

Article

Shaking Table Test on Response of Prestressed Concrete Hybrid-Reinforced Solid Square Piles in Soft Versus Stiff Clay

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Abstract

Prestressed concrete hybrid-reinforced solid square (PCHSS) piles combine strands and rebars for improved ductility. This study presents large-scale shaking table tests (1:2 scale) on PCHSS pile–superstructure systems embedded in soft and stiff clay, subjected to four ground motions with distinct frequency contents. The experimental program systematically captured the evolution of natural frequencies, damping ratios, dynamic earth pressure distributions, bending moment profiles, curvature ductility demands, and post-test cracking patterns. Results reveal that ground stiffness governs the degradation pathway and energy dissipation mode: soft clay exhibited 37.3% frequency degradation ($1.33\times$ that in stiff clay), while stiff clay showed a 101% damping increase ($>3\times$ soft clay). Long-period ground motions generated maximum curvature ductility demands and peak bending moments 2–3 times those of short-period records under identical PGA, attributed to near-resonance coupling. Pile–soil interaction transitions from compatible deformation to progressive separation with increasing seismic intensity, with gap spacings of 40–53 mm observed in soft clay. Notably, the code-specified 4D reinforcement zone was found insufficient for soft clay foundations, where crack distributions extended to 4D–7D, warranting an extended zone up to 7D. The findings provide experimental benchmarks for numerical model calibration and offer practical guidance for extending hybrid-reinforced precast piles into moderate-to-high-seismicity regions.

Keywords: PCHSS pile; shaking table test; soft and stiff clay ground; soil–structure interaction; dynamic response

1. Introduction

Prestressed high-strength concrete pipe piles (PHC piles) and prestressed high-strength concrete hollow square piles (PHS piles) have been extensively employed in foundation engineering across coastal regions, owing to their high bearing capacity, rapid constructability, reliable quality control, and notable cost-effectiveness [1–4]. However, due to their reduced cross-sectional area and the exclusive use of prestressed steel bars as longitudinal reinforcement, these pile types exhibit limited ductility, rendering them vulnerable to brittle flexural–shear failure under seismic loading, which restricts their application in high-seismicity zones [5–8].

To overcome this inherent limitation, a novel prestressed concrete hybrid-reinforced solid square pile (PCHSS pile) has been developed [5,9–11]. In this hybrid reinforcement



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system, prestressing strands provide high flexural capacity and superior crack-closing ability, ensuring satisfactory serviceability performance, whereas non-prestressed reinforcing bars enhance ductility and energy dissipation through post-yield plastic deformation under large displacements. The synergistic interaction between the two reinforcement types achieves an optimal balance between load-carrying capacity and deformability. To date, extensive experimental and analytical investigations have been conducted on the axial compressive, shear, and flexural behavior of PCHSS piles [5,10,12–15]. The accumulated evidence consistently demonstrates that, due to the hybrid reinforcement and solid cross-section, PCHSS piles outperform conventional PHC and PHS piles in terms of ultimate bearing capacity, displacement ductility coefficient, and hysteretic energy dissipation under static loading conditions [13,15].

Nevertheless, the aforementioned conclusions regarding PCHSS piles are drawn exclusively from static or quasi-static cyclic loading tests, in which the pile is typically treated as an isolated flexural member without considering the surrounding soil. While such loading protocols facilitate the observation of fundamental failure modes, they cannot capture the complex dynamic response characteristics under realistic seismic excitations, including inertial and kinematic interaction, cumulative fatigue damage evolution, and strain-rate-dependent material degradation.

When such piles are embedded in natural soil deposits, however, the soil–pile dynamic interaction profoundly alters the lateral restraint conditions, damping characteristics, and internal force distribution along the pile shaft [16]. A substantial body of research has been devoted to this interaction, which can be broadly categorized into two thematic groups based on ground conditions. The first group focuses on liquefiable sandy soils [17–22]. For example, Tang and Ling [21] investigated the dynamic response of elevated-cap pile foundations during liquefaction, while Hosseini et al. [23] and Nyseany et al. [20] examined the effects of pile group arrangement and bending stiffness on lateral spreading responses. These studies collectively suggest that in liquefied ground, pile failures are predominantly governed by kinematic soil movements rather than inertial forces from the superstructure. The second group concerns non-liquefiable soft clays, where the inertial effects from the superstructure are reported to play a more dominant role [24]. Boulanger et al. [25] developed numerical models for soft clay validated against element tests; Boominathan and Ayothiraman [26] identified pile length and load magnitude as key factors affecting the system frequency; Swagata et al. [27] observed significant period lengthening in soft clay–pile systems with low lateral stiffness. More recently, Zhang et al. [28] and Wang et al. [29] conducted shake-table tests on PHC piles and prestressed piles with hybrid reinforcement (PRHC piles) in clay; their results revealed that the latter exhibited improved energy dissipation and reduced curvature ductility demands. Despite these valuable contributions, two critical deficiencies persist in the existing literature. First, most of these studies have concentrated on conventional circular piles, leaving the dynamic behavior of PCHSS piles largely unexplored. Second, and more importantly, the quantitative impact of pile–soil interaction in clays of varying stiffness on pile internal forces—particularly the peak bending moments and shear forces along the depth—has received limited systematic investigation, despite its paramount significance for engineering design.

A further challenging obstacle lies in the experimental-scale bottleneck. Constrained by shaking table capacity and laboratory space, most existing dynamic tests on pile foundations have employed small-scale models [27,30]. However, because the mechanical behavior of prestressed concrete members is highly sensitive to both the sectional stress level and the ratio of non-prestressed to prestressed reinforcement, small-scale models inevitably involve compromises in material simulation (e.g., using fine aggregates as substitutes) and cannot maintain the prototype prestress level. Consequently, they fail to capture

the complex synergy between prestressing strands and reinforcing bars observed in full-scale prototypes [31]. Consequently, test results derived from such reduced-scale models often lack extrapolation reliability to actual engineering practice, particularly when the hysteretic energy dissipation and damage evolution of hybrid-reinforced members are of primary concern.

A critical examination of the above-mentioned studies reveals that none has simultaneously satisfied the three prerequisites essential for reliably evaluating the seismic performance of hybrid-reinforced piles under realistic soil–pile interaction. These prerequisites are: (a) the model pile must retain the solid square cross-section, the full-scale prestress level, and the non-prestressed reinforcement ratio, so as to preserve the authentic hybrid reinforcement mechanism; (b) the loading protocol must be truly dynamic (shake-table excitation) rather than quasi-static cyclic to capture strain-rate effects and cumulative damage; and (c) the ground conditions must encompass both soft and stiff non-liquefiable clays to allow systematic comparison of soil stiffness effects. To the best of the authors' knowledge, no published experimental investigation has yet addressed all three conditions in a single testing program.

Beyond these experimental limitations, a fourth gap exists at the methodological level. The interpretation and quantification of seismic damage in hybrid-reinforced members have benefited from substantial methodological advances in structural performance assessment. Park and Ang [32] proposed a dual-parameter damage index combining maximum deformation and cumulative hysteretic energy, which has been widely adopted for evaluating damage states in reinforced and prestressed concrete members. More recently, stiffness-based damage indices derived from the degradation of secant stiffness under cyclic loading have been applied to evaluate the progressive damage evolution of prestressed concrete piles in clay [28,29,31]. Energy-based seismic evaluation frameworks quantify structural damage through the ratio of hysteretic energy dissipated to the total energy input. These frameworks have been extended to account for the unique energy dissipation mechanism of hybrid-reinforced systems, in which the non-prestressed reinforcement contributes disproportionately to hysteretic energy dissipation [33]. In the domain of soil–pile interaction, nonlinear Winkler foundation models utilizing dynamic *p-y* curves have been progressively refined to incorporate strain-rate effects, radiation damping, and soil degradation under cyclic loading [34]. Within the performance-based earthquake engineering (PBEE) framework, incremental dynamic analysis (IDA) has been employed to develop fragility curves for pile foundations, enabling probabilistic assessment of pile damage states under varying seismic intensities [35,36]. Additionally, data-driven approaches, including machine learning and surrogate modeling, have recently emerged as efficient tools for rapid seismic response prediction and damage classification of pile foundations [37,38]. Concurrent material-level studies [39,40] on UHPC and UHPFRC have provided valuable methodological references for mechanical performance assessment and failure mechanism analysis under severe loading conditions, which are instructive for interpreting the damage evolution in hybrid-reinforced pile systems. Despite these methodological advances, however, their systematic application to PCHSS piles in clay deposits remains conspicuously absent, because the interaction between hybrid reinforcement mechanics and soil–pile dynamic coupling introduces unique challenges.

To systematically bridge these gaps, this study designed and implemented a large-scale shaking table test program on PCHSS pile–superstructure systems embedded in two representative ground conditions: soft clay and stiff clay. Prior to testing, two explicit scientific hypotheses were formulated based on theoretical considerations of soil–pile interaction: (H1) in soft clay, where the surrounding soil provides low lateral resistance, the PCHSS pile is expected to undergo a flexure-dominated failure, with the plastic hinge zone

forming at a relatively deeper depth due to the downward shift of the maximum bending moment location; (H2) in stiff clay, where the soil offers substantial lateral restraint, the failure mode is anticipated to transition toward flexural–shear cracking localized near the pile head, accompanied by more pronounced strength degradation under high-frequency cyclic loading.

The experimental program systematically acquired comprehensive measurements, including evolution of natural frequencies and damping ratios, dynamic earth pressure distributions around the pile shaft, superstructure acceleration and hysteretic response, and bending moment profiles along the pile depth. Post-test excavation allowed for meticulous mapping of surface crack patterns, which were cross-validated against the internal strain-gauge readings to ascertain the actual damage progression. The outcomes are expected to offer both theoretical insights and practical design aids for engineers seeking to safely extend the application of hybrid-reinforced precast piles into moderate-to-high seismic regions. Furthermore, the experimental dataset generated in this study serves as essential benchmark data for calibration and validation of advanced analytical models, including nonlinear Winkler formulations, PBEE-based fragility assessments, and data-driven predictive frameworks.

2. Experimental Setup and Procedure

Large-scale shaking table tests were conducted on two soil–pile–structure model systems: the soft clay–PCHSS pile–superstructure and the stiff clay–PCHSS pile–superstructure. Figure 1 illustrates the test setup for the shaking table tests on the soft/stiff clay–PCHSS pile–superstructure systems. During the tests, both the clay and the PCHSS pile were placed inside a large laminar shear box. The internal dimensions of the laminar shear box were 2 m × 1.9 m × 1.8 m (length × width × height), consisting of 17 steel shear laminates. Rollers were installed between the laminates to allow relative displacement, thereby simulating the shear deformation between soil layers. To mitigate boundary effects, EVA (Ethylene Vinyl Acetate) foam blocks were attached to the four inner walls of the shear box. A 6-ton mass block was bolted to the top of the model pile to represent the inertia force from the superstructure, as well as to simulate the pile-cap restraint and the superstructure weight. The model pile length was set to 2 m, as detailed in the similitude design (Section 2.1). The pile toe was connected to the bottom of the shear box through an adaptor plate with bolted semi-rigid connections, to simulate the vertical and rotational restraints provided by the deep soil layers. The effectiveness and potential limitations of these boundary conditions are further examined in the Discussion Section (Section 4.1).

Unlike conventional pile foundation configurations, a 0.18 m free segment was reserved between the loading mass and the soil surface in this test. This design not only facilitates observation of soil deformation near the ground surface, but also ensures that the lateral inertial force from the superstructure is applied directly to the pile top, without additional soil restraint on the cap.

The experimental program consisted of four primary phases: (1) similitude design and model pile fabrication, (2) soil preparation and placement, (3) sensor instrumentation, and (4) dynamic testing under selected ground motions. Detailed descriptions are provided in the following subsections.



