that mechanics-based decomposition and load-transfer formulations can effectively represent complex pile responses under coupled loading conditions. Nevertheless, both conventional pile-capacity methods and these recent extensions are mainly developed for terrigenous sand, clay, or other conventional geomaterials. Their resistance formulations are generally based on intact ground strength, effective stress, pile–soil interaction, and conventional interface friction. They do not explicitly account for grout penetration into the connected pore–fracture network of highly porous coral reef rock, nor do they treat the resulting cementation-enhanced zone as a measurable and quantifiable load-transfer region. Therefore, their direct applicability to cast-in-place piles in coral reef formations remains limited.

Some researchers have investigated the engineering properties of coral sand and the bearing behavior of pile foundations. Wang et al. [23] reported through comparative field tests on large-diameter bored piles that post-grouting can improve the mechanical properties of the pile-tip and pile-side soil, increase ultimate resistance, and reduce pile-tip settlement. Wang et al. [24] further investigated the effect of base grouting on bored pile bearing capacity and indicated that post-grouting can significantly improve pile bearing behavior, although the uncertainty of field test data should also be considered. In calcareous sand, Wan et al. [25] showed that post-side grouting increases both shaft resistance and tip resistance by forming cement-stabilized soil around the pile. Their results also indicated that the improvement effect in calcareous sand can differ from that in siliceous sand because of particle breakage and the special structure of calcareous particles. In addition, Wan et al. [26] studied post-side-grouted piles subjected to lateral loading in calcareous sand and found that grouting can improve the lateral restraint and deformation control capacity of piles. Interface strengthening is another important factor controlling the load-transfer mechanism of cast-in-place piles in coral reef formations. Zhang et al. [27] found that the failure mode of the coral reef limestone–concrete interface depends strongly on the interface condition. When the interface is unfilled or covered by a mud film, failure is mainly governed by sliding friction; whereas, when concrete is directly cast onto coral reef limestone and a cemented interface is formed, failure may be controlled by internal defects in the coral reef limestone. Xu et al. [28] conducted direct shear and interface shear tests on pile–coral sand interfaces and showed that the interface shear strength exhibits distinct frictional characteristics. Pile installation effects also need to be considered when evaluating the bearing capacity of bored cast-in-place piles. During drilling, cleaning, slurry protection, concrete casting, and grout penetration, the soil or rock around the borehole may experience disturbance, loosening, mud-film formation, toe-debris accumulation, local blockage, and interface roughening. Yao et al. [29] quantitatively investigated the influence of defects such as toe debris, mud cake, necking, and bulging on cast-in-place bored piles, and showed that these construction-induced defects can significantly affect both ultimate bearing capacity and reliability. Despite these advances, existing approaches remain insufficient for cast-in-place piles constructed in highly porous coral reef rock. Conventional bearing-capacity and load-transfer methods quantify resistance mainly through intact geomaterial strength, effective stress, and pile–ground interface friction, while post-grouting studies generally assess the resulting improvement in total capacity or deformation. Neither approach

explicitly treats the grout-penetrated and hardened coral reef zone as a calculable load-transfer region shared by the pile base and shaft. Consequently, the effects of pore structure, grout penetration extent, and cementation strength are not connected within a unified calculation framework, and the additional resistance generated by cementation cannot be separated from the conventional resistance components. Therefore, a calculation framework is required for estimating the end bearing capacity and shaft resistance of cast-in-place piles in coral reef formations, which can incorporate the influence of pore structure, grout penetration, and cementation while allowing the key parameters to be determined through a limited number of field and laboratory tests.

To address this issue, this study focuses on establishing a regional semi-empirical calculation framework for the end bearing capacity and shaft resistance of cast-in-place piles in coral reef formations by considering grout penetration and cementation effects. The study has three specific objectives. First, a load-decomposition framework is formulated, in which the pile end bearing capacity is divided into the contribution of the intact coral reef rock mass, the contribution of the cemented interface, and the effective-stress term, while the shaft resistance is divided into interface friction resistance and cementation-induced additional resistance. Second, the key parameters required by the calculation framework, including porosity, effective grout penetration radius, interface shear strength, and cementation-related shear strength, are determined using established field and laboratory techniques, such as field coring, CT-based back-analysis, interface shear tests, and strength tests on grouted cores. Third, a preliminary engineering case assessment is conducted by comparing the calculated results with field static load test data from one island-reef project. Therefore, the present study should be regarded as a regional semi-empirical calculation and parameter-determination framework, rather than a complete theoretical validation of the load-transfer mechanism.

2. Pore Structure and Cementation Effects of Coral Sand

Coral sand and coral reef rock formations are characterized by high porosity and strong pore connectivity. Their mechanical behavior differs substantially from that of conventional dense sands. During the construction of cast-in-place piles, the grout not only forms the pile body but also penetrates into the surrounding strata and hardens, thereby affecting both the end bearing capacity and shaft resistance of the pile. Figure 1 presents a schematic of grout migration from the pile or injection hole into the surrounding coral reef rock.

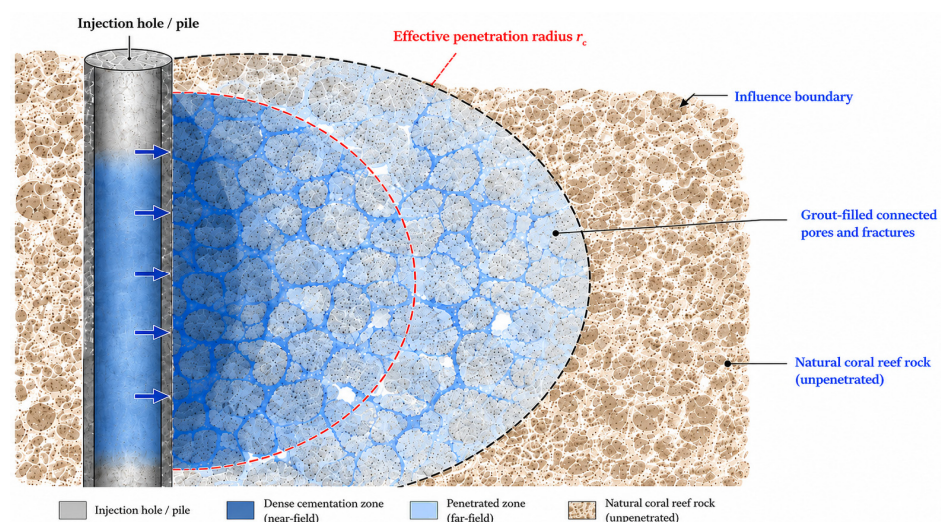


Figure 1. Schematic illustration of grout penetration in coral reef rock.

2.1. Penetration and Cementation Process of Cast-in-Place Piles in Coral Sand

During the construction of cast-in-place piles in coral reef rock, the coral reef rock generally exhibits large pore size and strong connectivity. Therefore, during casting, the grout not only fills the borehole to form the pile body, but also diffuses and penetrates the surrounding rock through connected pores and fractures intersecting the rough borehole wall under the combined effects of pressure and gravity. During the penetration process, the grout typically undergoes filtration, deposition, local blockage, and progressive attenuation of seepage. During setting and hardening, hydration and cementation occur within the penetrated zone, hydration and cementation occur within the penetration zone, ultimately forming a cemented composite of hardened grout and coral reef particles.

2.2. Three-Component Superposition Mechanism of Pile End Bearing Capacity

Traditional analyses of pile end bearing capacity usually consider only the strength and stress state of the intact rock mass. However, under grout penetration and cementation, the pile base is controlled not only by the strength of the intact coral reef rock itself, but also by the cementation-enhanced zone and the effective stress, which together increase the pile end bearing capacity.

In coral reef formations, the end bearing capacity of a cast-in-place pile depends not only on the strength of the intact rock mass but is also affected by the enhancement effect of the cemented interface formed between the grout and the rock during casting, as well as by the effective stress. Therefore, the analysis of pile end bearing capacity should consider the combined effects of the bearing contribution of the intact rock mass, the contribution of the cemented interface, and the effective stress. The bearing contribution of the rock mass refers to the resistance provided by the intact coral reef rock beneath the pile base, which is governed by the shear strength and compressive strength of the rock itself. In conventional calculations of pile end bearing capacity, only the strength of the intact rock mass is usually considered, that is the end bearing capacity is conventionally attributed solely to the shear failure or crushing failure of the intact rock. The contribution of the cemented interface refers to the interface formed after the grout penetrates the rock mass and then sets and hardens within the pores around the pile base. During compression and shear, this cemented interface can share part of the applied load, thereby increasing the ultimate end bearing capacity of the pile base. Effective stress is also an important factor controlling the mobilized resistance at the pile base. As the embedment depth increases, the vertical effective stress at the pile base increases, which can directly enhance the shear strength of the intact medium and help suppress damage to the pore structure, thereby improving the overall stability of the pile base (Figure 2).

Mechanical justification for the three-component superposition. The three-component superposition adopted in this study can be interpreted from the vertical force equilibrium of the pile base at the ultimate limit state. When the axial load is transferred to the pile tip, the end bearing resistance is mobilized by the coral reef rock beneath the pile base, the grout–coral reef cemented transition zone formed by grout penetration and hardening, and the confinement associated with the vertical effective stress at the pile base. The intact rock contribution is mainly governed by shear or local crushing failure within the original coral reef rock; whereas, the cemented-interface contribution is mobilized along the grout–rock boundary or within the grouted coral reef composite. These two strength-controlled components correspond to different load-transfer regions: the former is mainly developed in the intact rock skeleton beneath and around the pile base, while the latter is limited to the grout-penetrated zone characterized by the effective penetration radius r_c , which can be identified by field coring and CT imaging. Therefore, the two components can be approximately separated in engineering calculation. The vertical effective stress term is

not regarded as an independent material resistance, but as a confinement contribution that increases the normal stress on potential shear surfaces and thus enhances the mobilized shear strength according to the Mohr–Coulomb criterion. Accordingly, the proposed three-component model represents a first-order load-decomposition method based on distinguishable resistance mechanisms and vertical force equilibrium. It should be noted that the three components are not assumed to be completely independent. Possible interaction effects, such as stress redistribution between the intact rock and the cemented zone, are considered indirectly through reduction coefficients and experimentally determined strength parameters. A more rigorous coupled analysis of the pile–cemented zone–coral reef rock system requires numerical simulation or instrumented field tests, which is beyond the scope of the present semi-empirical framework and will be further investigated in future work.

