

Article

Calibration Chamber Test of CPT Penetration Based on Marine Sand with Parameter Interpretation Models

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Abstract

This study investigates marine sand collected from the southeastern coast of China through laboratory calibration chamber model tests under varying relative densities and consolidation stresses. The consolidation characteristics and cone penetration test (CPT) penetration response of soil specimens were examined, and interpretation models relating CPT parameters to soil unit weight, relative density, and shear wave velocity were established. Results show that shear wave velocity increases with relative density, with consolidation exerting a stronger enhancement. Lateral earth pressure exhibits a pronounced distance attenuation effect, with stress differences most prominent near-field and diminishing with distance. Cone tip resistance increases with both relative density and consolidation stress, with consolidation stress exerting a more significant influence on low-density specimens; sleeve friction increases linearly with relative density. The interpretation models achieve good correlations with unit weight ($R^2 = 0.78$), normalized cone tip resistance with the square of relative density ($R^2 = 0.72$), and shear wave velocity ($R^2 = 0.85$), and field validation confirms higher prediction accuracy than conventional empirical formulas for terrigenous sands. The models enable rapid, cost-effective parameter estimation from routine CPT data, though they remain site-specific, being based on nine chamber tests and validated against six field layers from a single site.

Keywords: calibration chamber test; marine sand; cone penetration test (CPT); shear wave velocity; parameter interpretation

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1. Introduction

The cone penetration test (CPT) is an advanced and widely used in situ investigation technique that provides high-resolution soil mechanical and hydrogeological profiles by penetrating a cone at a constant rate and continuously recording parameters such as cone tip resistance and sleeve friction. Compared with conventional drilling methods, CPT offers advantages including minimal soil disturbance, fast testing speed, continuous data acquisition, and simple, portable equipment [1–3]. It has become a reliable tool for soil

classification and parameter interpretation, making it particularly suitable for offshore and onshore engineering applications [4–8].

Accurately and quickly obtaining key parameters of soil during site investigation, especially marine sand, through in situ CPTs remains a technical challenge. Extensive efforts have been devoted to establishing empirical correlations between CPT parameters and soil properties using calibration chambers [9,10]. More recently, Eslami et al. [11] conducted calibration chamber tests on Babolsar coastal sand and developed correlations linking CPT parameters to relative density, elastic modulus, and internal friction angle, achieving coefficients of determination exceeding 94%. For chemically treated soils, Yu et al. [12] employed mini-calibration chambers with piezocone tests (CPTU) to evaluate the physical property changes in MICP-treated calcareous sand, proposing calibrated relationships between CPTU measurements and key physical parameters. For fine-grained soils, Basarah et al. [13] interpreted CPTU data from Western Java soft clays to assess undrained shear strength and the cone bearing factor. For cohesionless soils, Duan [14] demonstrated that incorporating the state parameter into seismic CPT interpretations significantly improves the prediction of the maximum shear modulus. Collectively, these studies confirm that calibration chamber testing, particularly when combined with multi-parameter monitoring, provides a robust basis for linking CPT measurements to soil properties.

The relationship between CPT parameters and V_s has also attracted considerable interest, as V_s is essential for evaluating soil dynamic response, liquefaction susceptibility, and seismic site classification [15–18]. Traditional empirical correlations derived from global or regional databases often exhibit bias when applied to specific sites due to site uniqueness in geological history and stress conditions. To address this, Meng and Pei [19] developed a quasi-site-specific CPTU- V_s transformation model that combines a generic database with local measurements, and demonstrated that a hierarchical Bayesian model incorporating multiple CPTU parameters and soil-type-specific normalization yields superior accuracy and robustness. Tonni and Simonini [20] interpreted CPTU and V_s data from the Treporti test site in Venice to update existing correlations for silty sediments and validated their findings against field stiffness measurements from an embankment test. These studies underscore that site-specific calibration is essential for reliable CPT- V_s correlations, and that generic empirical models should be applied with caution.

Despite these advances, two significant research gaps remain. First, most calibration chamber studies have focused on general sandy or clayey soils, with systematic investigations of natural marine sand being notably scarce, given the fundamental differences existing in depositional environment, particle morphology, and mineralogical composition between marine and terrigenous sands [21,22]. Second, existing interpretation models are predominantly site-specific, lack independent field validation, and rarely establish a unified framework for simultaneously interpreting unit weight, relative density, and shear wave velocity. As these three core parameters govern the in situ stress state, shear strength, and seismic performance of marine sand, their acquisition in marine environments is hindered by sample disturbance and high testing costs. These gaps highlight the need for a comprehensive calibration chamber study that establishes multi-parameter interpretation models and rigorously validates them with field data.

To address these gaps, this study conducts nine calibration chamber model tests on Jiangsu marine sand, with simultaneous monitoring of earth pressure, pore water pressure, and shear wave velocity. The CPT penetration response is systematically examined, and unified interpretation models relating CPT parameters to unit weight, relative density, and shear wave velocity are established. The novelty of this work lies in two aspects: (i) it provides comprehensive CPT calibration chamber dataset for marine sand from

southeastern China, and (ii) it establishes a unified multi-parameter interpretation framework and validates the proposed models against independent field data. This study aims to provide a robust technical reference for marine site investigation and engineering practice.

2. Methodology of Calibration Chamber Test

2.1. Test Soil

The test soil samples were collected from the southeast coast of Jiangsu Province, China. The physical properties of the tested marine sand are summarized in Table 1. The particle size distribution curve is shown in Figure 1. It indicates that the soil specimen consists of approximately 72% sand (0.075–4.75 mm) and 28% fines (<0.075 mm), with silt content (18%) higher than clay content (10%). According to the USCS, it is classified as silty sand (SM).

Table 1. Physical indices of test soil.

ρ_{dmax}	ρ_{dmin}	e_{max}	e_{min}	G_s
1.82	1.22	1.20	0.48	2.69

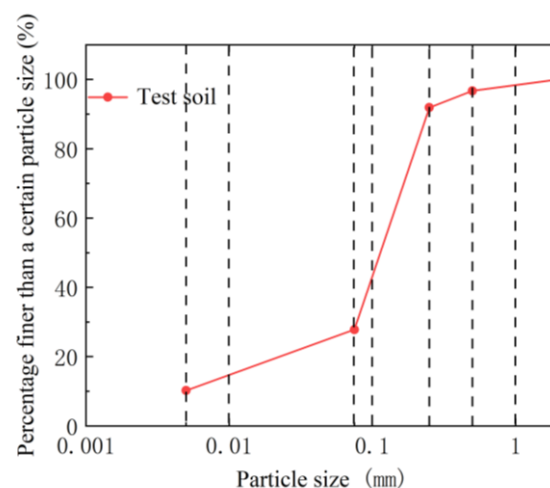


Figure 1. Particle size distribution of test soil.