Figure 1. Shaking table test setup.

2.1. Similitude Design

Similitude relationships are essential for ensuring that model tests can replicate the dynamic response of the prototype [41]. Shaking table tests of soil–structure systems involve multiple materials, rendering strict similitude difficult to achieve. Unlike previous small-scale tests [27], this study adopted a relatively large model size to better capture the true response of PCHSS piles. The same steel and concrete materials as those used in prototype piles were employed, along with identical cross-sectional reinforcement detailing.

The model dimensions were constrained by two practical factors: (1) the shaking table payload and the laminar shear soil container height limited the pile length to 2 m; and (2) the precast manufacturing facility required a minimum cross-section of 200 mm to effectively apply the prestressing force. Consequently, a unified geometric scale factor could not be maintained across all dimensions [30]. The following design principles were therefore established to achieve dynamic equivalence under these distorted similitude conditions:

First, in the superstructure design, the inertial force developed at the model pile head was required to match that of the prototype. To this end, the superstructure was represented by a 6-ton surcharge mass. Although the stiffness and mass of this block did not fully comply with similitude requirements, the lateral inertial force was equivalently reproduced by adjusting the mass distribution while keeping the exposed pile height constant to reflect typical practice. This ensured dynamic equivalence at the pile head.

Second, to ensure that the post-cracking and post-yield dynamic response of the model pile was similar to that of the prototype, the flexural and shear capacities were scaled from the prototype values according to the scaling laws. This approach preserved stress levels and damage mechanisms (cracking and yielding) at the model scale.

Third, to realistically simulate the influence of soil on structural response, the shear wave velocities of the two model soils were designed to match those of typical soft clay and stiff clay sites in coastal areas, respectively.

Based on these design principles, the length, elastic modulus, and acceleration of the model were selected as the fundamental physical quantities. According to the Buckingham π theorem, the similitude ratios of other physical quantities were derived and are summarized in Table 1.

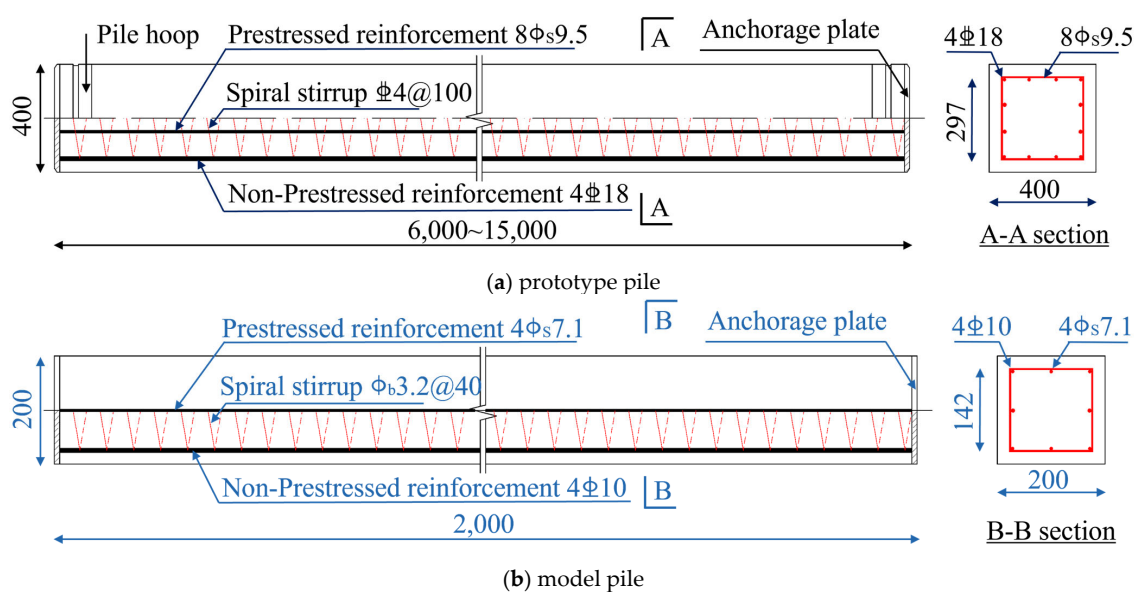
Table 1. Similitude relationship.

Type	Physical Quantity	Similitude Relationship		Model Structure	Model Soil
Geometry property	Length *	S_l	S_l	0.5	0.5
	Area	S_A	S_l^2	0.25	0.25
Material property	Stress *	S_σ	S_σ	1.0	1.0
	Elastic modulus	S_E	$S_E = S_\sigma$	1.0	1.0
	Mass	S_m	$S_l^2 S_\rho$	0.125	-
	Density	S_ρ	$S_\sigma / (S_A \cdot S_l)$	1.0	1.0
Dynamic property	Force	S_F	$S_l^2 S_E$	0.25	0.25
	Period	S_T	$S_l S_E^{-0.5} S_\rho^{0.5}$	0.5	0.5
	Frequency	S_f	$S_l^{-1} S_E^{0.5} S_\rho^{-0.5}$	2.0	2.0
	Shear wave velocity	S_v	$S_E^{0.5} S_\rho^{-0.5}$	-	1.0
	Horizontal acceleration *	S_a	S_a	2.0	2.0

Note: * indicates that the variable is a similar variable.

2.2. Model Pile Design and Fabrication

Following the similitude design principles established in Section 2.1, the prototype and model piles had the following specifications. In this study, the prototype PCHSS pile was designed with C70 concrete and a square cross-section of 400 mm side length, with pile lengths ranging from 6 to 15 m. The reinforcement comprised four 18 mm diameter corner bars, eight 9.5 mm diameter prestressing strands, and 4 mm diameter helical stirrups at 100 mm spacing. Following the scaling laws, the model PCHSS pile also used C70 concrete, with a length of 2 m and a square cross-section of 200 mm side length. The longitudinal reinforcement consisted of four 10 mm steel bars, which were selected to satisfy the flexural capacity similitude with the prototype, and the helical stirrups (3.2 mm diameter at 40 mm spacing) were determined based on the shear capacity similitude requirements [42]. Figure 2 illustrates the dimensions and reinforcement details of both the prototype and model piles.

**Figure 2.** PCHSS pile reinforcement.

The prestressing reinforcement in the model pile consisted of spiral-ribbed steel bars (also known as helical steel bars), which were tensioned to a control stress of 947 MPa, resulting in an effective prestress of 841 MPa after accounting for elastic shortening, creep, and shrinkage losses. This prestress level was designed to replicate the stress state of the prototype pile under service loads.

To verify that the fabricated model met the design specifications, material tests were conducted prior to the shaking table tests. Compressive tests were conducted on concrete cube specimens, yielding an average compressive strength of 74.4 MPa for the 100 mm specimens. Tensile tests were also performed on the steel rebars. All test results are presented in Table 2.

Table 2. Material properties of model pile.

Type	Elastic Modulus MPa	Yielding Strength MPa	Ultimate Strength MPa	Yielding Strain
$\phi_s7.1$	227,760	1572	1724	0.0069
ϕ_{10}	228,376	411	579	0.0018
$\phi_b3.2$	206,000	330	400	0.0016

Based on the confirmed material properties, the model piles were manufactured strictly following complete industrial production procedures of full-scale prototypes (Figure 3), including (a) cage binding, (b) tendon tensioning, (c) concrete casting, (d) natural curing, (e) detensioning and formwork stripping, and (f) storage and transportation. This approach ensured that the actual bond–slip constitutive relationship and the prestress relaxation behavior were preserved in the model piles.

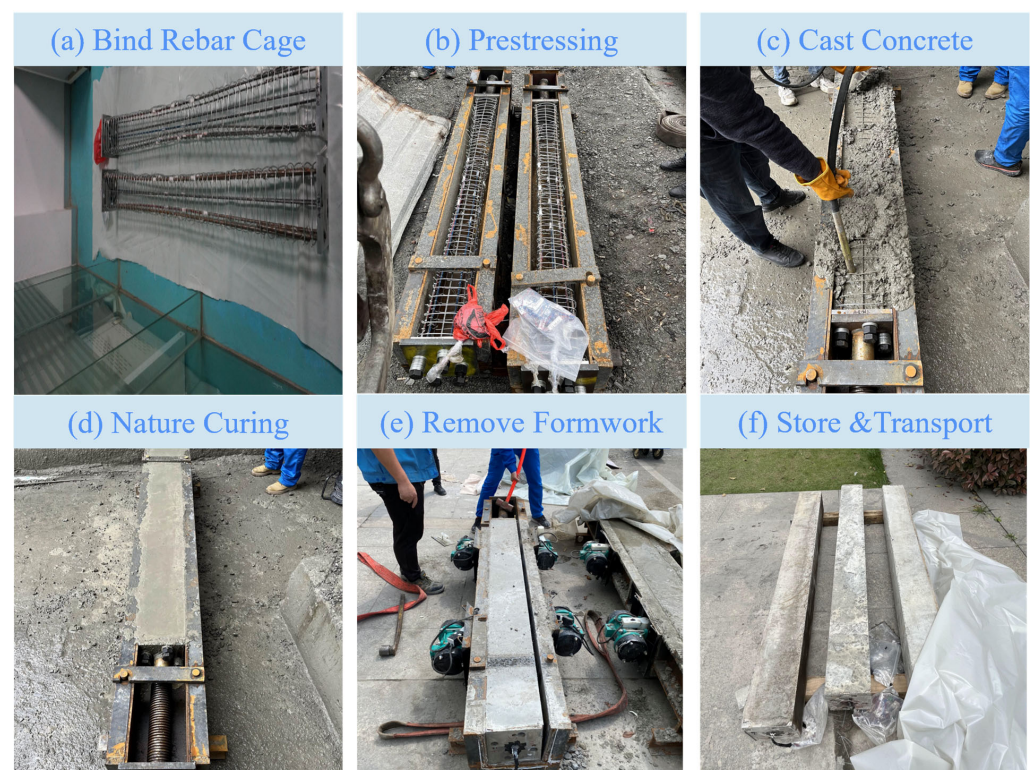


Figure 3. Fabrication steps of model PCHSS piles.

2.3. Sectional Performance Calculation and Verification

Prior to the shaking table tests, a systematic investigation was conducted to characterize the sectional behavior of the prestressed solid square piles. This section presents the strain, curvature, and moment relationships of the cross-section through theoretical analysis, numerical simulation, and experimental validation. The sectional analysis was performed based on the following assumptions: (1) plane sections remain plane after deformation; and (2) no bond–slip occurs between the steel reinforcement and the surrounding concrete. Figure 4 illustrates the strain and internal force distributions of a typical symmetrically reinforced prestressed section before and after cracking. Before cracking, both concrete and reinforcement exhibit linear elastic behavior, with the neutral axis located at the section centroid. As the growing tensile strain at the outermost concrete fiber exceeds the cracking strain, the concrete in the tension zone gradually spalls, causing the neutral axis to shift upward. Throughout this process, the strain distribution of each material satisfies the geometric compatibility, while the internal forces satisfy both the force and moment equilibrium equations. Relevant theories concerning sectional behavior have been extensively discussed in the existing technical literature [13].