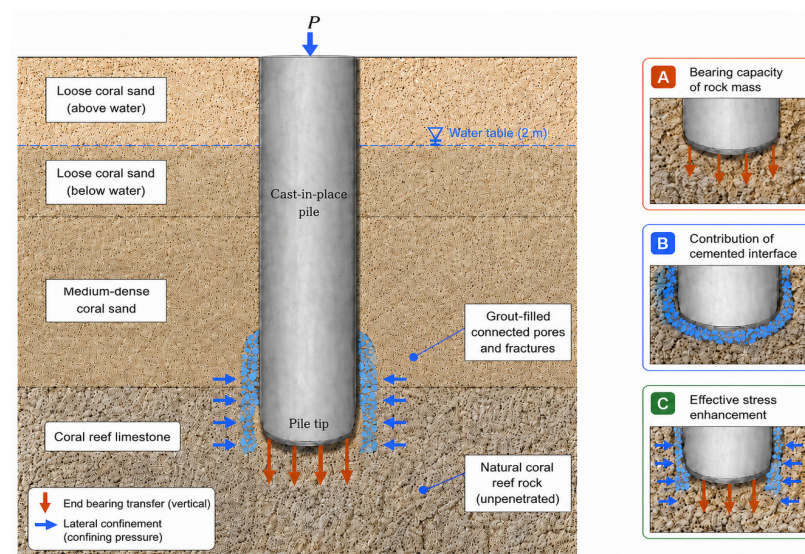


Figure 2. Schematic diagram of three-component superposition at the pile tip.

2.3. Two-Component Mechanism of Shaft Resistance

Shaft resistance originates from the contact between the pile and the surrounding ground. When the superstructure load is applied at the pile head, it is transferred to the surrounding formation through relative displacement along the pile–ground interface, relative displacement occurs at the pile–ground interface, thereby generating shaft resistance through interface shear. During the construction of cast-in-place piles in coral reef rock, shaft resistance is a key factor affecting the bearing capacity of the pile foundation. Compared with shaft resistance under conventional geological conditions, shaft resistance in coral reef rock has distinct characteristics, mainly because of grout penetration and cementation. The shaft resistance in coral reef rock can be analyzed in terms of two components: interface friction resistance and additional friction resistance induced by penetration-related cementation. Interface friction resistance refers to the frictional resistance generated between the pile-side concrete and the coral reef rock under effective normal stress. This is the basic concept of conventional shaft resistance and is also applicable to coral reef rock geological conditions. The additional friction resistance induced by penetration-related cementation is a unique shaft resistance mechanism under coral reef rock geological conditions. This component of resistance is provided by the cementation-enhanced ring formed along the pile shaft through grout penetration and cementation.

Mechanical justification for the two-component decomposition. The two-component shaft resistance model can be interpreted from the shear stress integration over the effective load-transfer area along the pile shaft. For the (i)-th stratum, the conventional interface

friction resistance is mobilized on the original geometric pile–rock contact area, $A_0 = \pi D L_i$, where D is the pile diameter and L_i is the thickness of the i -th layer. This component is mainly governed by the effective normal stress and the pile–coral reef rock interface friction angle, following the Coulomb friction mechanism. In coral reef rock, however, grout can penetrate radially into the connected pores and fissures around the borehole wall and harden to form a cementation-enhanced ring around the pile shaft. This cemented ring increases the effective shear-transfer area and introduces an additional resistance component associated with the shear strength of the grouted coral reef composite. The additional effective area can be expressed as $A_c = \xi A_0$, where ξ is the grout interlocking area coefficient defined in Section 3.2.2. Therefore, the shaft resistance is decomposed into the conventional interface friction component and the cementation-induced additional component. The former is mobilized mainly along the direct pile–rock contact surface; whereas, the latter is mobilized within the grout-filled pore network and the cemented ring surrounding the pile. Since the cemented ring is bonded to the pile concrete, these two components can be approximately mobilized under compatible shear displacement at small strains and near the peak resistance state. Accordingly, their contributions are summed in the proposed engineering calculation framework. It should be emphasized that this decomposition does not imply complete independence between the two components. Local debonding, non-uniform grout penetration, progressive interface failure, and strain softening of the cemented zone may cause interaction between the two mechanisms. In this study, such effects are considered indirectly through the grout interlocking coefficient, interface strength parameters, cementation-related shear strength, and reduction factors. The comparison with the field static load test in Section 5.2 provides preliminary support for this decomposition, but further instrumented pile load tests with strain gauges or distributed fiber-optic sensing are required to independently verify the mobilization of the two shaft-resistance components.

Figure 3 schematically illustrates the correspondence between the proposed two-component shaft resistance model and the physical load-transfer paths along the pile shaft. It shows that the shaft resistance is interpreted as the combined contribution of conventional interface friction and the additional shear/interlocking resistance mobilized by the grout-cemented zone.

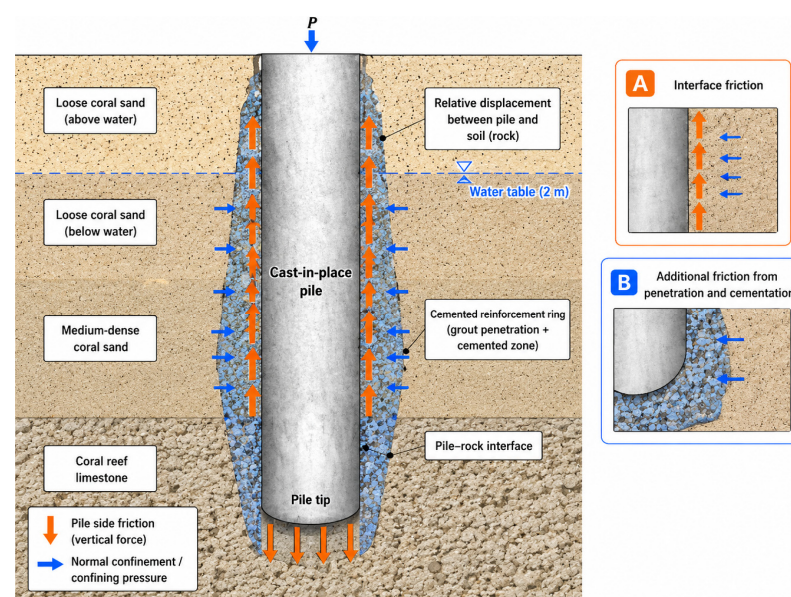


Figure 3. Schematic diagram of the two-component shaft resistance mechanism.

2.4. Differences from Conventional Conditions

Under conventional conditions, coral sand is generally treated as a dense sand layer. In dense sand, the development of end bearing capacity and shaft resistance is governed mainly by effective stress. Both pile end bearing capacity and shaft resistance depend strongly on the density of the sand and the level of effective stress. In particular, shaft resistance is mainly determined by the lateral earth pressure and the frictional behavior at the pile–soil interface. Because the pore size of sand is relatively small, and grout is prone to filtration and blockage during penetration, the continuous penetration and hardening of grout in the soil surrounding the borehole are usually limited. Therefore, the pile–soil contact interface can be approximately regarded as a frictional interface between concrete and sand, and the shaft resistance is mainly controlled by the interface friction characteristics and the effective stress state.

3. Parameters for Calculating the Bearing Capacity of Cast-in-Place Piles in Coral Sand Geology

In Section 2, the grout penetration and cementation process induced by cast-in-place pile construction in coral sand strata was systematically analyzed, together with the load-transfer mechanisms at the pile base and along the pile shaft. The preceding discussion indicates that, after grout penetration and hardening, the cementation-enhanced zone changes the load-bearing mode of the cast-in-place pile from the conventional single contact between the pile and the surrounding formation into a composite load-bearing system involving the pile, the cementation-enhanced zone, and the surrounding formation. Accordingly, the pile end bearing capacity consists of three superimposed components, while the shaft resistance is composed of two parts. Therefore, to convert the above mechanistic understanding into a mathematical model applicable to engineering calculation, a series of related tests was carried out. To apply the load-decomposition model proposed in this study, the required input parameters must first be determined. These parameters include the porosity of coral reef rock, the effective grout penetration radius, the shear strength of intact coral reef rock, the shear strength of the cemented interface, the cementation-related shear strength, the grout interlocking area coefficient, and the interface friction angle.

3.1. Key Parameters and Experimental Methods for End Bearing Capacity

3.1.1. Integrated Determination of Porosity n

(1) Saturated weighing method

Porosity was determined using the saturated weighing method to obtain the upper-bound porosity value n_1 . The intact coral reef rock core was oven-dried at 100–105 °C for 24 h to obtain the dry mass m_d . A 24 h water-saturation test was then conducted under vacuum conditions to obtain the saturated mass m_w .

The upper-bound porosity value n_1 was calculated using the saturated porosity Equation (1):

$$n_1 = \frac{(m_w - m_d)/\rho_w}{v} \times 100\% \quad (1)$$

where ρ_w is the density of water, and v is the bulk volume of the coral reef rock core specimen.