2.2. Experimental Setup

A calibration chamber test system was developed to simulate CPT penetration in marine sand under different overburden pressures. The calibration chamber's main body has an inner height of 70 cm and an inner diameter of 75 cm, with a 30 cm clearance from the ground (Figure 2a). The chamber boundary effect is a critical consideration in calibration chamber testing [23,24]. The calibration chamber system (Figure 2b) adopted in this study features a BC3-type boundary condition, with a constant stress applied at the bottom boundary during penetration, while the lateral boundary remains fixed (zero displacement). In general, the larger the chamber diameter relative to the cone size, the closer the measured CPT response approximates field conditions. According to Salgado et al. [23], boundary effects can be considered negligible when the chamber-to-probe diameter ratio (D/d) ranges between 25 and 120. In this study, $D/d = 750/25 = 30$, which falls well within this acceptable range and satisfies the test requirements. A detailed assessment of the boundary effects is provided in Section 3.2.

To replicate the overburden stress conditions at different burial depths, the system adopted a bottom-mounted 20-ton hydraulic jack for vertical loading. The top cover was

equipped with a controllable opening and reserved ports, enabling rapid CPT penetration after the soil consolidation was completed. A motor installed at the top of the device controls the penetration rate, which was set to 2 cm/s in the test. Meanwhile, to meet the requirements of saturated consolidation, water inlet and outlet channels were provided at the lower and upper sections of the chamber, respectively. The inlet was located 10 cm above the bottom, and the outlet is located 20 cm below the top.

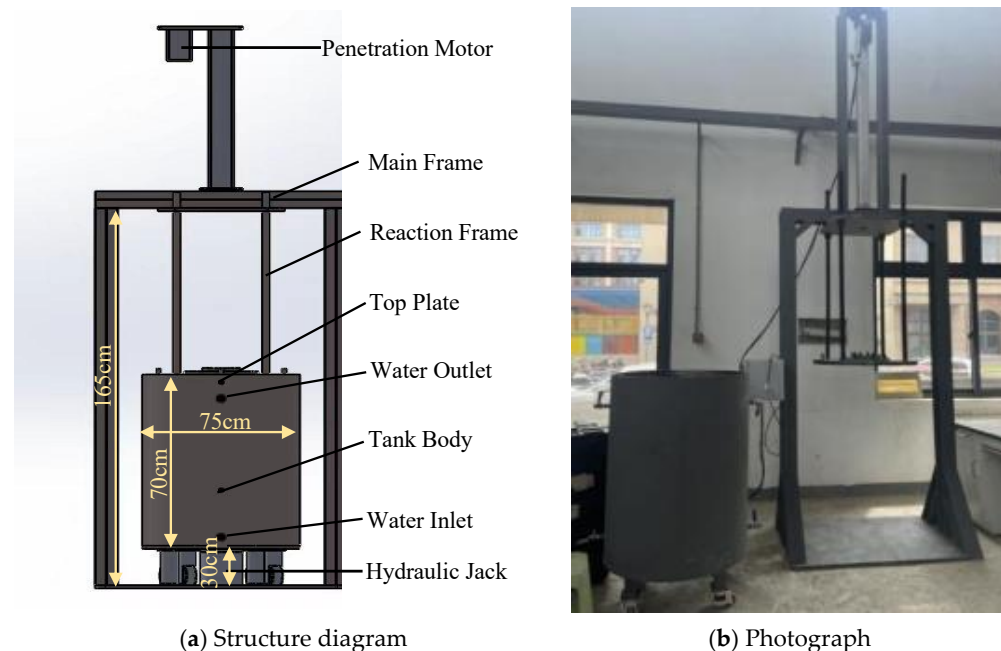


Figure 2. Schematic diagram of calibration chamber test system.

2.3. Sensor Arrangement

This test system can simultaneously measure the CPT parameters and shear wave velocity of soil specimens under different overburden pressures. The main geometric parameters of the adopted CPT probe are as follows: a cone angle of 60° , a base diameter of 25 mm, a projected cone base area of 10 cm^2 , and a side friction-sleeve surface area of 150 cm^2 . The shear wave velocity of the soil sample was measured using bender element sensors. The transmitter and receiver ends of the bender elements were fixed by a custom-designed aluminum alloy frame with adjustable spacing, which ensured precise alignment of the two ends and facilitated accurate determination of the shear wave propagation distance.

Meanwhile, earth pressure sensors and pore water pressure sensors were pre-embedded inside the soil specimen to monitor the CPT penetration characteristics and identify the consolidation state of the soil. Specifically, earth pressure sensors (S1, S2, S3) were arranged radially at a height of 50 cm below the top of the soil specimen along the radial direction of the calibration chamber to measure the lateral earth pressure induced by CPT penetration. Pore water pressure sensors (U1, U2, U3) were embedded at depths of 10 cm, 30 cm, and 50 cm from the top of the soil specimen, respectively, to judge the completion of soil consolidation.

2.4. Test Procedures

Three initial relative densities were designed in the tests, namely 30%, 50%, and 70%, corresponding to loose, medium-dense, and dense sand states, respectively. Three levels of vertical consolidation stresses (100 kPa, 200 kPa, 300 kPa) were applied, leading to a total of nine test cases. The detailed test procedures are described as follows:

(1) Specimen preparation. Soil specimens were prepared by the under-compaction layered method with a designed sand-filling height of 60 cm. The mass of sand for each layer was back-calculated according to the target relative density. Sand pluviation was adopted to uniformly fill the calibration chamber layer-by-layer up to the designed height, and sensor arrangement was completed in accordance with Figure 3 during the sample preparation process.

(2) Saturation and consolidation. After specimen preparation, the initial shear wave velocity V_{s1} was measured. Water was injected from the water inlet at the lower part of the calibration chamber until steady bubble-free outflow appeared at the upper outlet. The specimen saturation was achieved by continuous water injection, with the completion of saturation judged by the criterion that no excess pore pressure was generated and no observable settlement of the soil specimen occurred over the 48 h injection period. The limitation of the saturation criterion is discussed in Section 4.5.