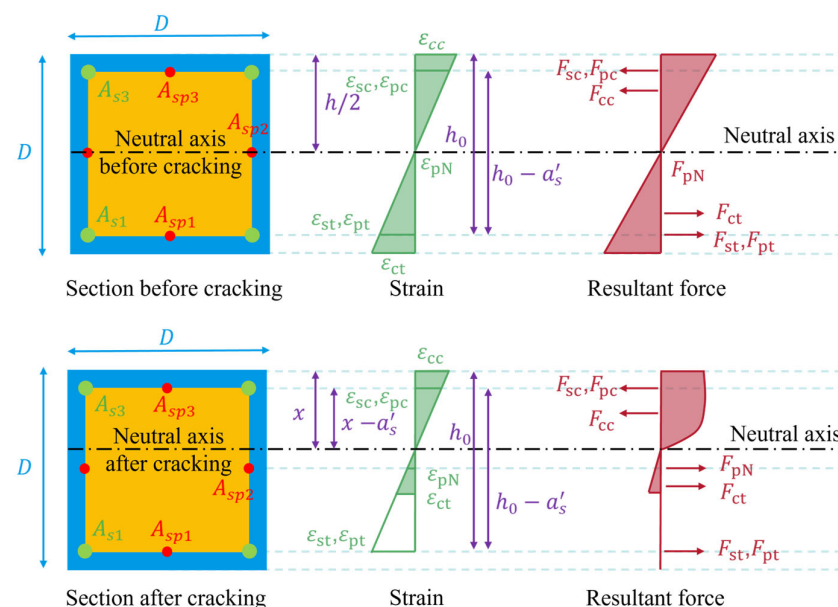


Figure 4. Strain and resultant force distributions of PCHSS pile sections.

The moment–curvature relationship is a key parameter for evaluating sectional performance. During seismic excitation, the PCHSS piles were subjected to combined axial and bending strains induced by the inertial forces of the superstructure and the kinematic forces of the soil. The curvature was calculated from the measured strain data as follows [29,43]:

$$\varnothing = (\varepsilon_{st} - \varepsilon_{sc}) / (D - a_s - a'_s) = (\varepsilon_{pt} - \varepsilon_{pc}) / (D - a_s - a'_s), \quad (1)$$

where $\varnothing$ is the curvature of the pile; $\varepsilon_{st}, \varepsilon_{pt}$ and $\varepsilon_{sc}, \varepsilon_{pc}$ are the measured tensile and compressive strains of longitudinal rebars, respectively, at the outermost edges of the section; and D is the side length of the PCHSS pile. The yield curvature, $\varnothing_y$, is defined as the curvature corresponding to the first attainment of the yield strain in the extreme tension reinforcement, which can be determined from the sectional analysis shown in Figure 5. The maximum curvature ductility demand (MCDD) can be determined from the measured pile curvature [29], which is defined as follows:

$$\mu_{\varnothing} = \varnothing_{\max} / \varnothing_y, \quad (2)$$

where μ_{ϕ} is the MCDD and $\phi_{\max}$ is the maximum curvature. For the present study, the yield curvature used in the MCDD calculation was obtained from the finite element moment–curvature curve under an axial load of 60 kN, which corresponds to the actual superstructure load condition.

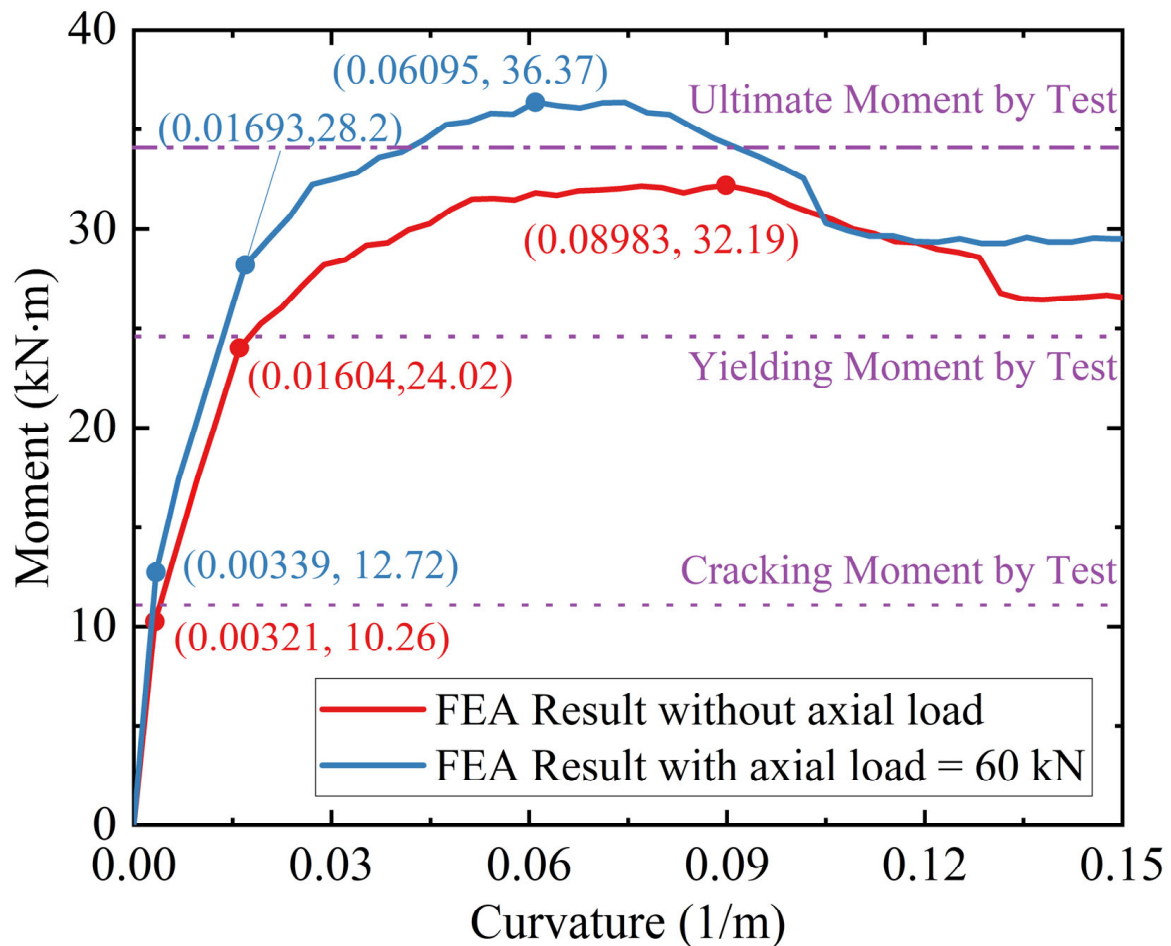


Figure 5. Moment–curvature result of model PCHSS piles.

With the curvature defined, the corresponding moment was determined using the nonlinear stress–strain relationships of the constituent materials and the sectional equilibrium equations. After cracking, the sectional stiffness degrades considerably; consequently, a nonlinear moment–curvature relationship is required to account for this stiffness degradation. The presence of prestressing force and axial load significantly influences this relationship. In this study, based on measured material strengths and constitutive models, numerical analyses were performed to obtain the moment–curvature curves under axial loads of 0 kN and 60 kN, as presented in Figure 5.

To validate the numerical method, bending tests were conducted on model piles from the same batch (without axial load), as shown in Figure 6. The tests yielded a cracking moment of 11 kN·m, a yielding moment of 24.5 kN·m, and an ultimate moment of 34 kN·m. The maximum deviation from the finite element results was within 5.3%, confirming the reliability of the numerical approach in predicting moment capacity. In the subsequent shaking table tests, the moment response was derived from the measured curvatures using the established moment–curvature curves.

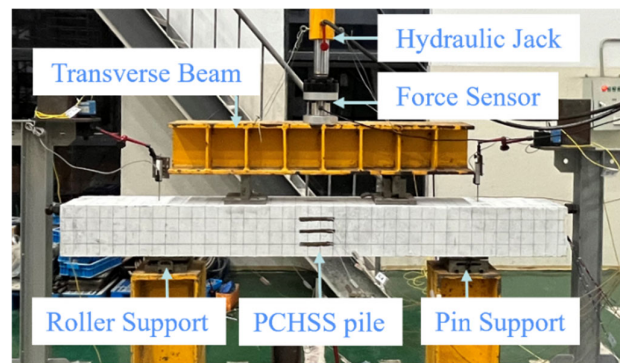


Figure 6. Bending test of model PCHSS piles.

2.4. Model Soil Preparation and Placement

To ensure the similarity of soil–pile interaction and eliminate the interference of soil layer variability on test results, the model ground was prepared using the same soil materials as those from the original site. The model soil adopted a single, uniform soil type to reveal the seismic response characteristics of PCHSS piles under clay ground conditions. To better simulate the characteristics of soft and stiff clays in coastal areas, relatively uniform soil from the first underground layer of a construction site in the Yangtze River Delta region was selected. The clay was air-dried, crushed, and sieved to prepare model soil powder with good uniformity and stable quality.

Prior to soil preparation, the relationship between soil water content and soil state was established through geotechnical tests. During the model soil preparation, the water content and relative density were precisely controlled using the gravimetric method to ensure that the elastic modulus and density met the similitude requirements. For soil filling, the model pile was first secured at the predetermined position within the shear box. The model soil was then placed in layers from bottom to top, with each layer approximately 20 cm thick. Simultaneously, sensors were positioned and embedded in the soil. After each layer was filled to the design elevation and leveled, surface surcharge loading was immediately applied. The surcharge system employed a load box containing steel weights with a total weight of 7 tons. Each loading stage lasted not less than 12 h, and after completion of the final soil layer, surcharge loading was maintained for 7 consecutive days. Throughout the entire surcharge process, drainage was maintained to ensure adequate consolidation of the model soil, thereby simulating the natural consolidation process of soil in coastal areas. The detailed preparation and filling process of the model soil is illustrated in Figure 7.



Figure 7. Model soil preparation and layered filling.

After completion of the surcharge loading, soil samples were collected using ring cutters and subjected to geotechnical tests to obtain the geotechnical parameters for the two types of shaking table model soils, which are summarized in Table 3. The test results indicate that the soft clay exhibited a flowing-plastic consistency, while the stiff clay exhibited a hard-plastic consistency. In addition, in situ shear wave velocity tests were conducted on the soil surface after surcharge loading, yielding values of 71 m/s for the soft clay and 96 m/s for the stiff clay. Differences in soil state between the two soil types were clearly observed during the surcharge loading and excavation processes, as shown in Figure 8.

Table 3. Physical parameters of soil model.

Soil Type	Water Content (%)	Natural Void Ratio	Liquid Limit	Liquidity Index	Plasticity Index	Soil State	Cohesion (kPa)	Angle of Internal Friction
Soft Clay	35.8%	1.013	35.6%	1.01	15.8	Flowing plastic	8.34	3.4°
Stiff Clay	23.10%	0.664	35.6%	0.21	15.8	Hard plastic	23.48	16.56°



Figure 8. Soil photos in the laminar box.

2.5. Instrumentation and Data Acquisition

The shaking table test setup was equipped with multiple types of sensors (Figure 9). Accelerometers (A1–A39) were installed in the upper mass block, along the pile shaft, and within the model soil at different elevations. The laser displacement sensors IL-300 and IL-600 (D1–D3) were placed at the center of the upper mass block, at the pile top, and on the shaking table. Soil pressure sensors (SP2–SP12) were pre-buried at the pile–soil interface. In addition, 44 strain gauges were mounted on the longitudinal reinforcing bars to monitor strain variations in the prestressing and non-prestressing reinforcement, and 22 concrete strain gauges were attached to the pile surface to monitor crack development under seismic loading.