(2) Mercury intrusion porosimeter

Porosity was also determined by porosimeter. The intact coral reef rock core was cut into slices 10 mm thick and oven dried. The dried slices were then placed in the porosimeter and subjected to pressure infiltration for 24 h to obtain the lower-bound porosity value n_2 .

The relative difference between the upper and lower bounds of porosity was then analyzed calculated Equation (2).

$$\varepsilon = 100\% \times (n_2 - n_1) / n_1 \quad (2)$$

The porosity n was determined using an integrated procedure combining saturated weighing, mercury intrusion porosimetry, and CT scanning. Saturated weighing was first used to obtain the bulk-scale porosity of the intact coral reef rock core. Mercury intrusion porosimetry was then used as an independent check. If the relative difference between the two results was not greater than 5%, the saturated weighing result was adopted as the representative engineering porosity. If the relative difference exceeded 5%, CT scanning was further introduced to identify the pore structure, and the final porosity was determined by combining the saturated weighing result and the CT-based porosity.

(3) CT scanning method

Three-dimensional reconstruction of the intact coral reef rock core was performed using industrial CT scanning at a resolution of 18 μm . The scanned porosity n_3 was obtained by local threshold segmentation. The saturated weighing method and the CT scanning method were then combined using Equation (3), so that the isolated pores and connected pores in the coral reef rock could be unified into an operational best-estimate porosity n :

$$n = kn_1 + (1 - k)n_3 \quad (3)$$

where k is the weighting coefficient, generally taken as 0.6–0.8. The linear combination in Equation (3) is motivated by the fact that the saturated weighing method captures the total porosity (connected + isolated pores); whereas, mercury intrusion only accesses the connected pore network; CT scanning, in turn, resolves the spatial pore geometry but is limited by the resolution threshold and partial-volume effects. A weighted average therefore unifies the isolated-pore information from the saturated measurement with the connected-pore geometry from CT into a single operational best estimate. The range $k = 0.6\text{--}0.8$ was determined from a sensitivity analysis on eight core specimens from the study site: varying k within this range changed the best-estimate porosity n by less than 1.2 percentage points; whereas, the saturated-only and CT-only estimates differed by up to 3.5 percentage points. The recommended value $k = 0.7$ is adopted in the case study of Section 5.

3.1.2. Method for Determining the Effective Grout Penetration Radius

(1) Definition and direct measurement of the effective grout penetration radius

The effective radius r_c refers to the equivalent distance from the center of the borehole to the boundary of effective grout penetration, following the penetration and diffusion of the cement grout through the pores of the coral reef rock during the grouting process and its subsequent hardening to form a cemented reinforcement zone. In engineering applications, the infiltration scale of grout within coral reef rock fractures is jointly governed by multiple factors including porosity, connectivity, pore size distribution, grouting pressure, and water–cement ratio. Relying solely on in situ core tracing or CT-seepage coupled inversion to determine r_c on a pile-by-pile basis not only involves high costs and long testing periods but also fails to meet the engineering design phase demand for rapid parameter acquisition and batch evaluation. Therefore, this study proposes establishing an empirical relationship between “porosity n and effective penetration radius r_c ”. The objective is to equate the complex, labor-intensive experimental process with a convenient empirical parameter for engineering applications.

The empirical formula was developed following the principle of limited calibration, regional application; within specific operational conditions, a finite number of field-injected

core sampling test points were deployed to obtain a dataset using tracer methods or CT inversion techniques. Subsequently, statistical regression analysis is performed on the data to derive coefficients a and b for the empirical formula. After completing a single regional calibration, subsequent similar projects only require measuring the in situ core porosity n .

(2) Regional empirical calibration

Field casting-and-coring tests can generally be used to establish the regional relationship between r_c and n for coral reef rock. The specific procedure is as follows. Under engineering conditions with a grout water–cement ratio of 0.45–0.55 and a grouting pressure of 0.2–0.4 MPa, grouted core specimens are prepared in the field. Grout is first cast into drilled holes. The test points should not only cover locations near the design pile positions but should also be distributed across multiple islands and reefs in the region according to the principle of uniform spatial distribution. The drilling depth is taken as the pile base depth, the borehole diameter is $\phi 110$ mm, and the grouting volume is controlled at 1/20 of the design grout volume for a single pile. After 24 h of grouting, grouted core specimens are extracted, with a specimen diameter of $\phi 75$ mm. The effective grout penetration radius is mainly affected by grouting pressure, grout viscosity, pore connectivity, pore-throat size distribution, and filtration or local blockage during grout migration. In this study, the grouting pressure and water–cement ratio are kept within narrow ranges. Therefore, the pressure gradient and grout viscosity can be approximately regarded as stable. Under these conditions, the variation in penetration radius is mainly controlled by the pore structure of coral reef rock. Statistical regression is performed using all test data to establish the regional relationship between r_c and n for the coral reef rock Equation (4):

$$r_c = a + bn \quad (4)$$

where a is the penetration-radius offset parameter, and b is the porosity weighting parameter.

(3) Recommended values and applicability range

To determine the effective radius r_c , tracer methods or CT-seepage coupled inversion methods can be used. The tracer method involves placing a slurry-impregnated core under a UV lamp to observe and measure the thickness of the fluorescent ring. Continuous coring is carried out outward from the concrete zone of the test hole until the edge of the penetration zone is reached. Based on the statistical analysis of the fluorescent measurements, the tracer-based effective radius r_{c1} is obtained. In the CT-seepage coupled back-analysis method, the grouted core is scanned by CT at a resolution not greater than 18 μm to obtain three-dimensional images. Using Avizo 9.5.0 three-dimensional threshold segmentation is performed to isolate the penetration zone, and the total grout-filled volume V_f is calculated. Based on a cylindrical diffusion model, the CT-based effective radius r_{c2} is back-calculated.

When $r_{c1} - r_{c2} \leq 10\text{mm}$, r_c is taken as the tracer-based effective radius. When $r_{c1} - r_{c2} > 10\text{mm}$, r_c is taken as the CT-based effective radius r_{c2} . If field conditions do not permit the preparation of grouted core specimens or the above tests cannot be conducted, the porosity n , may be determined from intact coral reef rock cores, and the empirical effective radius r_{c3} .

Based on the regional regression dataset described above ($n = 12$ grouted-core test points from the South China Sea site), the following empirical formula can be used for estimation, where the empirical coefficients are taken as $a = 0.1$ and $b = 0.05$. The coefficients $a = 0.1$ and $b = 0.05$ were obtained by least-squares regression on the 12-point dataset; the corresponding 95% confidence intervals are a in $[0.07, 0.13]$ and b in $[0.038, 0.062]$ ($R^2 = 0.82$, standard error of estimate = 0.014 m). Because the regression is based on a single regional

dataset, the coefficients should be regarded as site-calibrated values rather than universal constants; re-calibration is recommended when the method is applied to reefs of different facies or under grouting conditions outside the stated ranges ($n = 0.2\text{--}0.5$, grouting pressure $0.2\text{--}0.4$ MPa, water–cement ratio $0.45\text{--}0.55$). The expression is shown in Equation (5):

$$r_{c3} = 0.1 + 0.05n \quad (5)$$

This equation is applicable to engineering conditions where the porosity n ranges from 0.2 to 0.5, the grouting pressure is $0.2\text{--}0.4$ MPa, and the water–cement ratio is $0.45\text{--}0.55$. The fixed coefficients are not universal: they reflect the pore-throat geometry and filtration behavior of the South China Sea reef limestone from which they were calibrated. Extrapolation to other reef provinces (e.g., the Great Barrier Reef, the Maldives, or Red Sea reefs) should be preceded by a local re-calibration using at least three field grouted-core test points.

3.1.3. Shear Strength of Coral Reef Rock Mass

Grouted coral reef rock cores taken directly beneath the pile base were collected, and shear tests were conducted to determine the shear strength of the coral reef rock mass f_r . Cemented interface specimens formed by grout penetration into the coral reef rock beneath the pile base were also collected, and interface direct shear tests were carried out to determine the interface shear strength f_t . The interface direct shear tests were conducted under saturated conditions, with the normal stress ranging from 20 kPa to 400 kPa, and the peak shear strength was taken as f_t .

The interface shear specimens were prepared from intact coral reef rock cores and grouted interface specimens. The intact cores were cut into $60\text{ mm} \times 60\text{ mm} \times 60\text{ mm}$ blocks and saturated under vacuum for 24 h before testing. For the grouted interface specimens, cement grout with a water–cement ratio of 0.5 was cast against the intact coral reef rock surface to reproduce the grout–rock interface formed during pile construction. After casting, the grouted specimens were cured in a fog chamber at $20\text{ }^\circ\text{C}$ and relative humidity greater than 95% for 28 days.

The interface direct shear tests were conducted under fully drained conditions. Free drainage was provided through porous stones during loading. For each test, the prescribed normal stress was first applied, and a 30 min consolidation pause was maintained at each normal-stress increment before shearing. The horizontal shear displacement was then applied under displacement control at a rate of 1.0 mm/min . This loading rate was selected to ensure drained shearing of the carbonate rock interface specimens.

To ensure repeatability, each normal-stress level was tested using at least three parallel specimens. The peak shear stress obtained from the shear stress–displacement curve was taken as the interface shear strength under the corresponding normal stress. The coefficient of variation in the peak shear strength was below 12% for all groups, indicating acceptable repeatability of the interface shear test results.