Subsequently, the designed vertical stress was applied to the specimen, while pore water pressure and surface settlement were monitored synchronously. Consolidation was regarded as completed when the excess pore water pressure fully dissipated and the specimen deformation became stable, after which the post-consolidation shear wave velocity V_{s2} was measured.

(3) CPT penetration. The CPT probe was penetrated continuously at a constant rate of 2 cm/s until reaching the bottom of the calibration chamber. The data from earth pressure sensors were recorded throughout the entire penetration process.

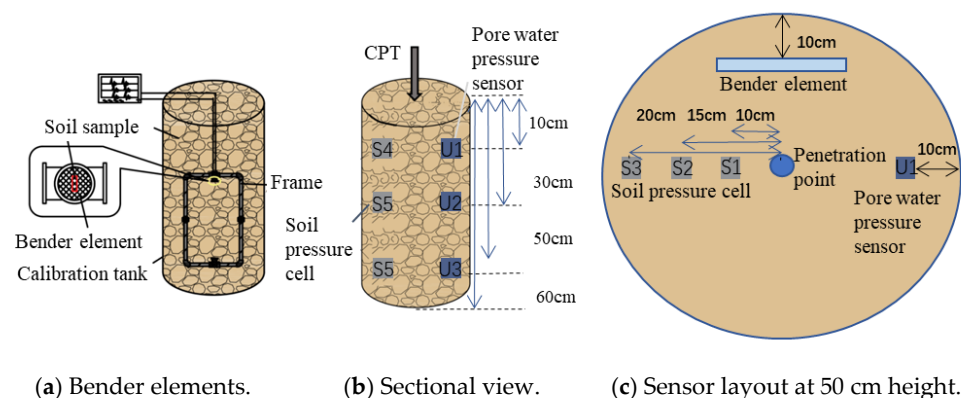


Figure 3. Sensor layout scheme.

3. Test Results

3.1. Consolidation Characteristics of Soil Specimens

The first arrival time method was adopted to determine the shear wave velocity of soil specimens. The shear wave is excited by the transmitter of the bender element, and the first obvious jump point of the received waveform is defined as the first arrival time of the shear wave. The shear wave velocity is calculated as the effective propagation distance divided by the first arrival travel time. During the test, stable and reliable waveforms were acquired by optimizing the excitation frequency, pulse width, and signal repetition times [25].

Figure 4 presents the shear wave velocity test curve under the condition of an initial relative density of 50% and a consolidation pressure of 100 kPa. The received waveform is clear and complete. The shear wave velocity can be accurately determined by selecting the time difference corresponding to the first peak values of the transmitted and received signals.

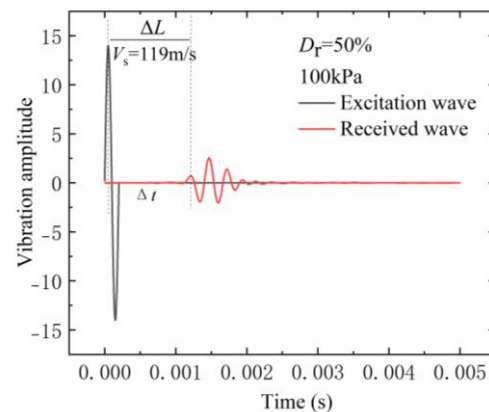


Figure 4. Shear wave velocity measurement of soil specimen ($D_r = 50\%$, consolidation pressure = 100 kPa).

Figure 5 illustrates the distribution characteristics of shear wave velocity before and after consolidation for all nine test specimens with different initial relative densities and consolidation pressures. Figure 5a shows the initial shear wave velocity after specimen preparation. For specimens with relative densities of 30%, 50%, and 70%, the average shear wave velocities are 104.7 m/s, 114.3 m/s, and 131.0 m/s, with corresponding coefficients of variation of 0.039, 0.044, and 0.035, respectively. The results indicate that the shear wave velocity increases significantly with the rise in initial relative density. The small coefficient of variation for each group demonstrates that the adopted test apparatus and sample preparation method can produce sand specimens with good uniformity and stable quality.

Figure 5b shows the shear wave velocity after the completion of consolidation. A comparison of the measured data before and after consolidation reveals that consolidation markedly increases the shear wave velocity. Taking the consolidation pressure of 200 kPa as an example, for specimens with relative densities of 30%, 50%, and 70%, the shear wave velocities are 109 m/s, 119 m/s, and 126 m/s before consolidation, and increase to 160 m/s, 170 m/s, and 183 m/s after consolidation, respectively. It is verified that consolidation compaction has a prominent enhancement effect on shear wave velocity, and the influence magnitude of relative density is weaker than that of consolidation.

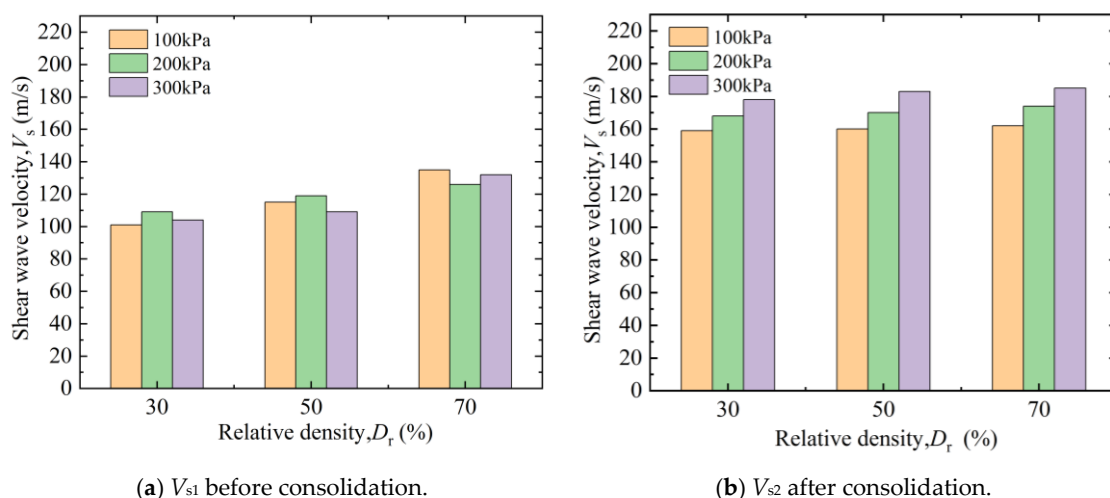


Figure 5. Comparison of shear wave velocity of soil specimens.