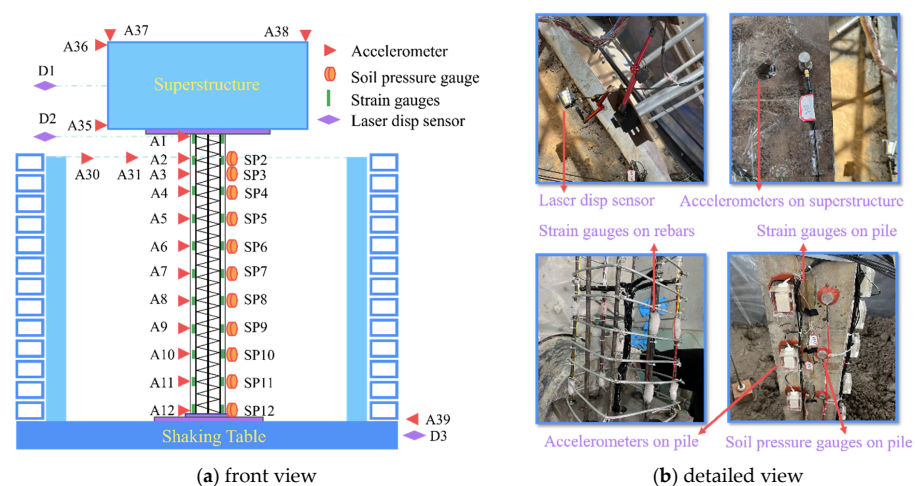


Figure 9. Schematic sensors layout of shaking table test.

2.6. Measurement Uncertainty and Data Reliability

To ensure the reliability of the experimental results, the uncertainties associated with sensor accuracy and measurement errors were quantitatively assessed. All sensors were newly purchased and factory-calibrated. The accelerometers had a full-scale range of ± 50 g with a resolution of 0.0002 g; the quantization uncertainty of the acceleration signal was therefore bounded by ± 0.0002 g. The strain gauges and earth pressure cells had manufacturer-specified nominal accuracies of $\pm 0.1\%$ and $\pm 0.5\%$ of full-scale output (FSO), respectively, giving maximum absolute uncertainties of ± 20 $\mu\epsilon$ for strain and ± 2.5 to ± 5.0 kPa for earth pressure (depending on the sensor range). Two laser displacement sensors were also employed: an IL-300 (measurement range, ± 140 mm) and an IL-600 (range, ± 400 mm). Both sensors had a linearity of $\pm 0.25\%$ FSO (referenced to the half-range), yielding absolute errors of ± 0.35 mm and ± 1.0 mm, respectively. The testing environment was maintained at 25–30 °C and standard atmospheric pressure, well within the operational limits of all sensors.

Prior to installation, all accelerometers and displacement sensors were mounted on the shaking table surface and calibrated against the table's built-in high-precision reference sensors. Cross-comparison among sensors was conducted, and only instruments exhibiting response deviations within 5% were retained. Temperature compensation was applied to the strain gauges to mitigate thermal drift. After installation into the concrete piles or the shear soil box, all embedded sensors underwent secondary verification via pre-loading of the shaking table. Owing to the prolonged curing periods, high water content of the soft clay, and surcharge loading, some sensors were confirmed to be damaged during this secondary inspection. Because sensors embedded in soil and concrete could not be replaced, data from these compromised instruments were excluded from subsequent analyses.

Overall, the combined effects of all measurement errors and sensor inaccuracies are estimated to yield a total system uncertainty within $\pm 10\%$, which does not affect the main conclusions of this study.

2.7. Input Motion and Testing Protocol

To investigate the effects of different ground motions on the seismic response characteristics of PCHSS piles in clay ground, four ground motions were selected based on their spectral characteristics: the Wenchuan Earthquake Songpang and Wolong records (SP and WL), the Chi-Chi record (CC), and the Imperial Valley record (EC). During the test, the ground motion duration was scaled to half the prototype duration according to the time similitude relationship, and the corresponding frequency was scaled to twice that of the prototype. Differences were observed in the frequency contents of the four seismic records. As Figure 10 illustrates, the scaled SP, WL, CC, and EC records have frequency contents of 21–24 Hz, 4.5–8 Hz, 1.25–5.0 Hz, and 2–4.5 Hz, respectively. The selected input motions were divided into two spectral categories, taking into account the characteristic period of the pile–soil system. Scaled ground motions (SP and WL) with predominant periods less than 0.25 s ($f > 4.0$ Hz) were designated as the short-period group; these excitations induce pronounced inertial shear forces at the pile head and are primarily used to examine the local brittle failure potential of the hybrid-reinforced section. Scaled ground motions (CC and EC) with predominant periods greater than or equal to 0.25 s ($f \leq 4.0$ Hz) were designated as the long-period group; these excitations are critical for evaluating the deep-seated flexural deformation and the overall kinematic response of the pile–soil system. In cases where a ground motion exhibited broadband spectral content spanning the 0.25 s threshold, its classification was determined by the mean period of its Fourier amplitude spectrum. White noise (WN) was used to evaluate the natural frequency and damping ratio of the test model before the ground motion loading. The test protocols used in this study are listed in

Table 4. Pre-test numerical simulations and theoretical sectional analyses, as detailed in Section 2.3, indicated that the PCHSS pile would develop cracking and enter the nonlinear stage at $\text{PGA} = 0.2 \text{ g}$. To avoid cumulative damage from prior sequences affecting later higher-intensity runs, only single-ground-motion excitations were applied after the 0.2 g stage. The EC record was selected as the input motion for these subsequent runs, as its frequency content closely matched the characteristic site period of the prototype soil–pile system, making it representative of the design-level seismic demand.

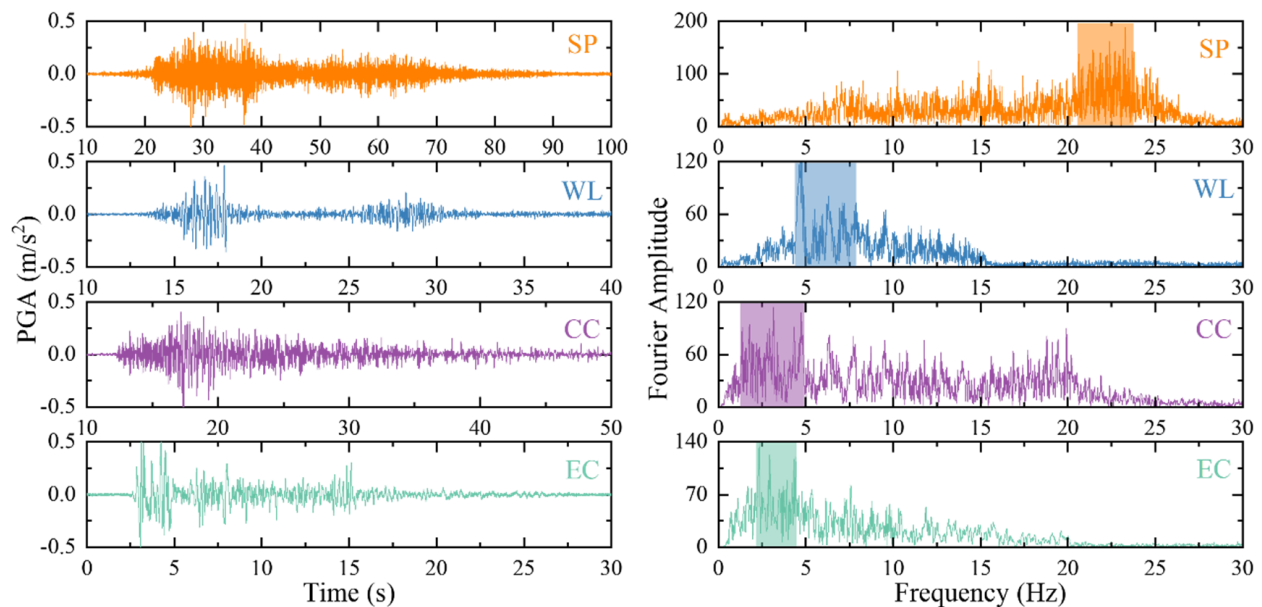


Figure 10. Time histories and frequency content of ground motion (scaled). (Note: The shaded areas indicate the predominant frequency ranges).

Table 4. Test cases.

Case	Ground Motion Record	PGA (g)	Case	Ground Motion Record	PGA (g)
WN-1	White Noise	0.03	CC-3	CC	0.2
SP-1	SP	0.05	EC-3	EC	0.2
WL-1	WL	0.05	WN-4	White Noise	0.03
CC-1	CC	0.05	EC-4	EC	0.3
EC-1	EC	0.05	WN-5	White Noise	0.03
WN-2	White Noise	0.03	EC-5	EC	0.4
SP-2	SP	0.1	WN-6	White Noise	0.03
WL-2	WL	0.1	EC-6	EC	0.5
CC-2	CC	0.1	EC-7	EC	0.6
EC-2	EC	0.1	EC-8	EC	0.7
WN-3	White Noise	0.03	EC-9	EC	0.8
SP-3	SP	0.2	WN-7	White Noise	0.03
WL-3	WL	0.2	/	/	/

3. Results

3.1. Frequency and Damping

Seven white noise sweep tests were conducted at different loading stages to track the evolution of natural frequencies and damping ratios of the pile foundation system. At the initial stage of the tests, the natural frequencies under soft and stiff clay conditions were 3.00 Hz and 3.13 Hz, with corresponding damping ratios of 7.04% and 9.60%, indicating that the stiff clay provided higher initial stiffness and more effective lateral restraint. With

increasing peak ground acceleration (PGA), the natural frequency decreased and the damping ratio increased in both clay types (Figure 11). After the final sweep at 0.80 g PGA, the soft clay system exhibited a 37.3% reduction in frequency and a 30.0% increase in damping, whereas the stiff clay system showed a 28.0% reduction and a 101.0% increase. The more pronounced stiffness degradation in soft clay is attributed to its lower initial stiffness and bearing capacity, exacerbating deterioration under strong shaking. Conversely, the stiff clay ground—with its higher cohesion, denser structure, and greater initial stiffness—demonstrated superior energy dissipation capacity, as reflected by the more substantial increase in damping ratio.

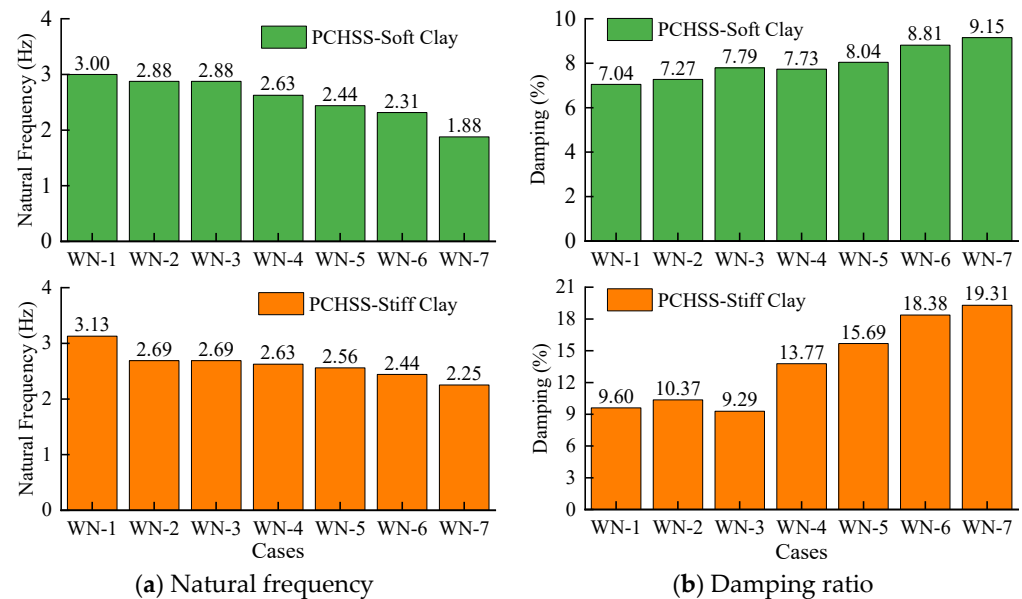


Figure 11. Frequency and damping ratio of the soil–pile system.