The interface direct shear test was performed using a direct shear-type interface shear box apparatus, as shown in Figure 4. The upper and lower shear boxes can slide relative to each other in the horizontal direction to form a controlled shear plane. The normal loading system applies a constant normal stress through a lever or servo loading device. The shear loading system imposes horizontal shear displacement under displacement control. The measurement system includes a normal load sensor, a shear force sensor, and displacement gauges for real-time recording of the shear stress–displacement relationship. The interface shear box is of square shape, with an internal dimension of $60\text{ mm} \times 60\text{ mm}$ for each half-box, which ensures a sufficient shear area and reduces the influence of boundary effects on the test results. The apparatus used was a strain-controlled direct shear apparatus

(model DJY-4, Nanjing Tekra Instruments, Nanjing, China), with a normal load capacity of 10 kN (resolution 1 N), a shear load capacity of 5 kN (resolution 1 N), and displacement transducers with a range of 10 mm and a resolution of 0.001 mm. The data were acquired at 1 Hz through a dedicated data-logging system.

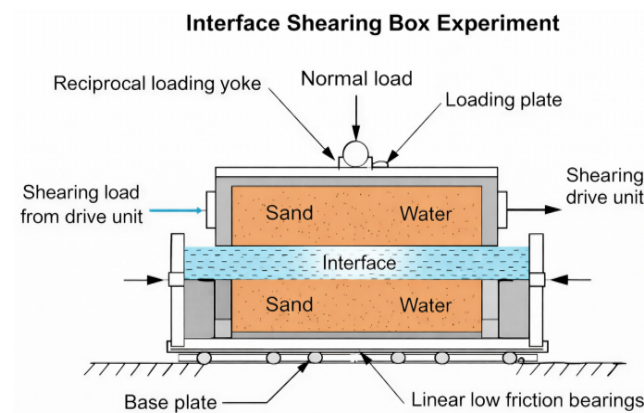


Figure 4. Schematic diagram of interface shear box.

3.1.4. Shear Strength of Intact Coral Reef Rock

Through systematic testing, an empirical strength equation reflecting regional characteristics can be established, thereby significantly reducing the workload associated with large-scale field grouting tests and enabling more rapid bearing-capacity calculation and design planning for projects in the same or adjacent areas. To this end, under commonly adopted engineering conditions, with a grouting pressure of 0.2–0.4 MPa and a grout water–cement ratio of 0.45–0.55, emphasis may be placed on investigating the empirical relationship between the shear strength of intact coral reef rock f_r and the unconfined compressive strength q_u . First, intact saturated coral reef rock core specimens are collected directly beneath the pile base. The unconfined compressive strength q_u of the coral reef rock is determined through saturated unconfined compression tests, which are conducted in strict accordance with the Standards Compilation Group [30]. Meanwhile, the shear strength f_r is determined through shear tests, and the relationship between f_r and q_u is shown in Equation (6).

The assumed linear proportionality between shear strength and unconfined compressive strength follows from the Mohr–Coulomb failure criterion. For a saturated, low-cohesion coral reef rock tested under unconfined conditions, the shear strength at failure on the critical plane is $\tau = c + \sigma_n \tan \varphi$, and the uniaxial compressive strength is $q_u = \frac{2c \cos \varphi}{1 - \sin \varphi}$ for a purely cohesive-frictional material. When the friction angle φ varies over a narrow range within a single reef facies, the ratio τ/q_u is approximately constant, so that $\tau = \lambda q_u$ is a valid first-order linearization. The proportionality coefficient λ is therefore not arbitrary but is the shear-to-compression ratio implied by the Mohr–Coulomb parameters of the rock; the regression slope in Figure 5 gives the regional value of λ directly.

$$f_r = \omega_1 q_u \quad (6)$$

where f_r is the shear strength of the coral reef rock mass, q_u is the unconfined compressive strength, and ω_1 is the shear-to-compression ratio of the intact coral reef rock core.

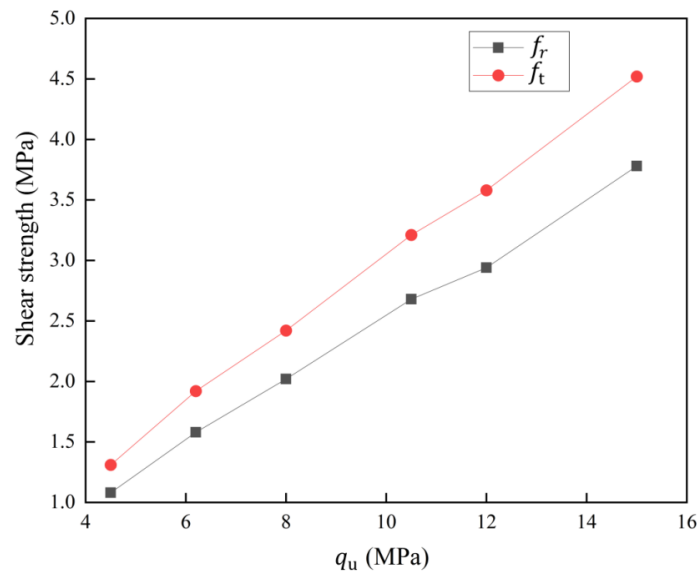


Figure 5. Results of coral reef–cement slurry interface shear tests.

Similarly, grouted core specimens are collected from the same stratum directly beneath the pile base during field casting. The shear strength of the grouted core f_t is determined by interface direct shear tests, and the relationship between f_t and q_u is shown in Equation (7).

$$f_t = \omega_2 q_u \quad (7)$$

where f_t is the shear strength of the grouted core, q_u is the unconfined compressive strength, and ω_2 is the shear-to-compression ratio between the grouted core and the intact coral reef rock core.

Specimens with different strength grades were selected for unconfined compressive strength tests and interface direct shear tests to obtain their unconfined compressive strength, the shear strength of the intact rock mass, and the shear strength of the grouted rock mass. By comparing and analyzing the test results of each specimen group, the empirical relationship between shear strength and unconfined compressive strength can be further established. Figure 5 and the subsequent tables present the results of field tests conducted on samples collected from the South China Sea, the values of ω_1 and ω_2 were determined by the slope of the graph.

3.2. Key Parameters and Experimental Methods for Shaft Resistance

3.2.1. Cementation-Related Shear Strength

To determine the mechanical properties of grouted cores, grouted core specimens were collected for shear testing. According to the design formation pressure, graded normal loads were applied to the grouted core specimens. Through interface direct shear tests, the ultimate shear stress at interface failure under different levels of normal stress was obtained, thereby determining the cementation-related shear strength τ_i for different strata. Furthermore, based on the Mohr–Coulomb failure envelope, the interface friction angle δ_i for each stratum was obtained by fitting.

By fitting the shear strength under different normal stresses, interface strength parameters applicable to engineering calculation can be obtained, providing a basis for subsequent analysis of the shear bearing capacity of the cemented reinforcement ring along the pile shaft. At the same time, these parameters can also be used to characterize the differences in grout diffusion and cementation effects under different stratigraphic conditions,

thereby providing experimental support for the determination and layered calculation of shaft resistance.

3.2.2. Grout Interlocking Area Coefficient

The grout interlocking area coefficient is used to characterize the relative extent of the effective cemented contact zone formed along the pile side surface after grout penetrates around the pile. It reflects the degree to which grout enlarges the effective interaction area at the pile–soil interface. A larger value of this coefficient indicates a wider effective penetration and cementation zone around the pile, a greater effective interface area participating in load transfer along the pile shaft, and, accordingly, a more significant contribution of grout to the enhancement of shaft resistance.

The grout interlocking area coefficient is defined as Equation (8):

$$\xi = \frac{2r_c}{D} \times \left(1 + \frac{r_c}{D}\right) \quad (8)$$

where r_c is the effective radius, and D is the pile diameter.

Physical derivation of the grout interlocking coefficient. Consider a pile of diameter D surrounded by an annular cemented ring of radial thickness r_c . The load-transfer perimeter along which the pile concrete is bonded to the cemented ring is πD , while the additional perimeter over which the cemented ring is bonded to the surrounding intact rock is $\pi(D + 2r_c)$. The ratio of the additional cemented contact area to the original pile–soil contact area, per unit pile length, is therefore $\frac{\pi(D+2r_c) - \pi D}{\pi D} = \frac{2r_c}{D}$. In the present definition the grout interlocking area coefficient ξ is taken as $\frac{2r_c}{D}$, which represents the fractional increase in the effective shear-transfer area due to grout penetration; this is a geometric ratio with a clear physical meaning rather than an empirical fitting factor. When the pile diameter D is approximately one order of magnitude larger than r_c , the term $\left(\frac{r_c}{D}\right)^2$ is negligible and the area coefficient reduces to $\xi = \frac{2r_c}{D}$, consistent with the simplified form given in Equation (8).

When $D \gg r_c$, that is, when D is approximately one order of magnitude greater than r_c , the term $\frac{r_c}{D}$ may be neglected. The calculation formula for the grout interlocking area coefficient can then be simplified as $\xi = \frac{2r_c}{D}$.

3.2.3. Unconfined Compressive Strength of Coral Reef Rock

Specimen preparation and parameter determination were carried out by collecting intact cores. Within the engineering site, intact saturated coral reef rock cores were collected according to stratigraphic layers. The unconfined compressive strength was then determined by conducting saturated unconfined compression tests on the above intact cores in strict accordance with the Standards Compilation Group [30] and the unconfined compressive strength of each stratum q_{ui} was obtained. Preparation of grouted cores and determination of cementation-related shear strength were then performed. Grouted core specimens were prepared and collected at the corresponding stratigraphic positions following the field test-hole procedure described above. Shear tests were subsequently conducted on the grouted cores under saturated conditions. At least four levels of normal stress, namely 50 kPa, 100 kPa, 200 kPa, and 400 kPa, were applied, and the peak shear strength under each loading level was taken to determine the cementation-related shear strength τ_i for the corresponding stratum.