Figure 6 shows the surface displacement evolution curves of soil specimens during consolidation. For all specimens, the consolidation displacement increases rapidly at first and then gradually stabilizes, which is highly consistent with the drainage consolidation mechanism of sandy soil. At the early consolidation stage, pore water pressure dissipates rapidly, soil voids are compressed and closed, and surface settlement increases sharply. With the extension of consolidation time, pore water pressure dissipates completely, and soil deformation tends to be stable.

Under the same vertical consolidation stress, specimens with a lower initial relative density ($D_r = 30\%$) exhibit larger final consolidation displacement, while those with a higher initial relative density ($D_r = 70\%$) show smaller displacement. The underlying mechanism is that loosely packed sand at low relative density possesses a large void volume and sufficient compressible space during consolidation. In contrast, sand at high relative density has dense particle contacts and a stable void structure, resulting in a relatively limited consolidation compression magnitude.

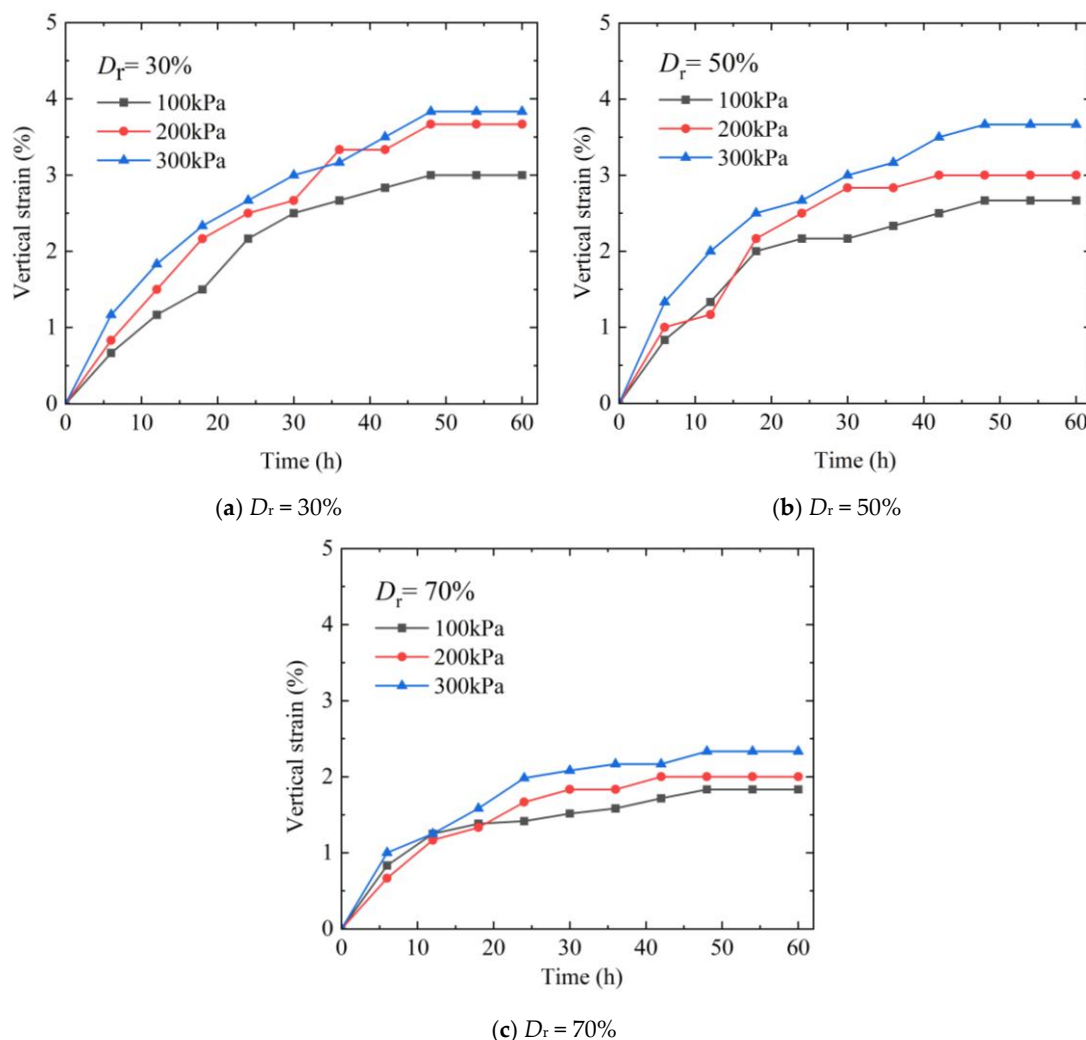


Figure 6. Vertical surface displacement variation in soil specimens during consolidation.

3.2. CPT Penetration Characteristics and Test Results

To investigate the mechanical behavior and stress transfer mechanism of marine sand during CPT penetration, lateral earth pressures induced by CPT penetration for the specimens with an initial D_r of 50% and 70% and three vertical consolidation stresses were analyzed, as shown in Figure 7. The lateral earth pressure response during penetration is

governed by horizontal distance from the penetration center, relative density, and vertical consolidation stress. Under the same vertical consolidation stress, specimens with higher initial relative density exhibit greater lateral earth pressure at the same position, and all six groups show a consistent trend of significant lateral earth pressure attenuation with increasing horizontal distance, indicating a pronounced distance attenuation effect of probe penetration on the surrounding soil. Under the same initial relative density, higher vertical consolidation stress leads to higher lateral earth pressure at the same horizontal distance, attributed to increased post-consolidation relative density and heightened sensitivity to penetration-induced additional stress. Furthermore, the earth pressure differences caused by variations in relative density and consolidation stress are most pronounced in the near-field region of the penetration center, and gradually diminish with increasing horizontal distance.