3.2. Dynamic Response of Superstructure

Figure 12 presents the acceleration response spectra of superstructures supported by PCHSS piles in both soft and stiff clay. At $\text{PGA} = 0.05 \text{ g}$, the spectral shapes are essentially identical between the two ground conditions, with only minor amplitude differences—the stiff clay generally exhibits slightly higher responses than the soft clay. The Fourier amplitude near the first natural frequency of the system (2.13 Hz) is significantly amplified, whereas at low and high frequencies, no appreciable amplitude changes are observed, indicating that vibrational energy away from the structural natural frequency is effectively filtered.

As the input PGA increases, several notable trends emerge. First, the spectral peak period shifts toward longer periods for each ground condition, indicating stiffness degradation and accumulated damage of the pile–soil–superstructure system. Second, for excitations above 0.05 g, a secondary peak gradually appears in the spectral curves, corresponding to the second natural frequency of the system (soft clay: 11.875 Hz; stiff clay: 12.563 Hz), which suggests that higher-mode contributions begin to influence the superstructure response. Under the WL motion, the spectral amplitude at this second peak even exceeds that at the first peak; similar observations have been reported in the literature [28]. Third, once the secondary peak develops, the spectral responses in stiff clay become notably larger than those in soft clay in the period range from 0.01 s to the first-peak period. Fourth, the amplitude discrepancies among different input motions grow; the CC and EC records yield peak spectral amplitudes up to 2–3 times those of SP and WL at $\text{PGA} = 0.2 \text{ g}$.

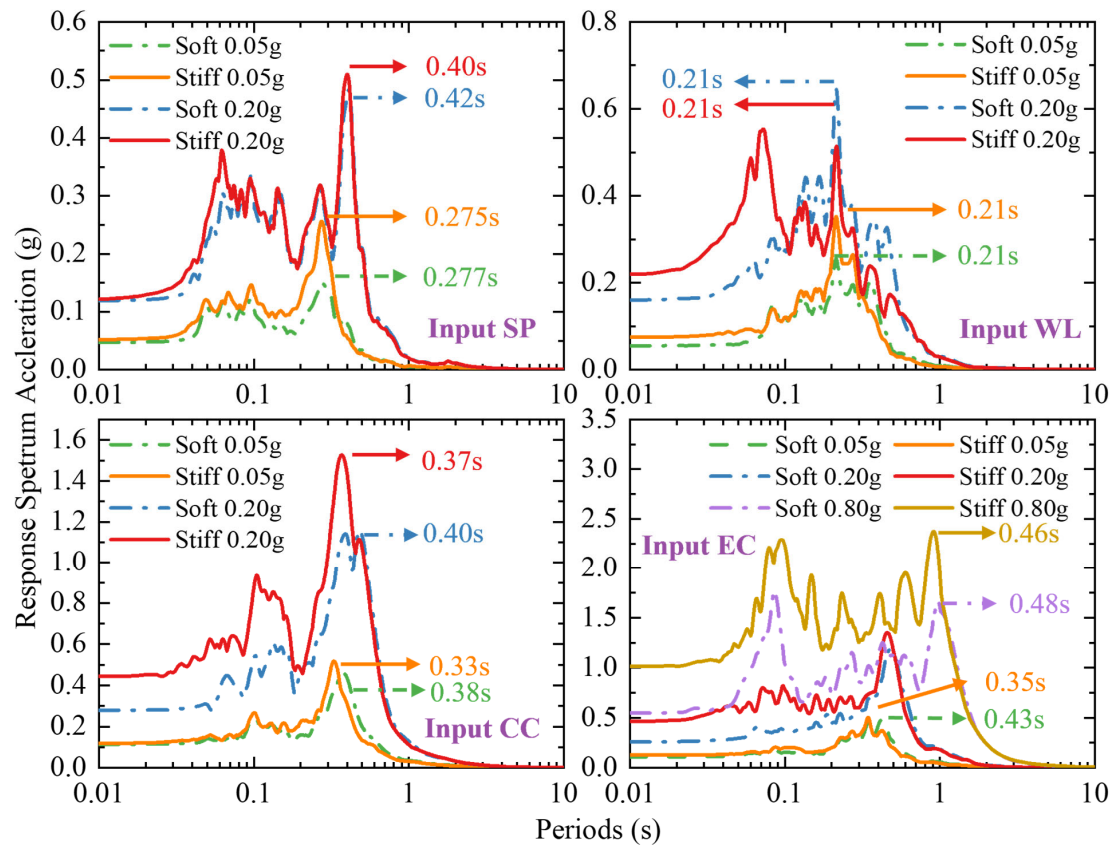


Figure 12. Acceleration response spectra of superstructures.

Figure 13 presents the dynamic load–displacement hysteresis curves of the superstructure under EC excitation. Ground stiffness significantly influences the energy dissipation mechanism and deformation characteristics. In soft clay, as PGA increases from 0.05 g to 0.8 g, the inertial force approaches saturation while deformation grows rapidly, resulting in flattened hysteresis loops—indicating that energy is dissipated mainly via irrecoverable plastic flow. At 0.8 g, the stiff clay condition develops an inertial force 1.9 times that of the soft clay condition, with fuller hysteresis loops, greater restoring force and damping, and consequently higher hysteretic energy dissipation efficiency.

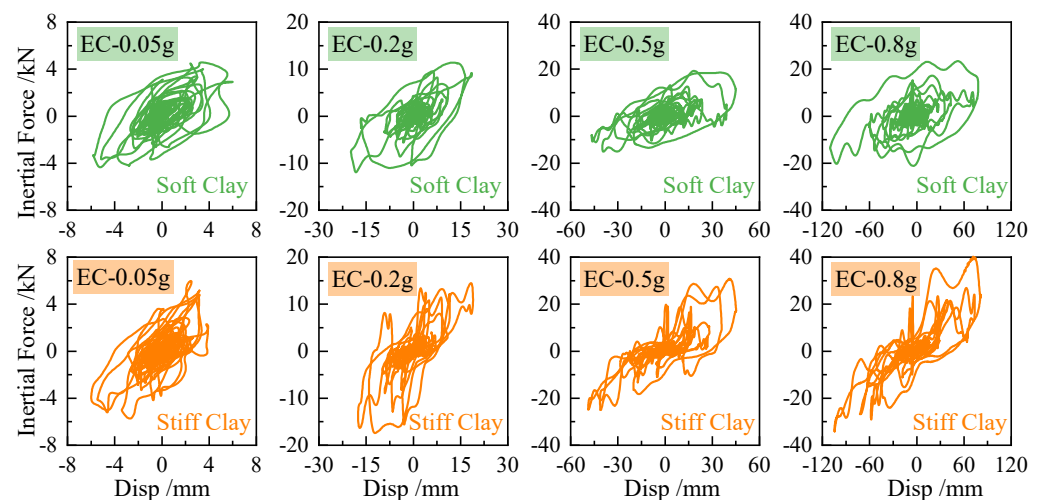


Figure 13. Load–displacement curves of superstructures under EC excitation.

3.3. Dynamic Response of Soil

The time-history curves of dynamic earth pressure on PCHSS pile in soft and stiff clay under the EC ground motion are presented in Figure 14. At $\text{PGA} = 0.05 \text{ g}$, the dynamic earth pressure in both soils exhibits pronounced positive–negative alternation. It should be noted that negative dynamic earth pressure does not indicate a tensile state in the soil, but rather a partial reduction relative to the at-rest earth pressure, implying that the soil remains in close contact with the pile shaft and deforms compatibly. As PGA increases to 0.8 g , the amplitude of dynamic earth pressure rises rapidly, and progressive separation occurs at the pile–soil interface. In regions with relatively large dynamic pressure amplitudes, the time-history curves exhibit a pronounced unilateral characteristic—the positive peak pressure continues to increase, while the negative peak essentially ceases to grow.

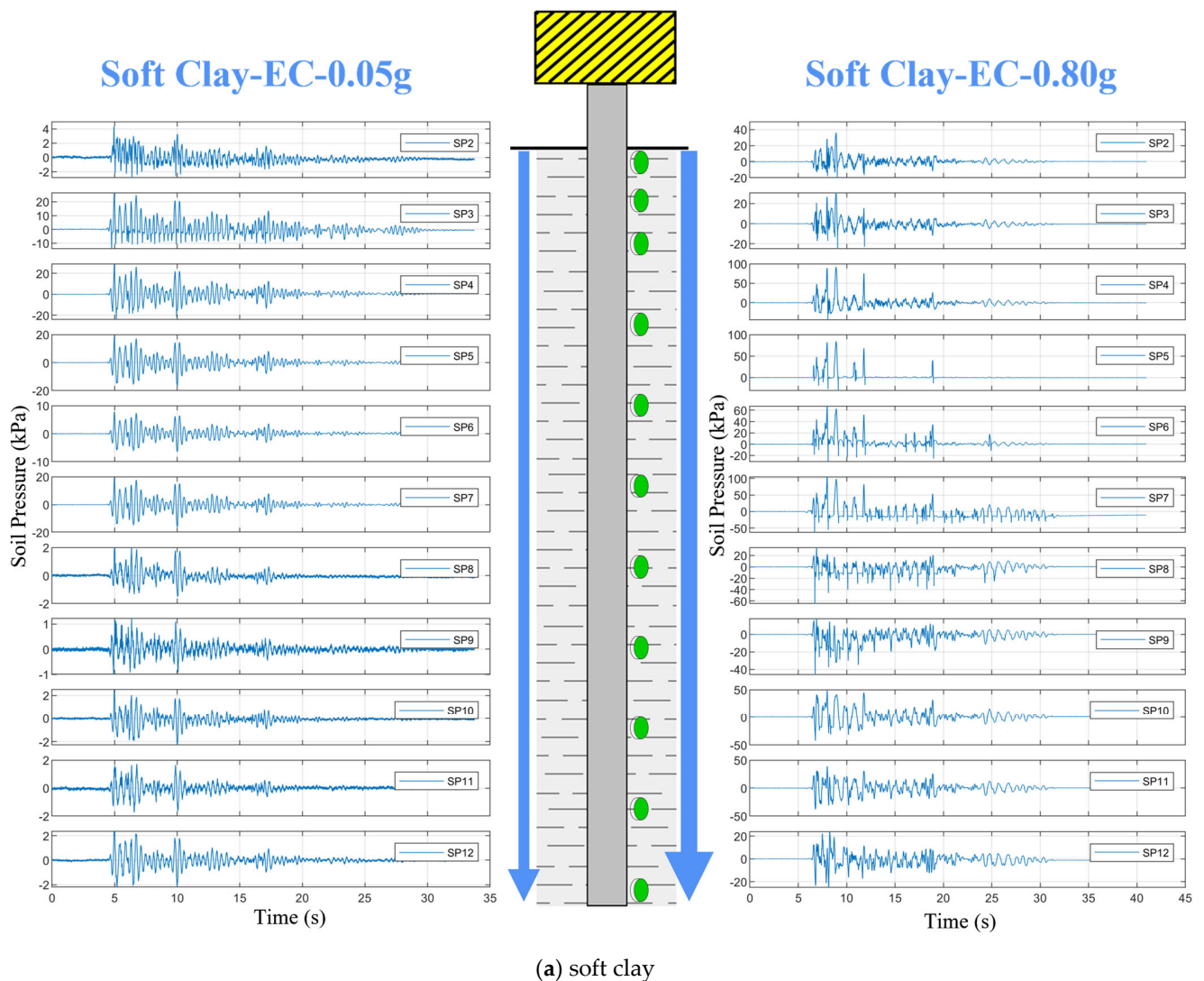


Figure 14. Cont.