To establish a regional empirical relationship, regression analysis was performed based on the test data of each stratum, and the relationship between cementation-related

shear strength and unconfined compressive strength $\tau_i - q_{ui}$ for the region was obtained as follows Equation (9):

$$\tau_i = a \times q_{ui} \quad (9)$$

where a is the shear-to-compression ratio, τ_i is the cementation-related shear strength, and q_{ui} is the unconfined compressive strength.

For coral reef rock with relatively low porosity and poor pore connectivity, field coring or CT observations may show only limited grout penetration around the pile shaft. In such cases, if interface shear tests also indicate no obvious cementation-related contribution, the grout interlocking area coefficient ζ may be neglected in preliminary calculations. Otherwise, ζ should be calculated according to the effective grout penetration radius and pile diameter. In the investigated project, an unconfined compressive strength of approximately 5 MPa was used only as a site-specific auxiliary reference for distinguishing relatively weak.

In this case, the calculation formula for shaft resistance considers only the interface friction term. The experimental data are shown in Table 1. To simplify the experimental procedures in practical engineering applications, a is set to 0.3 based on sampling data from a specific location in the South China Sea. Equation (9) should be interpreted as a regional first-order strength-correlation equation rather than a universal theoretical relationship. The use of a proportional relationship between cementation-related shear strength and UCS can be understood within the Mohr–Coulomb strength framework. For grouted coral reef composite specimens with similar lithology, pore structure, saturation condition, and curing condition, both the cementation-related shear strength and UCS are mainly governed by the cementation degree, pore structure, and integrity of the carbonate skeleton. Therefore, within the calibrated strength range, the cementation-related shear strength can be approximated as a proportional function of UCS.

Table 1. Coral reef–cement slurry interface shear box test.

Test Specimen	q_{ui} (MPa)	τ_i (MPa)	a
1	3.5	1.02	0.291
2	5.0	1.48	0.296
3	6.5	1.95	0.300
4	8.0	2.42	0.303
5	10.0	3.05	0.305
6	12.0	3.58	0.298
7	15.0	4.52	0.301
8	18.0	5.38	0.299

The coefficient (a) in Equation (9) is the regional shear-to-compression ratio of the grouted coral reef composite. It was determined by regression of the UCS and interface shear test results of grouted core specimens from the study site. As shown in Table 1, the ratio (τ_i/q_{ui}) remains approximately stable around 0.30 within the tested strength range. Therefore, ($a = 0.30$) is adopted in this study. This coefficient should not be regarded as a universal constant and should be recalibrated when the lithology, pore structure, grout material, curing condition, or specimen scale differs significantly from the present study.

3.2.4. Peak Friction Angle of Coral Sand

Intact saturated coral reef rock cores from each stratum were collected, and the peak friction angle ϕ_i was determined by triaxial shear tests. The interface friction angle δ_i between concrete and coral reef rock for each corresponding stratum was determined

using the interface direct shear test method described above. Regression analysis was then performed to establish the regional relationship between ϕ_i and δ_i as follows Equation (10):

$$\delta_i = b \times \phi_i \quad (10)$$

where b is the adhesion-friction ratio, generally taken as 0.7–0.85 ϕ_i is the peak friction angle, and δ_i is the interface friction angle. The experimental results are shown in Table 2, to simplify the experimental procedures in practical engineering applications, b is set to 0.75 based on sampling data from a specific location in the South China Sea.

Table 2. Results of coral reef-concrete interface shear tests.

Test Specimen	f_i	δ_i	b
1	35	26.5	0.757
2	38	28.5	0.750
3	40	30.0	0.750
4	42	31.5	0.750
5	45	33.8	0.751
6	48	36.0	0.750
7	50	37.5	0.750
8	52	39.0	0.750
9	55	41.3	0.751
10	58	43.5	0.750

It should be noted that the data listed in Tables 1 and 2 are group-averaged calibration data obtained from specimens belonging to the same reef facies and tested under similar preparation, saturation, and loading conditions. They should not be interpreted as a broad raw database covering different coral reef formations. The relatively strong linear trend is mainly related to the limited lithological range of the tested specimens, the use of averaged values for each specimen group, and the purpose of establishing regional strength-conversion relationships for engineering calculation.

To avoid overinterpretation, the revised tables report the number of parallel specimens, mean values, standard deviations, and coefficients of variation. The corresponding regression plots also include the original test points, error bars, and regression residuals. Therefore, the linear relationships derived from Tables 1 and 2 should be regarded as first-order regional calibration relationships within the tested strength range, rather than universal strength laws for all coral reef rocks. When the reef facies, pore structure, cementation degree, saturation condition, grout material, curing condition, or specimen scale differs significantly from the present study, the proportional coefficients should be recalibrated using local UCS, triaxial, and interface shear test data.

4. Calculation Model for the End Bearing Capacity and Shaft Resistance of Cast-in-Place Piles in Coral Reef Formations

4.1. Load Decomposition of the End Bearing Capacity of Cast-in-Place Piles

Coral reef rock is characterized by well-developed pores, weak cementation, and strong heterogeneity. During the construction of cast-in-place piles, grout penetrates the pore-fracture system and hardens, forming a cementation-enhanced ring near the pile base and along the borehole wall. As a result, failure at the pile base changes from the conventional single crushing mode in a homogeneous bearing stratum to a composite failure mode involving the intact rock bearing zone, the cementation-enhanced zone, and the interface transition zone. Based on this mechanism, the ultimate state of end bearing is jointly controlled by three contributions. The first is the bearing contribution of the rock

mass, which is provided by the shear strength of the intact coral reef rock. The second is the bearing contribution of the cemented interface, which is provided by the shear resistance of the concrete–coral reef rock cemented interface near the pile base. The third is the effective stress term, through which the vertical effective stress at the pile base increases the normal confinement and enhances the bearing capacity.

4.2. Calculation Model for the End Bearing Capacity of Cast-in-Place Piles

Based on the analysis of the end bearing mechanism of cast-in-place piles in coral reef geology presented in Section 4.1, it can be seen that the ultimate state of pile end bearing capacity is not controlled by a single factor, but is jointly governed by the strength of the intact coral reef rock mass, the cementation-enhanced zone formed during casting, and the effective stress condition at the pile base. To transform the above load-bearing mechanism into a calculation model applicable to engineering design, it is necessary to express each bearing contribution in a rational mathematical form and superimpose them within a unified framework.

The bearing capacity provided by the intact coral reef rock without considering the influence of grout can be expressed as the bearing contribution of the rock mass as follows Equation (11):

$$q_r = \alpha_r f_r \quad (11)$$

where f_r is the shear strength of the coral reef rock mass, and α_r is the strength reduction coefficient of the rock mass, which accounts for the difference between the shear strength measured from intact laboratory core specimens and the actual shear strength mobilized by the in situ coral reef rock mass beneath the pile base. This difference is mainly associated with joints, pore-scale defects, weakly cemented zones, scale effects, and construction disturbance. In this study, $\alpha_r = 0.6$ was obtained by back-analysis of six instrumented test piles at the investigated site. Therefore, α_r should be regarded as a site-calibrated parameter rather than a universal constant.

Unlike conventional end bearing models, this component of resistance does not originate from the rock mass itself but is provided by the shear capacity of the cemented interface. The bearing contribution of the cemented interface can be expressed as Equation (12):

$$q_i = \alpha_t f_t \left(\frac{2r_c}{D} \right) \quad (12)$$

where f_t is the shear strength of the grout–coral cemented interface, α_t is the interface strength reduction coefficient, r_c is the effective grout penetration radius, and D is the design pile diameter.

The effective stress term is particularly relevant under saturated coral reef rock strata conditions. Acting together with the above two components, it determines the overall bearing capacity at the pile base. The effective stress term can be expressed as Equation (13):

$$q_\sigma = \sigma'_{vb} \quad (13)$$

where σ'_{vb} is the vertical effective stress at the pile base.

The vertical effective-stress term in Equation (13) is introduced as a first-order engineering approximation of the confinement effect at the pile base. It should not be interpreted as an independent material-resistance component parallel to the intact-rock resistance and the cemented-interface resistance. Strictly, according to the Mohr–Coulomb criterion, the vertical effective stress contributes to pile-end resistance by increasing the normal stress acting on the potential failure surfaces and thereby enhancing the mobilized shear strength. In the present formulation, this confinement contribution is separated from the cohesion-

controlled strength terms and represented as an equivalent correction for the stress state within the pile-base failure zone. Because the internal friction angles of the intact coral reef rock and the cemented zone are within a relatively similar range under the investigated conditions, the test results in Table 2 provide limited support for this first-order approximation. Nevertheless, Equation (13) should be regarded as a simplified engineering decomposition rather than a rigorous solution of the coupled three-dimensional failure process.

By combining the bearing contribution of the intact coral reef rock mass, the bearing contribution of the cemented interface, and the vertical effective stress at the pile base, the end bearing capacity of the cast-in-place pile can be expressed as Equation (14):

$$Q_b = A_p(\alpha_r f_r + 2\alpha_t f_t r_c / D + \sigma'_{vb}) \quad (14)$$

Equation (14) represents a first-order semi-empirical decomposition of the pile-end bearing capacity based on deformation compatibility among the intact coral reef rock, the grout-cemented zone, and the pile base. The present closed-form formulation does not explicitly resolve the stress redistribution at the boundary between the intact rock and the cemented zone, the progressive mobilization and possible overlap of the failure surfaces, or the coupling between effective-stress confinement and cementation-induced cohesion. These interactions may influence the actual pile-base failure process and are not assumed to be physically absent. However, within the calibrated range of the present study and under conditions of small deformation and compatible mobilization, their effects are assumed to be secondary relative to the three principal resistance contributions. Therefore, the linear summation in Equation (14) should be interpreted as a first-order semi-empirical engineering approximation rather than as a strictly uncoupled mechanical solution or a rigorous theoretical boundary solution.