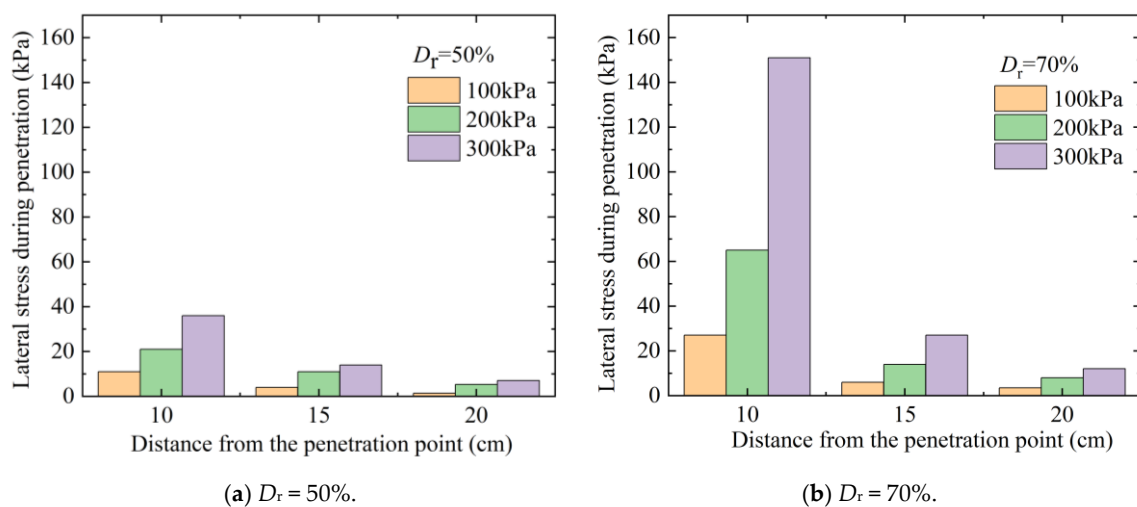


Figure 7. Horizontal stress-response diagrams during penetration.

Based on the measured lateral additional stresses induced by CPT penetration (Figure 7), the stress attenuation rate (SAR) is defined as the percentage ratio of the lateral earth pressure at a given point to the maximum lateral additional stress at the same depth; the corresponding contour map along the radial direction is presented in Figure 8. The radius at which SAR reduces to zero is defined as the CPT penetration influence radius, while the near-field and far-field regions are distinguished by SAR thresholds of 50%.

Quantitative analysis reveals that at a relative density of 70% and a consolidation stress of 300 kPa, the influence radius is approximately 30 cm, confirming that the chamber dimensions adequately eliminate boundary effects. The most pronounced stress differences induced by variations in relative density and consolidation stress are observed in the near-field region, indicating significant stress concentration and nonlinear soil behavior, whereas these differences gradually diminish in the far-field region, where stress transfer becomes predominantly elastic and penetration effects decay. The additional stresses generated at the cone tip are radially transmitted outward through particle contacts, and the stress level is governed jointly by the initial relative density and the effective consolidation stress. This explains why higher relative density and consolidation stress lead to larger stress magnitudes and wider influence radii.

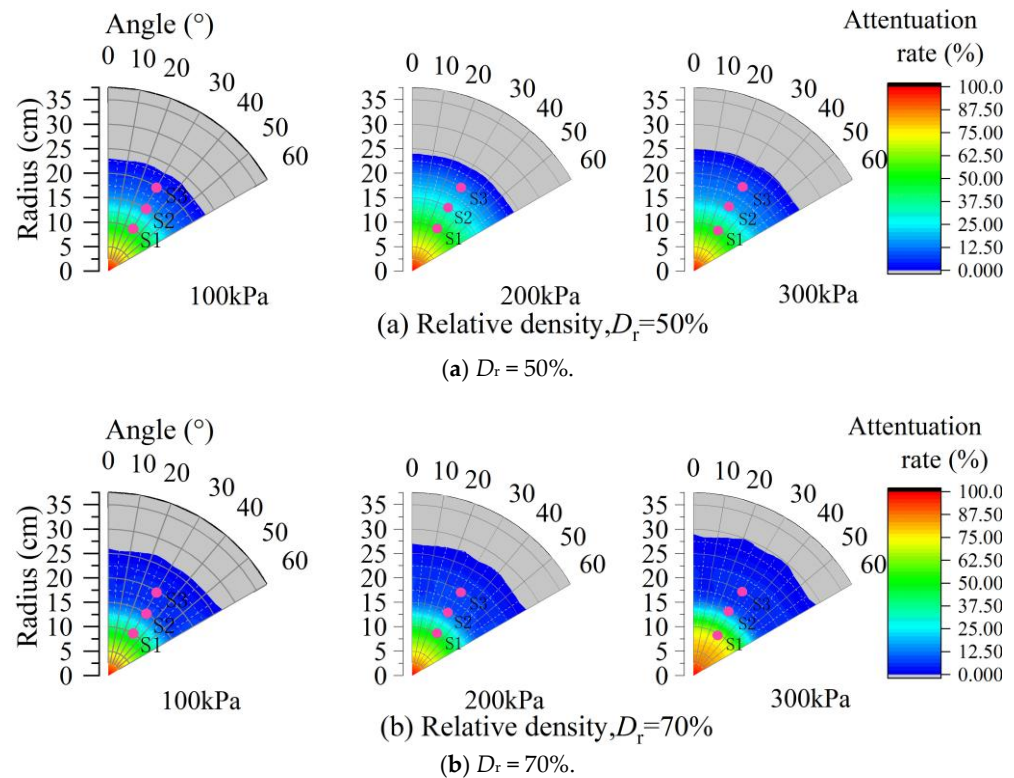


Figure 8. Contour plots of stress attenuation rate (S1, S2 and S3 are soil pressure sensors in Figure 3).

Figure 9 presents the variations in cone tip resistance (q_c) and sleeve friction (f_s) with penetration depth from CPTs on marine sand under different initial relative densities. It can be observed that the f_s is considerably lower than the q_c .

As shown in the cone tip resistance curves in Figure 9a,c,e, q_c clearly increases with the rise in initial relative density. For a given relative density, q_c also increases with the increase in consolidation stress. The influence of consolidation stress is more pronounced for loose specimens ($D_r = 30\%$), while the incremental effect is relatively weaker for dense specimens ($D_r = 70\%$), indicating that the cone tip resistance is highly sensitive to relative density.

According to the sleeve friction results in Figure 9b,d,f, f_s increases approximately linearly with increasing relative density. Meanwhile, the influence of consolidation stress on f_s gradually weakens as the relative density rises. This demonstrates that sleeve friction is sensitive to relative density but less sensitive to consolidation stress.