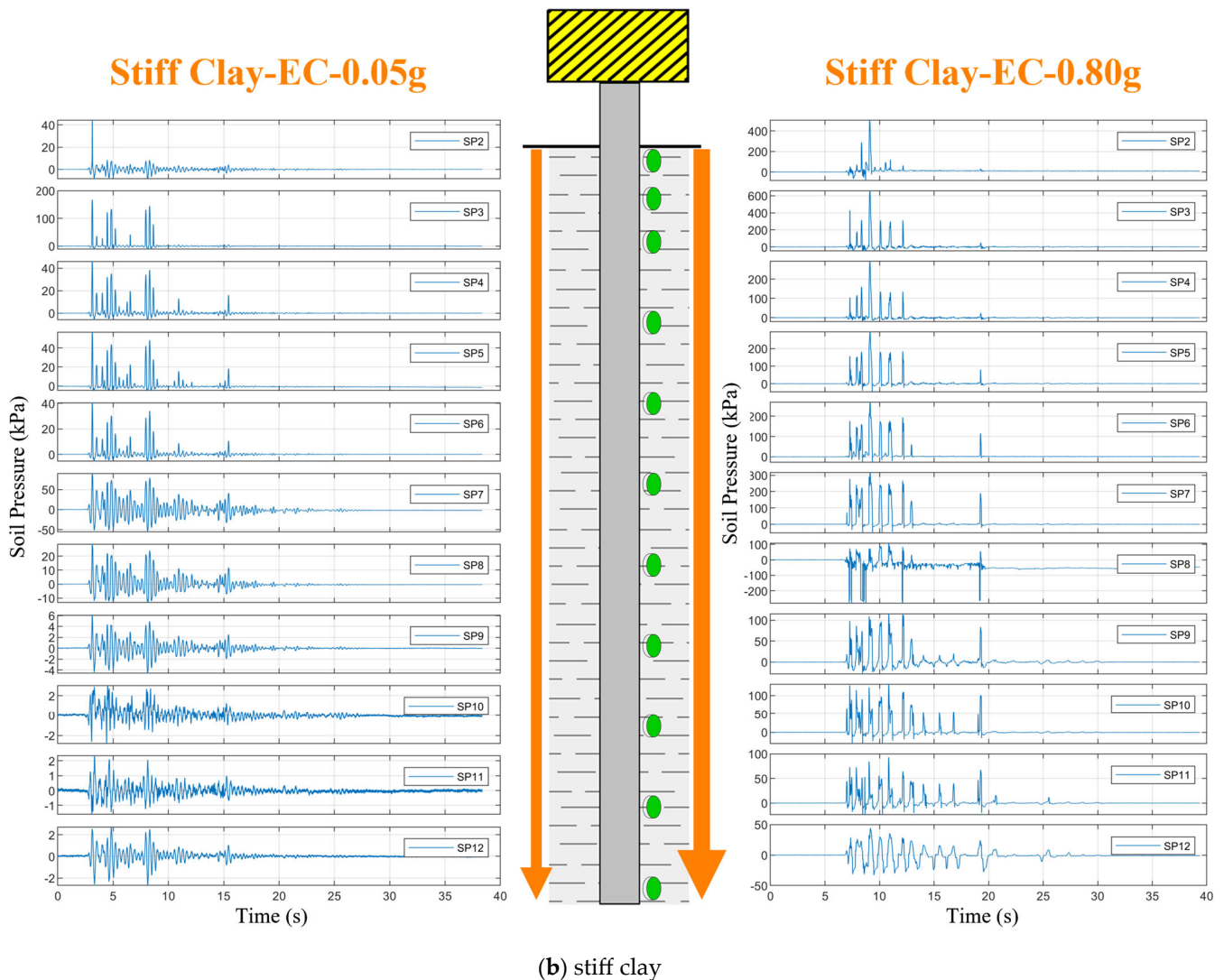


Figure 14. Time-history curves of dynamic earth pressure on pile foundations.

Figure 15 presents the distribution of maximum dynamic earth pressure along PCHSS piles in soft and stiff clay under various ground motions. The pressure decreases with burial depth, with the peak occurring slightly below the surface owing to the free-surface effect. Amplitudes in stiff clay are substantially higher than in soft clay, indicating stronger lateral support. Long-period group (CC and EC) motions produce larger dynamic pressures than short-period group motions (SP and WL), suggesting that pile–soil interaction is governed not only by superstructure inertia but also by motion characteristics.

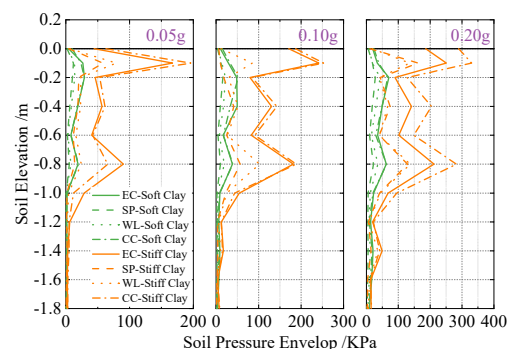


Figure 15. Dynamic earth pressure on pile foundations.

3.4. Maximum Curvature Ductility Demand

The MCDD values reported herein were calculated using ϕ_y from the finite element analysis under the 60 kN axial load, as described in Section 2.3. Figure 16 presents the MCDD of PCHSS piles under various ground motions and PGA levels. The MCDD generally increases with PGA, indicating a positive correlation between excitation intensity and curvature demand. For a given PGA and input motion, the MCDD in stiff clay consistently exceeds that in soft clay, suggesting that stiffer ground conditions impose higher ductility demands on the pile. Under the final loading case with a PGA of 0.80 g, the MCDD in stiff clay exceeds that in soft clay by 42.63%. This soil-dependent trend is, however, secondary to the effect of motion frequency content. For the same PGA, long-period records (CC and EC) produced MCDD values 2–3.36 times greater than those from short-period records (SP and WL), indicating that long-period motions are considerably more detrimental to the PCHSS piles.

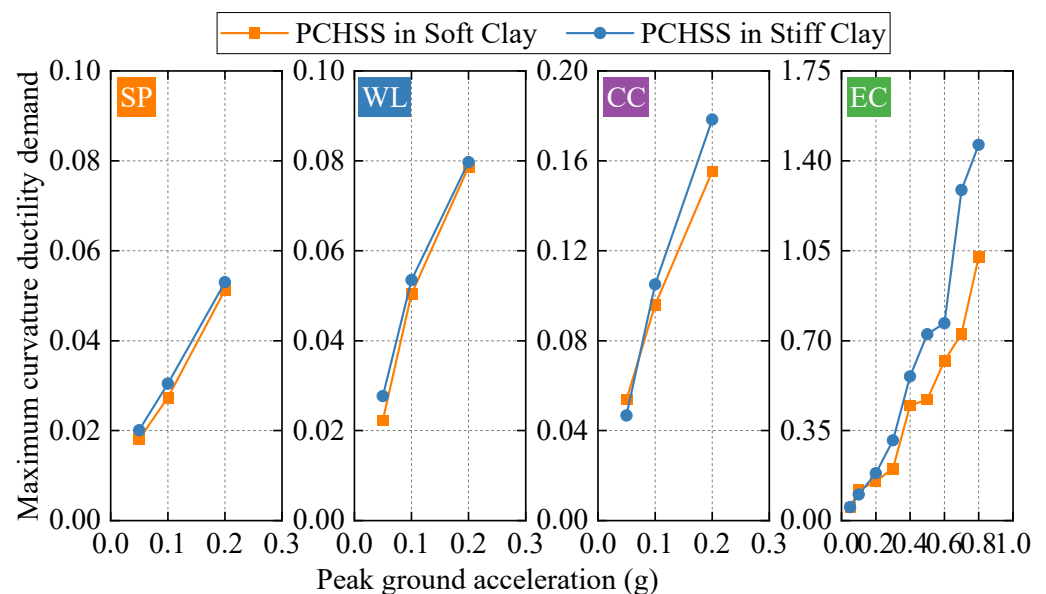


Figure 16. MCDD of PCHSS pile in soft verse stiff clay under various motions.

3.5. Bending Moment Response

Figure 17 presents the bending moment envelopes of the PCHSS piles in soft and stiff clay. At PGA = 0.05 g, the moment distributions in both clays are nearly identical, exhibiting an initial increase followed by a decrease with depth, with the peak located at 2D (D denotes the side length of the PCHSS pile). As the PGA increases, the peak bending moment rises progressively and eventually reaches the cracking moment. Concurrently, the depth of the maximum moment begins to diverge between the two soil conditions. In soft clay with lower shear strength, the peak-moment location shifts downward to the 4D depth range, attributed to larger soil deformation around the pile; in stiff clay with relatively higher shear strength, the peak shifts to the 2D–3D range. This divergence becomes increasingly pronounced under higher seismic loading and associated soil deformation, highlighting the dominant role of soil shear strength in governing the internal force redistribution of the pile. Furthermore, the peak bending moment is strongly dependent on the ground motion type. Under the same PGA, the moment values under the CC and EC motions are 2–3 times those under the SP and WL motions.

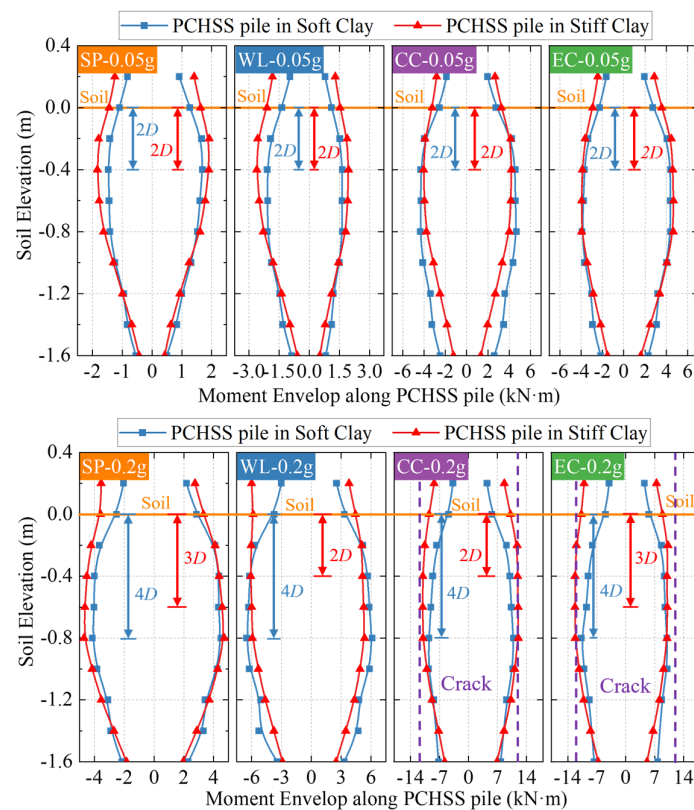


Figure 17. Bending moment along PCHSS pile.

Figure 18 illustrates the bending moment profiles under the EC motion. At $\text{PGA} = 0.8 \text{ g}$, the peak moment deepens to $4D$ – $6D$ in soft clay, versus $1D$ – $3D$ in stiff clay. Under identical PGA, bending moments in the stiff clay foundation exceed the yielding moment within the $1D$ – $2.8D$ range, whereas those in the soft clay foundation remain below the yielding threshold. This discrepancy implies distinct capacity requirements for the two soil types, warranting consideration in design.