It should be noted that the present experimental results provide indirect rather than direct evidence for the assumed load-transfer role of the cementation-enhanced zone. Field coring and CT imaging confirm the existence and spatial extent of the grout-penetrated zone, while the shear tests demonstrate that the grouted coral reef composite possesses measurable shear resistance. However, these tests mainly support parameter determination and do not independently measure the load carried by the cemented zone during pile loading. Therefore, the comparison with the field static load test should be interpreted as a preliminary consistency check rather than a direct validation of the proposed mechanism. Direct mechanism-level verification requires instrumented pile tests, such as strain gauges, distributed fiber-optic sensing, or segmented testing elements, together with coupled numerical analysis of the pile–cemented zone–coral reef rock system.

4.3. Calculation Model for Shaft Resistance of Cast-in-Place Piles

4.3.1. Interface Friction Resistance

Interface friction resistance refers to the frictional resistance generated between the pile concrete and the coral reef rock along the borehole wall under effective normal confinement, and it is a typical frictional component of shaft resistance. Within a layered calculation framework, the interface friction resistance in the i -th stratum can be written as Equation (15):

$$q_{f,i} = K_i \sigma'_{vb} \tan \delta_i \quad (15)$$

where σ'_{vb} is the vertical effective stress level around the pile in the i -th stratum, K_i is the lateral earth pressure coefficient, and δ_i is the interface friction angle between concrete and coral reef rock.

4.3.2. Additional Cementation-Induced Friction Resistance

The additional cementation-induced friction resistance is a distinctive component of cast-in-place piles in coral reef formations, and it is often the key factor causing the measured shaft resistance to exceed that predicted by conventional friction-based calculations. Its physical essence lies in the hardening of grout after penetrating the pores along the borehole wall, thereby forming a composite system consisting of the pile concrete and the cementation-enhanced ring near the pile shaft. Accordingly, the additional cementation-induced friction resistance in the i -th stratum can be shown as Equation (16):

$$q_{c,i} = \xi \tau_i \quad (16)$$

where τ_i is the grout-coral cementation-related shear strength, and ξ is the grout interlocking area coefficient, given by $\xi = \frac{2r_c}{D} \times (1 + \frac{r_c}{D})$.

Equation (15) follows the conventional Coulomb-type interface-friction formulation and is retained as the baseline shaft-resistance component. The extension of the present model lies in the cementation-induced friction term in Equation (16). When grout penetration is absent or negligible, and the proposed model naturally reduces to the standard interface-friction formulation, which provides a consistent limiting case.

Based on the above decomposition, the total shaft resistance intensity per unit side area in the i -th stratum can be expressed as Equation (17):

$$q_{s,i} = K_i \sigma_{vi} \tan \delta_i + \xi \tau_i \quad (17)$$

The linear superposition in Equation (17) is based on the geometric distinction and compatible mobilization of the two shaft-resistance components. The conventional interface-friction component is mobilized on the original pile–coral reef rock contact surface; whereas, the cementation-induced component is mobilized on the grout-filled pore surfaces and within the cementation-enhanced ring. These two components therefore act over distinct effective shear-transfer areas. Since the cemented ring is bonded to the pile concrete, compatible shear displacement between the pile and the cemented ring can be assumed at small strains and near the peak mobilization stage of shaft resistance. Under this condition, the two components can be regarded as parallel shear-transfer paths and summed as a first-order engineering approximation. This treatment does not imply complete independence during progressive failure; possible interaction effects, including non-uniform grout penetration, local debonding, cracking of the cemented ring, and interface softening, are considered indirectly through, interface strength parameters, and site-calibrated reduction factors.

4.3.3. Layered Shaft Resistance Formula and Summation of Total Shaft Resistance

Because the porosity, strength parameters, and effective stress level of coral reef formations vary significantly with depth, the shaft resistance along the pile shaft exhibits a distinctly non-uniform distribution. Therefore, to ensure that the calculation results can reflect the actual stratigraphic conditions, a layered calculation framework is adopted in this study. The ground surrounding the pile is divided into n calculation layers according to the site investigation profile. The thickness of the i -th layer is l_i and the corresponding parameters, including the interface friction angle, lateral earth pressure coefficient, cementation-related shear strength, and effective grout penetration radius, are all taken as the values for that layer.

Based on the above decomposition, the total shaft resistance per unit cross-sectional area for the i -th layer can be expressed as Equation (18):

$$q_{s,i} = K_i \sigma'_{vb} \tan \delta_i + \zeta \tau_i \quad (18)$$

The side area of the pile can be expressed as Equation (19):

$$A_{s,i} = \pi D l_i \quad (19)$$

The shaft resistance of the i -th stratum can be expressed as Equation (20):

$$Q_{s,i} = q_{s,i} A_{s,i} = (K_i \sigma'_{vb} \tan \delta_i + \zeta \tau_i) \pi D l_i \quad (20)$$

By summing the shaft resistance contributions of all strata along the pile length, the total shaft resistance is defined as Equation (21):

$$Q_s = \sum_{i=1}^n Q_{s,i} = \sum_{i=1}^n [(K_i \sigma'_{vb} \tan \delta_i + \zeta \tau_i) \pi D l_i] \quad (21)$$

5. Preliminary Engineering Case Assessment

5.1. Engineering Case Study Verification of End Bearing Capacity

It should be noted that the following engineering case is used as a preliminary case assessment rather than as an independent validation of the predictive capability of the proposed model. Several parameters used in the calculation, including the effective grout penetration radius, interface strength parameters, shear-to-compression ratios, grout interlocking coefficient, and reduction coefficients, were obtained from local field tests, laboratory tests, or back-analysis under the investigated site conditions. Therefore, the comparison between the calculated and measured values should be interpreted as a consistency check for the proposed load-decomposition framework within this specific project, rather than as statistical evidence of general model predictability. The project is in a reef area in the South China Sea, as shown in Figure 6. The site geology consists primarily of undisturbed coral reef rock and coral sand deposits. Drilled cast-in-place piles were selected as the foundation type to accommodate construction requirements under the complex geological conditions of the reef.

For this case study, core samples were collected from typical coral reef rock at the pile tips within the project area. Laboratory tests were conducted to determine their physical properties, providing baseline data for subsequent pile tip bearing capacity calculations. Basic physical parameters of the test samples, including core dimensions, apparent volume, dry mass, and saturated mass, were obtained through on-site sampling and laboratory testing. Specific on-site data are shown in Table 3.

To verify the applicability of the three-component superposition mechanism at the pile tip in cast-in-place pile projects in coral reef rock formations, an analysis of typical engineering case studies was conducted. First, the porosity must be determined using the “combined three-step method.” According to Equation (1), $n_1 = 34.7\%$ and $n_2 = 38.2\%$. Since the result from Equation (2) is $\varepsilon = 10.1\% > 5\%$, and $n_3 = 36.6\%$ was obtained via CT scanning, the calculated value of n according to Equation (3) is 35%. The porosity results are shown in Table 4.

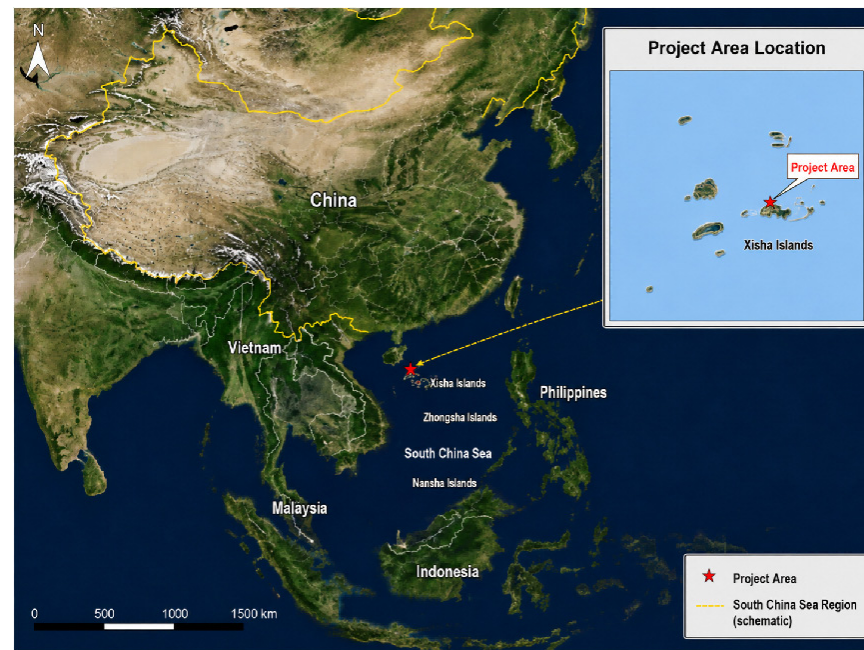


Figure 6. Map of the project site.

Table 3. Summary of construction site conditions.

Parameters	Numerical Value
Core diameter D	75 mm
Core height L	150 mm
Apparent volume V	662.3 cm ³
Dry mass m_d	985.6 g
Saturation mass m_w	1215.3 g
Water mass m	229.7 g
Volume of water V	229.7 cm ³

Table 4. Porosity determination results.

n_1	n_2	n_3	n
34.7%	38.2%	36.6%	35%

The effective grout penetration radius r_c can be calculated using the regional empirical Equation (5).

$$r_c = 0.1 + 0.05n = 0.1 + 0.5 \times 0.35 = 0.1175 \text{ m} \quad (22)$$

According to Equation (22), the effective grout penetration radius was calculated as 0.118 m and was adopted in the subsequent analysis. Based on the unconfined compressive strength test, q_u was measured to be 10 MPa.