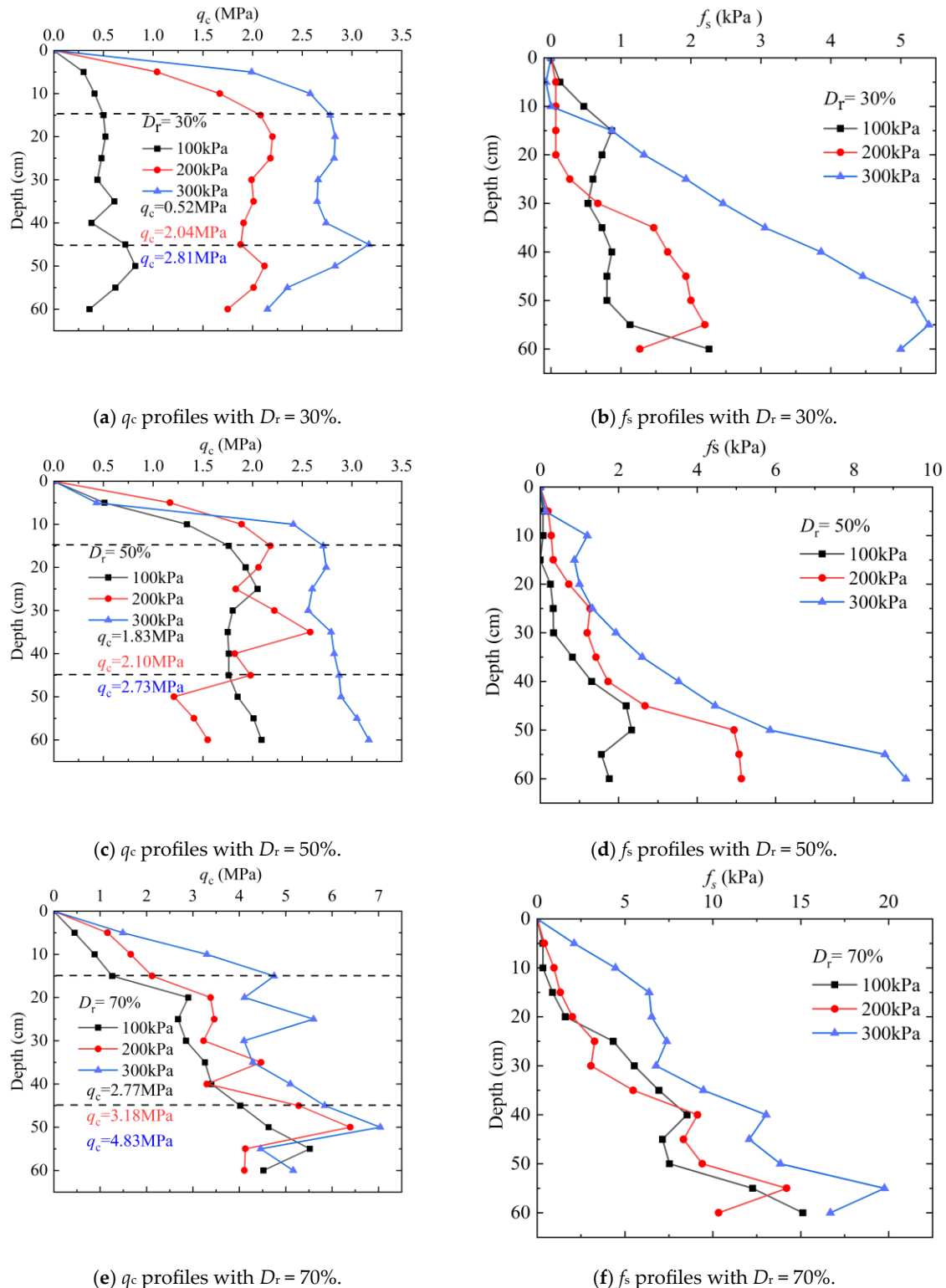


Figure 9. Results of CPT.

4. Interpretation of CPT Parameters

4.1. Unit Weight

Unit weight (γ) is a fundamental physical state parameter of soil. Conventionally, undisturbed soil samples are collected by fixed-piston thin-walled samplers or open thin-walled samplers. The unit weight is calculated by measuring the mass of a soil specimen with a known volume. Previous studies have shown that cone penetration test (CPT) parameters can be adopted to rapidly estimate soil unit weight.

Two interpretation models were adopted for comparison in this study. One is the Robertson model [1] established from field measurements (Equation (1)), where R_f is the friction ratio and defined as $(f_s/q_c) \times 100\%$; γ_w is the unit weight of water; and p_a is the atmospheric pressure. The other is the Duan model [14] established from CPT calibration chamber tests, as listed in Equation (2).

$$\gamma = \gamma_w [1.236 + 0.36 \lg(q_c / p_a) + 0.27 \lg(R_f)] \quad (1)$$

$$\gamma = 17.8 + 1.77 \lg(q_c) \quad (2)$$

In this study, the correlation between CPT cone tip resistance (in MPa) and unit weight (in KN/m³) is established in Figure 10. The fitted curve derived from this study shows a strong positive correlation with a coefficient of determination (R^2) of 0.78, and almost all test points fall within the 95% confidence band, which proves the reliability of the established relationship. The corresponding empirical formula is expressed as:

$$\gamma = 18.617 + 0.7472 \lg(q_c) \quad (3)$$

The three interpretation models share consistent overall trends, but systematic deviations exist among them. This discrepancy may arise because the two reference models were developed for general terrigenous sands, overlooking the unique particle composition and depositional characteristics of coastal marine sand. Further study is warranted to refine the model coefficients and to extend the calibration to a broader range of marine sand types with varying fines content and mineralogy.

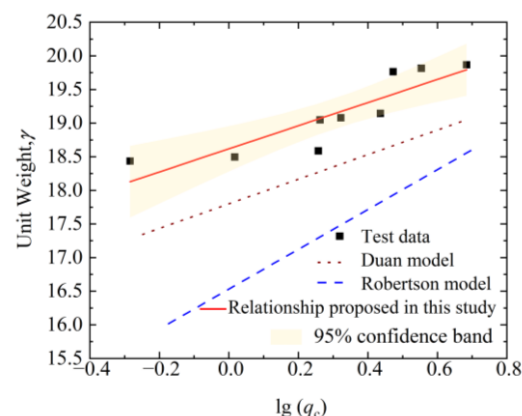


Figure 10. Relationship between unit weight and CPT parameters.

4.2. Relative Density

Relative density (D_r) is one of the most widely used state parameters in geotechnical engineering. It serves not only as a core index for sand liquefaction evaluation and site risk assessment, but also as a critical indicator to characterize the densification degree, shear strength, and compaction behavior of sandy soils. It plays an irreplaceable role in quality control in liquefiable ground engineering.

Direct in situ measurement of D_r requires obtaining the in situ void ratio, maximum and minimum void ratios, and corresponding density parameters. Such field procedures are complicated, significantly affected by soil disturbance, and accompanied by considerable data scatter, which cannot meet the requirements of rapid and accurate engineering evaluation. Accordingly, indirect interpretation methods based on in situ test parameters are commonly adopted in engineering practice to estimate D_r .