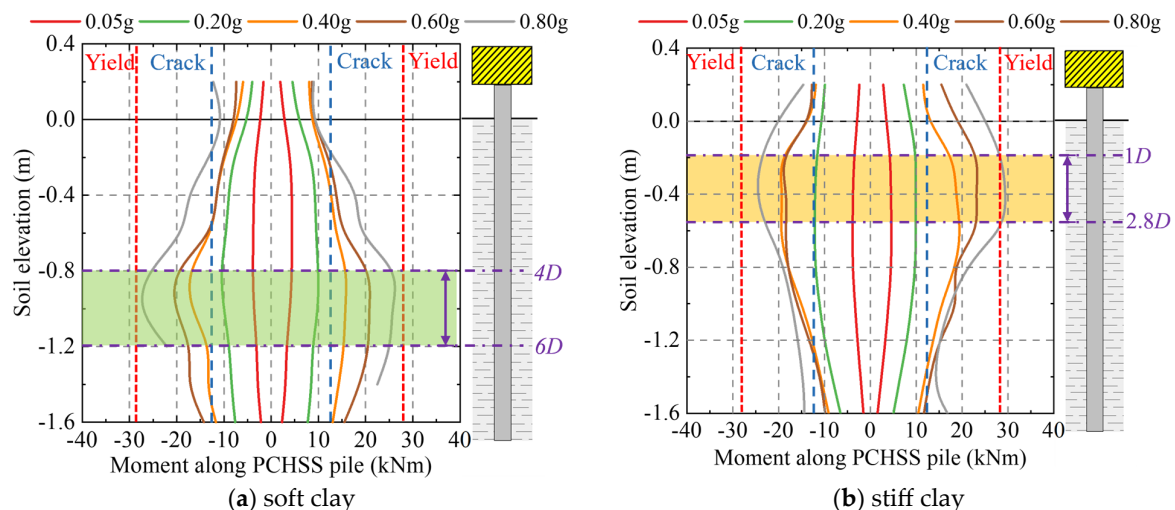


Figure 18. Envelope diagram of pile bending moment under EC ground motion.

3.6. Post-Test Physical Observations

After removal of the mass block at the end of the test, the relative displacement characteristics at the pile–soil interface were clearly observed, as shown in Figure 19. In soft clay with a fluid-like consistency, the ground surface exhibited notable heaving and

a pronounced plastic flow tendency, with gap spacings of 40 mm and 53 mm measured on the left and right sides of the pile shaft, respectively. In the stiff clay foundation, the surface soil predominantly showed compaction and tension cracking, with corresponding gap spacings of 38 mm and 43 mm.



Figure 19. Ground surface deformation after shaking table test.

According to the Chinese design codes for pile foundations subjected to lateral loads, the failure criterion may be defined either by the pile shaft strength reaching its ultimate bearing capacity or by the pile deformation reaching a specified displacement limit. In this study, pile failure is herein defined as the attainment of ultimate bearing capacity, manifested as shaft fracture or local crushing. Excavation and cleaning of the model piles embedded in the two soil types revealed that no severe damage or fracture occurred along the full length of the PCHSS piles; only relatively uniformly distributed circumferential cracks were observed (Figure 20). This failure pattern indicates that the PCHSS pile possesses satisfactory bearing capacity and durability. However, the distribution ranges of cracks along the pile shaft differed markedly between the two soil conditions: in soft clay, cracks were concentrated in the depth interval of $4D$ – $7D$, whereas in stiff clay, they fell within $0D$ – $5D$. Furthermore, the crack distribution closely matched the bending moment diagrams.

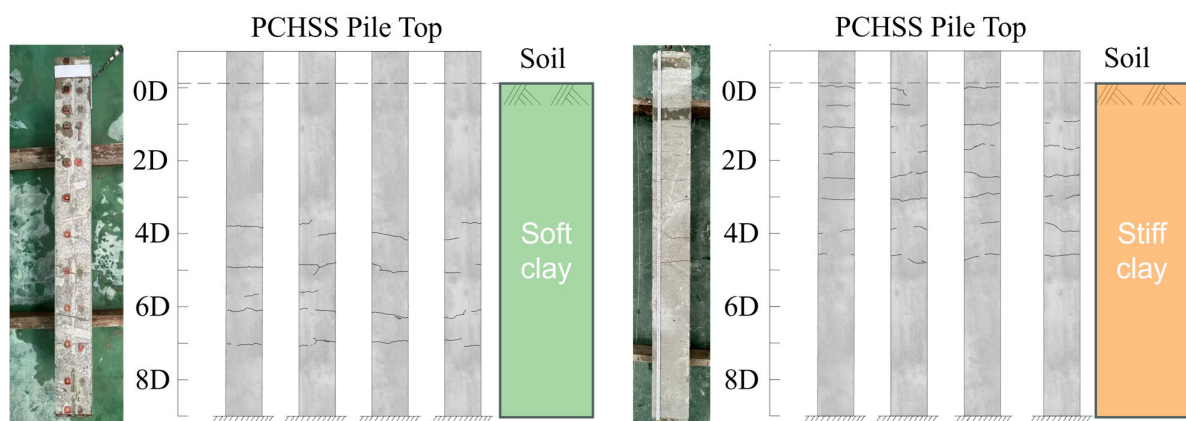


Figure 20. Post-excavation crack distribution along the PCHSS piles.

However, the above cracking patterns are not entirely consistent with the provisions of current codal guidelines for prefabricated piles. These codes typically specify a strengthened zone within a fixed depth of $4D$ or 2 m below the pile head. Nevertheless, for piles embedded in soft soil regions subjected to strong seismic loading, this prescribed depth may be insufficient to cover the actual damaged zone, and thus a re-evaluation considering soil properties is warranted.

4. Discussion

This study presents large-scale experimental evidence on the seismic response of PCHSS piles in clay deposits of varying stiffness, enabling direct validation of the two hypotheses formulated in the Introduction.

4.1. Validity of Boundary Conditions

Before interpreting the experimental results, the effectiveness of the boundary conditions was examined. EVA foam was applied to the inner walls of the shear box to reduce wave reflection, as validated previously [29,31]. In the present tests, two extra observation points (A30, A31) on the soil surface recorded waveform consistency and amplitude deviations less than 10% (Table 5), confirming that the foam effectively attenuated reflected waves and that the soil mass simulated a semi-infinite medium.

Table 5. Comparison of acceleration amplitudes at observation points (m/s^2).

Soil Condition	Observation Points	0.05 g (EC)	0.4 g (EC)	0.8 g (EC)
Soft clay	A30 near the shear box	1.007	8.923	19.746
	A31 near the model pile	1.015	8.495	18.459
	Amplitude deviation	+0.79%	−4.80%	−6.52%
Stiff clay	A30 near the shear box	0.873	7.601	17.880
	A31 near the model pile	0.878	8.304	17.916
	Amplitude deviation	+0.57%	+9.25%	+0.20%

Regarding the pile-toe boundary, the pile toe was connected to the shear box bottom via an adaptor plate with bolted semi-rigid connections, simulating vertical and rotational restraints from underlying deep soils. Before the shaking table test, finite-element analysis was performed to evaluate the influence of pile length on bending moment response. The bending moment profiles (magnitudes and peak locations) for a 2 m pile were essentially consistent with those for 10 m and 20 m piles, with a maximum amplitude deviation of −3.8% (Figure 21). During the shaking table tests, no significant relative displacement or rotation was observed at the pile base, confirming that the semi-rigid connection adequately reproduced the intended restraints without introducing spurious fixity.

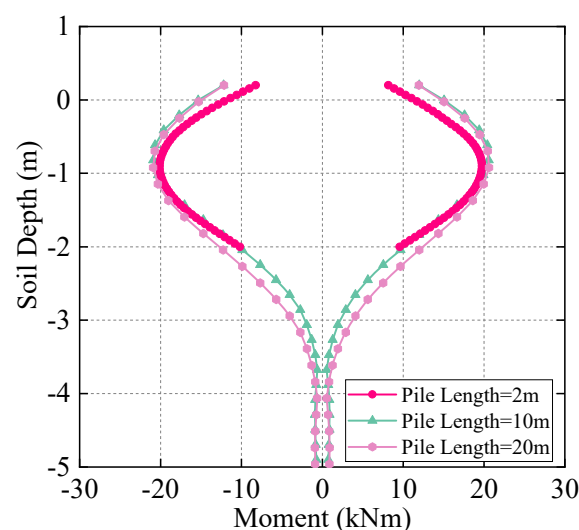


Figure 21. Finite element results for bending moment distribution under different pile lengths.

4.2. Seismic Failure Mechanisms: Soft Versus Stiff Clay

Hypothesis H1. This hypothesis proposed that in soft clay, the pile would undergo flexure-dominated failure with a deeper plastic hinge zone. This hypothesis is unequivocally validated by the experimental results. The peak bending moment in soft clay migrated from 2D at PGA = 0.05 g to 4D–6D at PGA = 0.8 g (Figures 17 and 18), and post-test crack inspection confirmed damage concentration at 4D–7D (Figure 20). The downward shift in the maximum moment region is attributed to the low lateral resistance of soft clay, which allows the pile to undergo larger lateral deflections and transfer the bending demand to deeper sections. This finding is consistent with the numerical predictions of Boulanger et al. [25]. However, the present study extends these prior works by quantifying the specific depth range (4D–7D) where the plastic hinge develops, thereby providing actionable design guidance.

Hypothesis H2. This hypothesis proposed that in stiff clay, the failure mode would transition toward flexural–shear cracking localized near the pile head. This hypothesis is partially confirmed. Post-test cracks are concentrated at 0D–5D in stiff clay versus 4D–7D in soft clay, confirming the shallower damage zone. Furthermore, at the same PGA of 0.8 g, bending moments in the stiff clay foundation exceed the yielding moment within the 1D–2.8D range, whereas those in the soft clay foundation remain below the yielding threshold. These results indicate that flexural–shear interaction, rather than pure flexure, governs the failure mechanism in stiff clay. The higher lateral restraint provided by stiff clay restricts the pile’s lateral deflection, concentrating flexural demands near the pile head while simultaneously increasing shear demands. This observation aligns with the findings of Zhang et al. [28] and Wang et al. [29] on PHC and PRHC piles, though the magnitude of the bending moment increase in PCHSS piles is notably higher, likely due to the enhanced stiffness of the hybrid-reinforced solid section.

4.3. Effect of Ground Motion Frequency Content

A critical finding is the pronounced effect of ground motion frequency content on pile response. Under identical PGA, long-period motions (CC, EC) generated MCDD and peak bending moments 2–3 times greater than those from short-period records (SP, WL). Similarly, at PGA = 0.2 g, CC and EC amplified the superstructure response spectra, with peak spectral amplitudes 2–3 times greater than those of SP and WL. This amplification arises from near-resonance coupling between the dominant frequency band of long-period motions (1.25–5.0 Hz) and the elongated fundamental period of the pile–soil system (initially 3.0–3.125 Hz, degrading with damage), corroborating Boominathan and Ayothiraman [25]. Additionally, the second modal period of the system begins to influence the superstructure response, suggesting growing higher-mode contributions. Practically, PCHSS piles in regions prone to long-period motions (e.g., distant large earthquakes or soft soil sites) demand flexural capacity exceeding conventional design provisions.

4.4. Evolution of Pile–Soil Interaction and Cumulative Damage Under Seismic Sequences

For shaking table tests, cumulative damage under sequential ground motions is an inherent concern. To address this, the loading protocol was designed based on pre-test simulations, which predicted crack initiation at approximately 0.2 g. Thus, a single-earthquake loading scheme was adopted for all PGA levels above 0.2 g to avoid prior damage accumulation. White noise sweeps after each PGA level monitored system frequency and stiffness in real time, providing ongoing damage assessment. Post-test strain-based calculations confirmed that CC and EC records induced cracking at 0.2 g, with bending moments

reaching the cracking moment. In contrast, under SP and WL records at the same PGA, calculated moments remained below the cracking threshold, indicating an essentially elastic response. Although no cracking occurred in the piles, white noise results still revealed a reduction in system frequency, indicating localized deformation of the surrounding soil under sequential ground motions. To allow sufficient soil recovery, the intervals between successive loading events were extended, and the total testing period was prolonged to two days.

Based on these validated comparisons, the evolution of pile–soil interaction under sequential shaking is examined next. The dynamic earth pressure records (Figures 14 and 15) reveal a transition from compatible pile–soil deformation under minor earthquakes to progressive separation under strong shaking. This transition affects the lateral resistance mechanism, as loss of soil–pile contact in the upper shaft reduces lateral restraint and shifts the peak bending moment downward. Such separation is more pronounced in soft clay, where low shear strength promotes earlier gap opening, with 40–53 mm spacings observed at the interface (Figure 19).