The rock mass strength parameters were calculated based on Equation (6) and Equation (7) together with the results in Table 1. Based on the slope results in Figure 5, fitting yields $\omega_1 = 0.25$ and $\omega_2 = 0.3$, and the calculations of the rock mass, strength parameters are given as follows:

$$f_r = \omega_1 q_u = 0.25 q_u = 0.25 \times 10 = 2.5 \text{ MPa} \quad (23)$$

$$f_t = \omega_2 q_u = 0.3 q_u = 0.3 \times 10 = 3.0 \text{ MPa} \quad (24)$$

The calculation of the rock mass bearing capacity is based on Equation (11), where $f_r = 2.5$ MPa according to Equation (23).

The bearing capacity of the rock mass itself is calculated using Equation (11).

$$q_r = \alpha_r f_r = 0.6 \times 2.5 = 1.5 \quad (25)$$

In this case, $f_r = 2.5$ MPa; based on the inversion of six sets of field test piles, the measured value of α_r was 0.6.

In this calculation f_r was determined as 3.0 MPa from Equation (24), based on the combined calibration of eight sets of interface shear and test pile data, the interface strength reduction factor α_t is determined to be 0.8. Substituting the above parameters into Equation (12), the bearing contribution of the cemented interface can be calculated as follows:

$$q_i = \alpha_t f_t \left(\frac{2r_c}{D} \right) = 0.8 \times 3.0 \times \left(\frac{2 \times 0.118}{1.0} \right) = 0.5664 \quad (26)$$

To calculate the effective stress at the tip of the cast-in-place pile in the coral reef strata, the distribution of soil layers at the site was determined based on the results of the field investigation, as shown in Table 5. From top to bottom, the site is divided into three layers: loose coral sand, moderately dense coral sand, and coral reef limestone, with thicknesses of 3.0 m, 5.0 m, and 7.0 m, respectively. The corresponding natural unit weights are 18.0, 19.0, and 21.0 kN/m³, respectively; the saturated unit weights are 19.5, 20.5, and 22.0 kN/m³; and the submerged unit weights are 9.5, 10.5, and 12.0 kN/m³.

Table 5. Soil layers at the construction site.

Layer Number	Soil Layer	Soil Layer Thickness (m)	Natural Unit Weight γ (kN/m ³)	Saturation Unit Weight γ_{sat} (kN/m ³)	Submerged Unit Weight γ' (kN/m ³)
1	Loose coral sand	3.0	18.0	19.5	9.5
2	Medium-density coral sand	5.0	19.0	20.5	10.5
3	Coral reef limestone	7.0	21.0	22.0	12.0

Table 5 presents the thickness and unit weight parameters of the three layers of coral reef strata at the project site, from top to bottom, providing the basic data for calculating the effective stress at the pile tip. Based on these soil parameters, the total vertical stress at different depths was calculated using a stratified weighting method. By combining this with pore water pressure, the effective stress distribution at each stratum was determined. The calculation results are shown in Table 6.

Table 6. Effective stress calculation table.

Depth (m)	Soil Layer	Vertical Stress σ_v (kPa)	Pore Water Pressure u (kPa)	Effective Stress σ'_{vb} (kPa)
0–2	Loose coral sand (waterfront)	36.0	0.0	36.0
2–3	Loose coral sand (underwater)	55.5	10.0	45.5
3–8	Medium-density coral sand	158.0	60.0	98.0
8–15	Coral reef limestone	312.0	130.0	182.0

The vertical effective stress at the pile base σ'_{vb} is obtained as 182.0 kPa. Based on Equation (25) and Equation (26) and the adopted values of the key parameters, and by

substituting them into Equation (14), the end bearing capacity under coral reef geological conditions can be calculated as follows:

$$Q_b = A_p(\alpha_r f_r + 2\alpha_t f_t r_c / D + \sigma'_{vb}) \\ = 0.785 \times (0.6 \times 2.5 + \frac{2 \times 0.8 \times 3.0 \times 0.118}{1.0} + 0.182) = 1.76 \text{ MN} \quad (27)$$

Therefore, the end bearing capacity is obtained as $Q_b = 1.76 \text{ MN}$. To conduct a preliminary consistency check for the proposed calculation framework, the calculated pile end bearing capacity was compared with the measured value obtained from the field static load test at the same pile base depth. The error between the theoretical value and the measured value was calculated as follows, where the field measured value is $Q_{b'} = 1.72 \text{ MN}$.

It should be noted that this comparison constitutes a site-calibrated engineering consistency check rather than an independent validation. The pile diameter, pile length, layer thickness, porosity, unconfined compressive strength, interface shear strength, and effective grout penetration radius were obtained directly from field investigations, coring observations, CT imaging, and laboratory tests. In contrast, the rock-mass strength reduction coefficient, interface reduction coefficient, and several regional empirical coefficients were calibrated using data from the same project. Therefore, the agreement between the calculated and measured results indicates only that the proposed framework can reasonably reproduce the bearing response of the investigated project under site-calibrated conditions. It should not be interpreted as evidence of independent predictive capability across different projects or geological conditions.

$$\varepsilon = \frac{|Q_b - Q_{b'}|}{Q_{b'}} = \frac{|1.76 - 1.72|}{1.72} \times 100\% = 3\% \quad (28)$$

The above calculation shows that the difference between the calculated and measured values is approximately 3%. The difference between the calculated and measured pile-base resistances is approximately 3%. This agreement indicates that the proposed framework can reproduce the overall pile-base response of the investigated project under site-calibrated conditions. Future studies will focus on expanding the validation data by including independent engineering cases from different island-reef sites. Multi-site field static load tests, grouted core observations, CT-based grout penetration analyses, and interface shear tests will be conducted to evaluate the transferability and stability of the proposed coefficients. On this basis, the empirical relationships may be further refined into generalized prediction equations with explicit applicability ranges and statistical confidence intervals.

5.2. Engineering Case Study of Shaft Resistance

The engineering case study of shaft resistance was based on the same cast-in-place pile project located on an island reef as that used for the end bearing capacity case study. The site strata are mainly composed of intact coral reef rock and overlying coral sand deposits. Cast-in-place piles were adopted as the foundation type to satisfy the bearing and construction requirements of island-reef engineering under complex geological conditions. Based on the field geotechnical investigation data and laboratory test results, the geometric parameters and the physical and mechanical properties of the strata required for pile design were selected. Among them, the strength parameters of the coral reef rock were obtained from unconfined compressive strength tests, and the peak friction angle was determined by triaxial shear tests. These parameters were used as the basic input conditions for the subsequent calculation and model verification of pile bearing capacity, and their specific values are listed in Table 7.

Table 7. Input parameters for shaft resistance calculation.

Material Parameters	Pile Diameter D (m)	Unconfined Compressive Strength q_u (kPa)	Peak Friction Angle f_i
Numerical value	1.0	600	42°

The rock-mass-related parameters were calculated based on Equation (9) and Equation (10). The values of a and b were determined from the test data in Tables 1 and 2, respectively, yielding $a = 0.3$ and $b = 0.75$. The calculation process is shown below. The rock mass-related parameters are calculated as follows:

$$\tau_i = a \times q_{ui} = 0.3q_{ui} = 0.3 \times 600 = 180 \text{ kPa} \quad (29)$$

$$\delta_i = b \times f_i = 0.75f_i = 0.75 \times 42 = 31.5^\circ \quad (30)$$

The calculation of shaft resistance depends on the determination of several key parameters. Based on the calculation results for end bearing capacity, the effective radius at the pile base was taken as $r_c = 0.118$ m. According to the stress distribution in the strata, the vertical effective stress at the midpoint of each layer was adopted for the layered calculation, as listed in Table 8. In combination with the lateral pressure characteristics of the coral reef ground, the lateral earth pressure coefficient K_i was taken as 0.8. By substituting the above values of r_c , σ'_{vb} , and K_i into Equation (8), $\xi = 0.236$ was obtained. Considering the in situ soil parameters together with the above value of ξ , and substituting them into Equation (17), the shaft resistance under coral reef geological conditions was calculated. The detailed calculation results are presented in the following Table 8.

Table 8. Layered calculation of shaft resistance parameters.

Layer Number	Soil Layer	Soil Layer Thickness l_i (m)	Midpoint of a Layer σ'_{vb} (kPa)	K_i	δ_i ($^\circ$)	τ_i (kPa)	Shaft Resistance q_{si} (kPa)
1	Loose coral sand	3.0	35.8	0.8	26.3	105	38.93
2	Medium-density coral sand	5.0	36.8	0.8	28.5	150	51.38
3	Coral reef limestone	7.0	45.0	0.8	31.5	180	64.54

The total shaft resistance was then calculated using the layered summation formula in Equation (21).

$$Q_s = \sum_{i=1}^n [(K_i \sigma'_{vb} \tan \delta_i + \xi \tau_i) \pi D l_i] \quad (31)$$

$$= \pi \times 1.0 \times (3.0 \times 38.93 + 5.0 \times 51.38 + 7.0 \times 61.54) = 2.59 \text{ MN}$$

The calculated total shaft resistance was 2.59 MN, while the measured value was 2.63 MN, giving a relative error of approximately 1.5%. By comparing the calculated result with the measured value, the calculation error can be determined using the following formula:

$$\varepsilon_1 = \frac{|Q_s - Q_{s'}|}{Q_{s'}} = \frac{|2.59 - 2.63|}{2.63} \times 100\% = 1.5\% \quad (32)$$

The calculated total shaft resistance was 2.59 MN; whereas, the measured value was 2.63 MN, corresponding to a relative difference of approximately 1.5%. This agreement

indicates that the proposed formulation can reproduce the overall shaft-resistance response of the investigated pile under site-calibrated conditions.