Similar to the approach used for unit weight, the relative density interpretation models proposed by Robertson [1] and Duan [14] were adopted for comparison. Equation (4)

and Equation (5) present the Robertson model and the Duan model, respectively, where Q_{tln} is the normalized cone penetration resistance corrected for overburden stress [26]. It should be noted that the square of relative density (D_r^2) is commonly employed in establishing such correlations to improve linearity.

$$D_r^2 = Q_{tln} / 350. \quad (4)$$

$$D_r^2 = -0.0636 + 0.0074(Q_{tln}). \quad (5)$$

$$Q_{tln} = \left(\frac{q_c}{p_a}\right) \left(\frac{p_a}{\sigma'_{v0}}\right)^n, \quad (6)$$

where the units of the cone tip resistance q_c and initial vertical effective stress σ'_{v0} in Equation (6) are in kPa; p_a is a reference pressure = 100 kPa; and the exponent n is typically equal to 0.5.

Figure 11 presents the fitting relationships between Q_{tln} and D_r^2 for marine sand, together with the Robertson and Duan models for comparison. A satisfactory linear relationship is observed, with a coefficient of determination of $R^2 = 0.72$. Both reference models—derived from field measurements and calibration chamber tests on general terrigenous sands—tend to overestimate the relative density of the marine silty sand investigated in this study. The proposed interpretation model for relative density is given in Equation (6).

$$D_r^2 = -0.0352 + 0.00228(Q_{tln}). \quad (7)$$

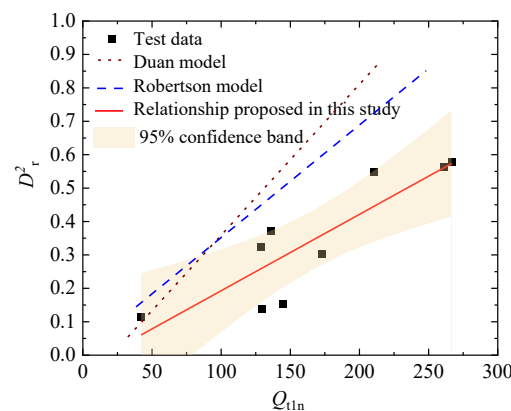


Figure 11. Relationship between relative density and CPT parameters.

4.3. Shear Wave Velocity

Shear wave velocity (V_s) is a core parameter in geotechnical dynamics and engineering site investigation. It can reflect the in situ density, stiffness, soil structure, and stress state of soils in an undisturbed manner. V_s is widely applied in site classification, characteristic period determination, seismic design and seismic response analysis, as well as in the calculation of dynamic parameters such as the small-strain shear modulus. Moreover, it can be used to evaluate the liquefaction potential of sand and silt, and to assess the in situ reinforcement effect of ground improvement.

Previous studies have established various empirical correlations between q_c and V_s for sandy soils, with the most common forms including linear, polynomial, logarithmic, and piecewise fitting models [27]. Based on the calibration chamber test data obtained in this study, a linear relationship between V_s (in m/s) and q_c (in MPa) is proposed in Figure

12, which exhibits a strong correlation with a coefficient of determination of $R^2 = 0.85$. The regression expression is given as follows:

$$V_s = 154.54 + 6.45(q_c) \quad (8)$$

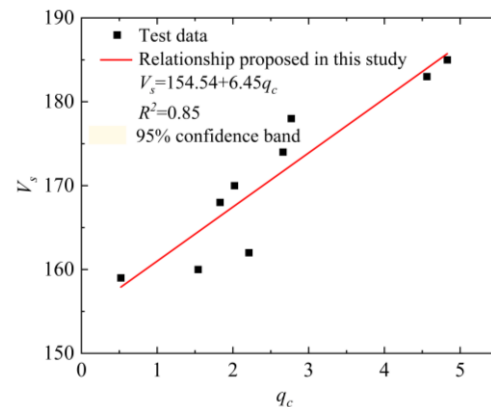


Figure 12. Relationship between shear wave velocity and cone tip resistance.

4.4. Validation of the Proposed Interpretation Models

To validate the accuracy of the proposed interpretation models, field CPT measurements along with corresponding unit weight and relative density data were collected from the sampling site and compared against the model predictions. It is noted that shear wave velocity data is not available at the site. Figure 13 presents two q_c profiles from the boreholes, where the silty sand layers correspond to the calibration chamber test material. A total of six silty sand layers were identified across the two boreholes and used for validation of the proposed unit weight and relative density interpretation models. The average q_c values within each of the six layers were employed for interpretation, and three models were applied for comparison: the proposed model (M1), the Duan model (M2), and the Robertson model (M3). The calculation results are summarized in Table 2, where the relative error is defined as the absolute difference between the predicted and measured values divided by the measured value.

As shown in Table 2, the proposed model (M1) exhibits the best overall performance in predicting relative density, with relative errors ranging from -0.52% to 0.19% and a mean relative error (MRE) of only -0.243% , which is significantly lower than those of M2 (MRE = 2.007%) and M3 (MRE = 0.427%). Notably, M2 yields considerably larger errors for layers three and four, indicating that the Duan model may introduce systematic bias when applied to the marine sand deposits in this study area. In terms of unit weight prediction, all three models achieve relative errors within $\pm 2\%$, demonstrating that CPT parameters exhibit a generally good correlation with unit weight. The MRE for M1, M2, and M3 are -0.095% , 0.209% , and -0.133% , respectively. However, the relative error of M3 is approximately -0.13% for all layers, which may limit its capability to capture the actual variability of soil properties. In contrast, M1 provides errors comparable to M2 while maintaining reasonable sensitivity to layer-specific variations. Overall, the validation results confirm that the proposed interpretation models offer reliable and accurate predictions for marine sand.