4.5. Implications for Code-Specified Reinforcement Zone

Regarding the code-specified reinforcement zone, the experimental evidence suggests that the standard 4D (or 2 m) strengthened zone below the pile head may be insufficient for soft clay foundations under strong seismic loading. The observed crack distribution (4D–7D in soft clay) warrants an extended reinforcement zone of up to 7D for PCHSS piles. For soils softer than those tested (e.g., silty clay), even longer reinforcement may be necessary. However, due to the limited soil types investigated, the generalizability of this conclusion remains open. For stiff clay, although the damage zone (0D–5D) falls within the code-specified 4D range at lower PGAs, the elevated bending moments near the pile head require enhanced flexural–shear capacity.

4.6. Comparative Assessment with Previous Prestressed Pile Studies

To contextualize the contributions of the present study within the broader literature on prestressed concrete piles, Table 6 summarizes the key differences between this investigation and previous experimental studies on PHC and PRHC pile systems. The comparison focuses on five aspects that critically influence seismic response: (1) pile cross-section type, (2) reinforcement configuration, (3) soil condition, (4) loading protocol, and (5) primary failure mode observed.

Table 6. Comparative assessment with previous prestressed pile studies.

Aspect	Previous PHC Pile Studies	Previous PRHC Pile Studies	This Study (PCHSS Pile)	Implication/Advancement
Cross-section	Hollow circular	Hollow circular	Solid square	Solid section provides higher flexural and shear stiffness
Reinforcement configuration	Prestressing strands only	Hybrid reinforcement	Hybrid reinforcement	Hybrid system delays spalling of concrete cover and improves ductility under cyclic loading
Scale ratio	1:3	1:3	1:2	/
Soil condition	Stiff clay	Stiff clay	Saturated soft clay and stiff clay	Systematic comparison of PCHSS pile response across clay stiffness gradients

Table 6. *Cont.*

Aspect	Previous PHC Pile Studies	Previous PRHC Pile Studies	This Study (PCHSS Pile)	Implication/Advancement
Loading protocol	Dynamic shaking table			/
Mass block	6-ton			/
Primary failure mode	The pile shaft fractured and tilted, and the prestressed reinforcement buckled at 0.40 g	Local concrete spalling was observed on the pile shaft at 0.62 g, with no residual deformation	No pile failure occurred at 0.80 g, and no residual deformation was observed	PCHSS piles exhibit higher seismic bearing capacity and deformation capacity
Key quantitative finding	Cracks were few and unevenly distributed within 0D–3.1D, and the plastic hinge zone was typically within 1.5D–2D	Cracks were uniformly distributed within 0D–4D, with the plastic hinge zone typically within 1.5D–2D	Soft clay: cracks were uniformly distributed within 4D–7D; cracks were uniformly distributed within 0D–5D	Provides experimental evidence that code-specified 4D reinforcement zone may be insufficient in soft clay

4.7. Limitations and Future Research

This study was conducted on single piles in uniform clay layers under unidirectional horizontal shaking. While the instrumentation and procedures described here offer a reproducible framework for numerical validation, several factors limit the direct generalization of the findings.

It should also be mentioned that bi-directional horizontal tests were included in the experimental program, and numerical simulations covering both model calibration and systematic parametric studies have been carried out alongside the physical testing. These results are not presented in this paper, as our focus here is on the fundamental response under the most critical loading direction. The complementary findings will be reported in separate publications.

4.7.1. Layered Soil Profiles

The soil used in this study was intentionally chosen as uniform soft and stiff clay layers to isolate the influence of soil stiffness. Natural deposits are rarely homogeneous; they often contain interbedded sand or silt lenses that affect wave propagation and soil–pile interaction. Future work could examine layered profiles with different stiffness and strength combinations to better represent field conditions.

4.7.2. Group-Pile Effects

All tests were performed on single piles, whereas real foundations usually consist of pile groups. In a group configuration, interaction among adjacent piles through the surrounding soil can alter the lateral response, load distribution, and failure modes of individual piles. The influence of pile spacing and cap connectivity would be a logical next step, whether through large-scale table testing or well-calibrated numerical parametric studies.

4.7.3. Liquefiable Soil Conditions

Although the soft clay showed significant stiffness degradation under cyclic loading, it remained in a saturated, undrained state and did not liquefy. In loose saturated sands or silts, liquefaction can drastically reduce lateral resistance and induce large lateral spreading. The response of PCHSS piles under such conditions—especially the effects of excess pore pressure build-up and post-liquefaction deformation—remains largely unexplored and warrants both experimental and numerical investigation.

4.7.4. Varying Superstructure Characteristics

The superstructure was modeled as a rigid mass block with constant mass and stiffness. Actual buildings have more complex dynamic properties, different natural periods, mass distributions, and damping ratios—all of which affect the inertial forces transmitted to the piles. Broader parametric coverage, including flexible superstructures and various mass levels, would help to generalize the design recommendations derived from this work.

4.7.5. Further Numerical Parametric Studies

The calibrated numerical model developed in parallel study provides a solid basis for additional parametric exploration. Future work could extend this model to cover a wider range of soil types, pile geometries, and loading conditions that are difficult to realize on the shaking table. Such studies would complement the experimental evidence presented here and support more broadly applicable design guidance for PCHSS piles under diverse seismic scenarios.

5. Conclusions

Large-scale shaking table tests were conducted to investigate the seismic response and failure mechanisms of PCHSS pile–superstructure systems in soft and stiff clay grounds under strong earthquake excitations. Based on the experimental results and analyses, the following conclusions are drawn:

- (1) Ground stiffness governs the degradation pathway and energy dissipation mode of the system. Soft clay frequency dropped 37.3% ($1.33\times$ that in stiff clay), while stiff clay damping rose 101% ($>3\times$ soft clay's 30.0%). At 0.8 g, stiff clay developed $1.9\times$ the inertial force of soft clay, with fuller hysteresis; soft clay showed saturated force and excessive deformation, indicating plastic flow dissipation. The implication is a fundamental design dichotomy: stiff clay imposes strength-controlled demands, whereas soft clay imposes deformation-controlled demands—two distinct failure paradigms requiring fundamentally different reinforcement strategies.
- (2) Long-period ground motions represent a resonance-driven threat that PGA-based design cannot capture. Under identical PGA, long-period records (CC, EC) produced MCDD and peak moments $2\text{--}3\times$ those from short-period records (SP, WL), due to near-resonance with the pile–soil fundamental period. The practical corollary is that site-specific response spectra—rather than code-specified acceleration coefficients—should govern the flexural design of PCHSS piles in long-period-prone regions.
- (3) Pile–soil interaction is not a static property but an intensity-dependent state variable. The system shifts from compatible deformation ($\text{PGA} \leq 0.1\text{ g}$) to progressive separation (0.8 g), with stiff clay producing higher pressures than soft clay and long-period motions generating $2\text{--}3\times$ higher pressures than short-period ones. This transition challenges conventional linear–elastic interaction models and demands pressure-dependent $p\text{--}y$ formulations for seismic design.
- (4) Failure modes are not intrinsic to the pile but emerge from soil–pile kinematic compatibility. Soft clay shifted the peak moment from 2D (0.05 g) to 4D–6D (0.8 g), confirming flexure-dominated failure with a migrating plastic hinge. Stiff clay limited the shift to 1D–3D with cracks at 0D–5D and exhibited bending moments exceeding the yielding moment within 1D–2.8D at 0.8 g, indicating flexural–shear dominance near the pile head. The implication is that code provisions prescribing a fixed reinforcement zone implicitly assume a stationary plastic hinge—an assumption this study demonstrates to be invalid for soft clay foundations.
- (5) The code-specified 4D reinforcement zone embodies a strength-based philosophy that may be inadequate for soft clay under strong motion. Observed cracks extended

to 4D–7D in soft clay, warranting an extended zone up to 7D. For stiff clay, cracks fell within 0D–5D, yet bending moments exceeded the yielding moment within 1D–2.8D at 0.8 g, indicating that flexural–shear capacity near the pile head—rather than reinforcement length—should be the primary design concern. The recommended 7D extension is not merely a quantitative adjustment—it signals a paradigm shift from “strengthening the anticipated plastic region” to “anticipating the migration of the plastic region.”

- (6) Beyond these specific findings, this study makes four cross-cutting scientific contributions. First, it provides the first experimental evidence that hybrid reinforcement remains effective under true dynamic soil–pile interaction, sustaining 0.8 g without fracture. Second, it quantitatively disentangles soil stiffness and spectral content as independent demand drivers, revealing that spectral shape governs ductility demand more than PGA. Third, it establishes a comprehensive benchmark dataset for calibrating numerical models and data-driven predictors. Fourth, its 1:2-scale methodology preserving full prestress offers a reproducible template for future soil–pile interaction studies.
- (7) The conclusions drawn herein are necessarily bounded by the experimental scope, and their generalization requires cautious extrapolation. The response of PCHSS piles in liquefiable sands, group-pile configurations, layered profiles, and under multidirectional excitation remains unexplored. Future numerical parametric studies and PBEE integration are needed. Nevertheless, the central message is clear: seismic design of hybrid-reinforced piles in clay must embrace intensity-dependent soil–pile interaction, spectral-shape-aware demand characterization, and a damage-containment philosophy that anticipates, rather than prescribes, the location of the critical section.

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Notation

Abbreviations

PHC pile	Prestressed High-strength Concrete Pipe pile
PHS pile	Prestressed High-strength Concrete Hollow Square pile
PCHSS pile	Prestressed Concrete Hybrid-reinforced Solid Square pile
PBEE	Performance-Based Earthquake Engineering
IDA	Incremental Dynamic Analysis
EVA	Ethylene Vinyl Acetate Copolymer
PGA	Peak Ground Acceleration
MCDD	Maximum Curvature Ductility Demand
FSO	Full-Scale Output

Symbols

D	Side length of the PCHSS pile
h_0	Effective depth of the pile section under bending
a_s	Distance from the centroid of the tensile longitudinal reinforcement to the concrete surface.
a'_s	Distance from the centroid of the compresile longitudinal reinforcement to the concrete surface.
A_{st}	Total area of hot-rolled reinforcement in tension zone
A_{sc}	Total area of hot-rolled reinforcement in compression zone
A_{pt}	Total area of prestressing reinforcement in tension zone
A_{pN}	Total area of prestressing reinforcement in neutral axis zone
A_{pc}	Total area of prestressing reinforcement in compression zone
ε_{cc}	Compressive strain at the extreme compression fiber of the section
ε_{ct}	Compressive strain at the extreme tension fiber of the section
ε_{st}	Tensile strain in the hot-rolled reinforcing bars
ε_{sc}	Compressive strain in the hot-rolled reinforcing bars
ε_{pt}	Tensile strain in the prestressing steel
ε_{pN}	Strain in the prestressing steel located at the neutral axis of the section
ε_{pc}	Compressive strain in the prestressing steel
F_{cc}	Resultant compressive force of concrete (in the compression zone)
F_{ct}	Resultant tensile force of concrete (in the tension zone)
F_{st}	Resultant tensile force of hot-rolled reinforcement (in the tension zone)
F_{sc}	Resultant compressive force of hot-rolled reinforcement (in the compression zone)
F_{pt}	Resultant tensile force of prestressing steel (in the tension zone)
F_{pN}	Resultant force of prestressing steel located at the neutral axis
F_{pc}	Resultant compressive force of prestressing steel (in the compression zone)
$\varnothing$	Curvature of the section
$\varnothing_y$	Yield curvature of the section
$\varnothing_{max}$	Maximum curvature of the section
$\mu\varnothing$	Maximum Curvature Ductility Demand (MCDD)
f	Natura frequency of pile system

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