Nevertheless, the field test does not independently distinguish the conventional interface-friction resistance from the cementation-induced additional resistance. Therefore, the agreement in total shaft resistance does not directly validate the two-component decomposition or the calculated contribution assigned to each component. The result should be regarded as an overall-response consistency check only.

5.3. Sensitivity and Analysis of Empirical Coefficients

Several coefficients used in the calculation were obtained from regional regression, local laboratory tests, field back-analysis, or engineering experience. To quantify the influence of these coefficients on the calculated pile-end bearing capacity and shaft resistance, a local sensitivity and interval-based uncertainty analysis was conducted.

To avoid ambiguity among the repeated symbols a and b used in different equations, the penetration-radius coefficients in Equation (5) are denoted in this subsection as a and b , respectively:

$$r_c = a + bn \quad (33)$$

The shear-to-compression ratio in Equation (9) is denoted as a_s :

$$\tau_i = a_s q_{ui} \quad (34)$$

and the interface-friction ratio in Equation (10) is denoted as b_δ :

$$\delta_i = b_\delta \varphi_i \quad (35)$$

These revised symbols are introduced only for the sensitivity analysis and do not represent additional model parameters. The calculated values of the investigated engineering case, namely and $Q_b = 1.76$ MN and $Q_s = 2.59$ MN, were adopted as the reference results. A one-factor-at-a-time method was used, in which each empirical coefficient was varied individually while all measured geometric, stress, and stratigraphic parameters were maintained at their reference values.

For coefficients without statistically established confidence intervals, a local perturbation range of $\pm 20\%$ around the reference value was adopted. This range is used as an engineering sensitivity scenario rather than as a statistical confidence interval. The dimensionless local sensitivity index is defined as

$$S_j = \frac{(Y_j^+ - Y_j^-) / (2Y_0)}{(c_j^+ - c_j^-) / (2c_{j0})} \quad (36)$$

where Y_0 is the reference calculated resistance, Y_j^+ and Y_j^- are the calculated resistances corresponding to the upper and lower values of coefficient c_j , respectively, and c_{j0} is its reference value. For a symmetric $\pm 20\%$ perturbation, the denominator in Equation (36) is equal to 0.20.

5.3.1. Sensitivity of Pile-End Bearing Capacity

Substituting the empirical relationships for intact-rock strength, cemented-interface strength, and effective penetration radius into Equation (14), the pile-end bearing capacity may be expressed as:

$$Q_b = A_b \left[\alpha_r \omega_1 q_u + 2\alpha_c \omega_2 q_u \frac{a + bn}{D} + \sigma'_{vb} \right] \quad (37)$$

where α_r and α_c are the strength reduction coefficients of the intact rock mass and cemented interface, respectively; ω_1 and ω_2 are the corresponding shear-to-compression ratios; and a and b are the regional penetration-radius regression coefficients. The sensitivity results are presented in Table 9.

Table 9. Sensitivity of the empirical coefficients affecting pile-end bearing capacity.

Coefficient	Reference Value	Perturbation Range	Calculated Range (MN)	Relative Change in Q_b (MN)	Sensitivity Index S_j
α_r	0.60	0.48–0.72	1.528–1.999	13.36%	0.668
ω_1	0.25	0.20–0.30	1.528–1.999	13.36%	0.668
α_c	0.80	0.64–0.96	1.675–1.852	5.02%	0.251
ω_2	0.30	0.24–0.36	1.675–1.852	5.02%	0.251
a	0.10	0.08–0.12	1.688–1.838	4.27%	0.214
b	0.05	0.04–0.06	1.750–1.776	0.75%	0.037

The results indicate that the pile-end bearing capacity is most sensitive to the intact-rock reduction coefficient α_r and the intact-rock shear-to-compression ratio ω_1 . A variation of $\pm 20\%$ in either coefficient results in a variation of approximately $\pm 13.36\%$ in Q_b . This is because the intact-rock contribution constitutes the largest component of the calculated pile-end resistance in the investigated case.

The interface reduction coefficient α_c and the cemented-interface shear-to-compression ratio ω_2 have a secondary influence, with sensitivity indices of 0.251. The penetration-radius offset coefficient a has a moderate effect; whereas, the porosity weighting coefficient b has a relatively limited effect under the reference porosity of $n = 0.35$.

5.3.2. Sensitivity of Shaft Resistance

For the shaft-resistance calculation, Equation (21) can be rewritten by introducing the regional empirical relationships as

$$Q_s = \sum_{i=1}^n [K_i \sigma'_{vi} \tan(b_\delta \varphi_i) + \xi_i a_s q_{ui}] \pi D l_i \quad (38)$$

where K_i is the regional shear-to-compression ratio of the grouted coral reef composite, b_δ is the interface-friction ratio, and K_i is the lateral earth-pressure coefficient.

For consistency with the engineering calculation in Section 5.2, the simplified interlocking-area coefficient actually used in the case calculation was adopted:

$$\xi_i = \frac{2r_c}{D} = \frac{2(a + bn)}{D} \quad (39)$$

The corresponding sensitivity results are presented in Table 10.

Table 10. Local sensitivity of the empirical coefficients affecting shaft resistance.

Coefficient	Reference Value	Perturbation Range	Calculated Range (MN)	Relative Change in Q_s (MN)	Sensitivity Index S_j
a_s	0.30	0.24–0.36	2.243–2.929	13.27%	0.664
a	0.10	0.08–0.12	2.294–2.878	11.30%	0.565
b_δ	0.75	0.60–0.90	2.387–2.811	7.69%	0.409
K_i	0.80	0.64–0.96	2.412–2.760	6.73%	0.336
b	0.05	0.04–0.06	2.535–2.637	1.98%	0.099

The cementation-related shear-to-compression ratio a_s is the most influential empirical coefficient for shaft resistance. A variation of $\pm 20\%$ in a_s changes the calculated shaft resistance by approximately $\pm 13.27\%$. The penetration-radius offset coefficient a is the second most influential parameter because it directly controls the effective grout penetration radius and, consequently, the additional cemented shear-transfer area.

The interface-friction ratio b_δ exhibits a nonlinear influence because it is included within the tangent function. Increasing b_δ by 20% results in an 8.68% increase in Q_s ; whereas, decreasing it by 20% results in a 7.69% reduction. The lateral earth-pressure coefficient K_i has a moderate influence, while the porosity weighting coefficient b has the smallest influence under the investigated conditions.

6. Conclusions

This study investigated the influence of grout penetration and cementation on the end-bearing and shaft-resistance behavior of cast-in-place piles in highly porous coral reef formations. The principal conclusions are as follows.

(1) Field coring, tracer observations, CT imaging, and mechanical tests consistently indicate that grout can penetrate the connected pore–fracture network of coral reef rock and form a cementation-enhanced zone with measurable shear resistance. This zone provides an additional load-transfer path beyond the conventional resistance of the intact rock mass and the original pile–rock interface. In the investigated project, the cementation-related contribution accounted for approximately 25% of the calculated pile-end resistance and approximately 15% of the calculated shaft resistance, indicating that neglecting grout-induced cementation may lead to an incomplete evaluation of pile capacity in highly porous coral reef formations.

(2) The extent of grout-induced enhancement is governed not only by total porosity but also by pore connectivity, pore-throat geometry, grout rheology, grouting pressure, and water–cement ratio. Under the relatively controlled grouting conditions of the investigated site, the measured porosity of approximately 35% corresponded to an effective grout penetration radius of 0.118 m. This result supports the use of porosity as a practical regional indicator of grout penetrability; however, porosity should not be regarded as the sole governing variable, and the associated empirical relationships should be interpreted as first-order regional approximations.

(3) The three-component end-bearing model and the two-component shaft-resistance model provide a mechanically interpretable first-order decomposition of the principal resistance contributions. The intact-rock and cementation-related components correspond to distinguishable load-transfer regions, while the effective-stress term represents an equivalent confinement contribution rather than an independent material-resistance mechanism. The proposed superposition assumes compatible deformation and does not explicitly resolve stress redistribution, progressive failure, local debonding, or nonlinear interaction between the intact rock and the cemented zone. Therefore, the formulation should be regarded as a semi-empirical engineering approximation rather than a fully coupled analytical solution.

(4) Under the site-calibrated conditions of the investigated project, the calculated pile-end resistance was 1.76 MN, compared with the measured value of 1.72 MN, while the calculated and measured shaft resistances were 2.59 MN and 2.63 MN, respectively. These comparisons demonstrate preliminary engineering consistency between the proposed framework and the measured response of this project. Because several reduction factors and regional empirical coefficients were calibrated using data from the same project area, these results do not constitute independent validation of the model's predictive capability.

The present assessment is limited to a single engineering case, and several regional coefficients and reduction factors were calibrated using field measurements, laboratory tests, or back-analysis data from the same project area. The proposed equations should therefore be regarded as a site-calibrated semi-empirical framework for preliminary engineering assessment under conditions comparable to those investigated in this study. Additional independent field validation is necessary before the framework can be recommended for broader engineering application. Future studies should include multiple projects covering different reef facies, pore-connectivity conditions, grout materials, water–cement ratios, grouting pressures, pile dimensions, and construction procedures. Instrumented pile-load tests, comparative grouted and ungrouted piles, distributed fiber-optic sensing, field coring, CT-based grout-penetration measurements, and coupled three-dimensional numerical analyses should be combined to evaluate the transferability of the empirical coefficients and directly investigate the load-transfer behavior of the individual resistance components.

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