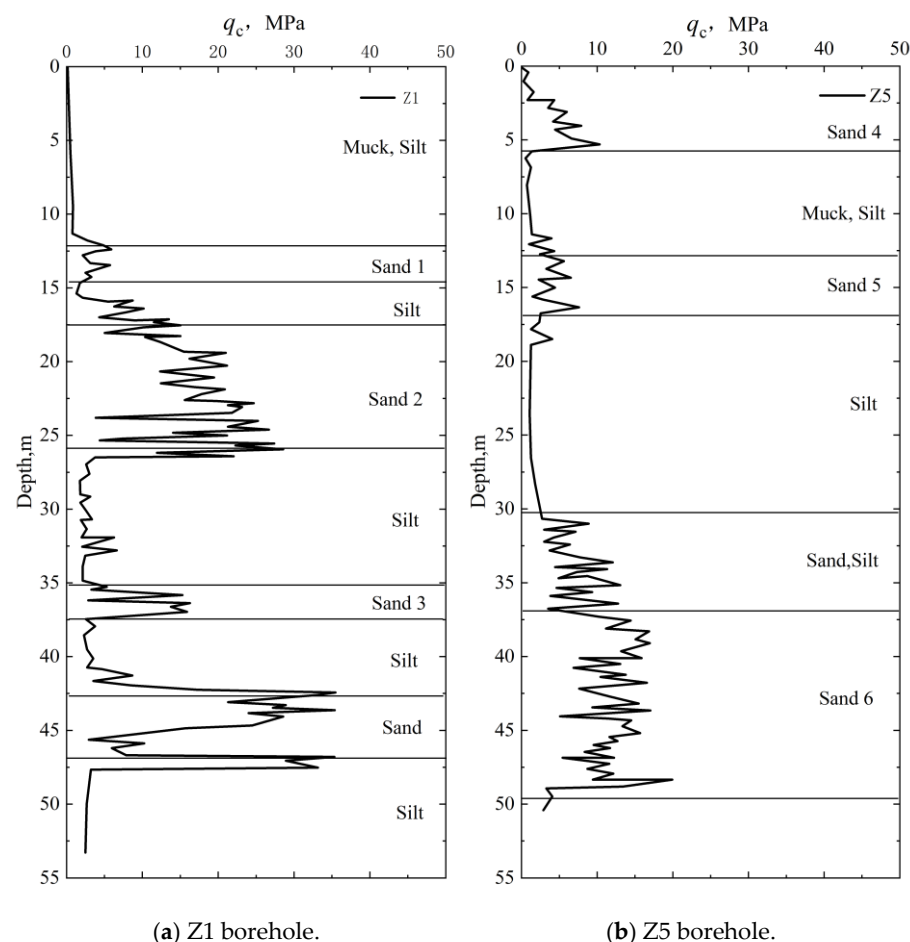


Figure 13. q_c profiles at sampling site.

Table 2. Validation of the interpretation models for unit weight and relative density.

Silty Sand Layer No.	Depth (m)	q_c (MPa)	σ'_{vo} (kPa)	Q_{tin}	γ (kN/m ³)	D_r (%)	Relative Error of γ (%)			Relative Error of D_r^2 (%)		
							M1	M2	M3	M1	M2	M3
1	13.0	3.6	136.5	30.8	19.0	25	0.17	−1.13	−0.13	−0.44	1.63	0.41
2	26.5	18.7	260.6	115.8	19.8	50	−1.18	1.27	−0.13	−0.08	2.17	0.32
3	36.8	11.6	355.2	61.5	19.6	35	−0.96	0.43	−0.13	−0.14	2.20	0.44
4	5.7	4.0	69.4	48.0	19.0	25	0.35	−0.71	−0.13	0.19	3.67	1.19
5	15.3	3.8	157.6	30.3	18.7	25	1.87	0.67	−0.15	−0.46	1.57	0.38
6	48.7	12.5	464.6	58.0	19.6	45	−0.83	0.72	−0.13	−0.52	0.81	−0.18

Note: M1 denotes the proposed model, while M2 and M3 represent the models of Duan and Robertson, respectively.

4.5. Discussion

The proposed interpretation models, particularly for relative density, demonstrate superior performance over existing models. This improved accuracy is attributed to site-specific calibration using marine sand from southeastern China, highlighting the importance of developing region-specific correlations for marine deposits. Nevertheless, several limitations should be acknowledged: the tests were conducted on a single soil type within a limited stress range (100–300 kPa) and relative densities (30–70%), and the models were developed from only nine test cases and validated against six field layers. In addition, owing to the large specimen dimensions and instrumentation constraints, neither Skempton's B -value verification nor V_p -based saturation checks were performed, which may affect the reliability of the CPT and shear wave velocity data.

Future work should rigorously verify the degree of saturation and extend the test matrix to include a wider range of soil types, higher stress levels, and variable drainage conditions, etc. Despite these limitations, the proposed models provide a practical and cost-effective tool for preliminary estimation of unit weight, relative density, and shear wave velocity of marine sand from routine CPT data, particularly for sites along the south-eastern coast of China. For safety-critical projects, site-specific validation is recommended prior to application.

5. Conclusions

This study investigated marine sand collected from the southeastern coast of China through laboratory calibration chamber model tests under varying relative densities and consolidation stresses. Interpretation models relating CPT parameters to soil unit weight, relative density, and shear wave velocity were established. The principal findings are summarized as follows:

- (1) The first arrival wave method combined with bender element sensors enables accurate measurement of shear wave velocity in marine sand. Initial shear wave velocity increases with relative density, while consolidation exerts a stronger enhancing effect than relative density. The coefficients of variation among parallel specimens were all below 0.044, confirming the uniformity and repeatability of the adopted sample preparation method.
- (2) Lateral earth pressure during CPT penetration exhibits a pronounced distance attenuation effect: higher relative density and consolidation stress both lead to greater earth pressure magnitudes, with stress differences most prominent in the near-field region and diminishing with increasing horizontal distance.
- (3) Regarding the CPT penetration characteristics of marine sand, the cone tip resistance increases with both relative density and consolidation stress, with consolidation stress exerting a more significant influence on low-density specimens. The sleeve friction increases linearly with relative density, and its sensitivity to consolidation stress decreases as density increases.
- (4) The interpretation models were established, correlating CPT cone tip resistance with unit weight, relative density, and shear wave velocity. The correlation coefficients for cone tip resistance versus unit weight, normalized cone tip resistance versus the square of relative density, and cone tip resistance versus shear wave velocity reached 0.78, 0.72, and 0.85, respectively. Validation against field test data for unit weight and relative density demonstrated that the proposed models achieve higher prediction accuracy than conventional empirical formulas derived for terrigenous sands.

The proposed interpretation models enable rapid, cost-effective estimation of key marine sand parameters from routine CPT data, and are particularly suited for preliminary site investigations and offshore design where direct sampling is challenging. The experimental framework also provides a technical reference for optimizing large-volume sand specimen preparation and uniformity control. However, these models are based on nine calibration chamber tests and validated against six field layers from a single site, and thus remain site-specific to the investigated marine sand. Further validation with broader datasets encompassing diverse soil types and stress conditions is recommended before broader application.

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