

Article

Numerical Investigation of the Lateral Loading Behaviour of Plate–Monopile Hybrid Foundations in Clay

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Abstract

Plate–monopile hybrid foundations, as a potential alternative to monopiles, have demonstrated promising potential in enhancing load-bearing capacity and structural stability. To investigate its load transfer mechanisms and pile–soil interaction in clay, numerical models are developed under varying undrained shear strength (S_u), pile diameter (D), and plate-to-pile diameter ratio (R). Through comparative analyses within different parameters configurations, the load-bearing capacity, pile deflection, bending moment and shear force distributions are systematically examined. The results indicate that: (1) S_u , D and R are all positively correlated with the load-bearing capacity of the hybrid foundation, which can be expressed as the superposition of the monopile capacity and a quadratic function of R ; (2) with increasing R , load transfer shifts from deep to shallow soil, accompanied by an upward pivot shift; increasing D causes a downward shift, more pronounced in weak soils; (3) for small-diameter hybrid foundation, the bending moment decreases progressively with increasing R , while for large-diameter, a stage-dependent response is observed, characterised by local moment concentration near the mudline within a certain range of R ; (4) the shear force exhibits a double-peak pattern; increasing R strengthens the shallow peak and weakens the deep one, while increasing D localises the distribution near the mudline.

Keywords: hybrid foundation; plate; monopile; bearing capacity; soil-pile interaction

1. Introduction

Offshore wind energy has become a cornerstone of the global transition towards low-carbon and sustainable energy systems and has experienced rapid development in recent years [1–4]. To meet the increasing demand for clean energy, the capacity of individual wind turbines has increased from early values of 3 MW to the current 15 MW, accompanied by a growth in blade length from 60 m to 110 m [5–8]. Meanwhile, the number and scale of wind farms continue to expand. These developments subject wind turbines to larger environmental loads and more complex geological conditions during operation, thereby imposing more stringent requirements on their structural performance [9–11]. As the primary load-bearing structure responsible for maintaining the overall stability of wind turbines, the foundation system plays a crucial role in ensuring the safety and economic efficiency of offshore wind energy systems. Therefore, the design of safe, reliable,

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and efficient foundation systems is essential for the stable operation of offshore wind farms. At present, monopile foundations are widely adopted in offshore wind energy due to their structural simplicity, construction convenience, and mature engineering applications [12,13]. However, with the increasing requirements for foundation stability and load-bearing capacity, the diameter of monopiles has continued to increase [14–16]. While large-diameter monopiles enhance load-bearing capacity, they also give rise to a series of challenges. On the one hand, fabrication, transportation, and offshore installation become more technically demanding and involve elevated construction risks. On the other hand, the substantial increase in size results in higher material consumption and significantly increased costs, thereby reducing their economic efficiency. Accordingly, the development of alternative foundation solutions that preserve structural simplicity while reducing construction difficulty and maintaining economic viability has become an important research direction in offshore wind engineering.

In recent years, the concept of hybrid foundations has been progressively proposed. Various types of hybrid foundation have been developed based on monopile, including plate–monopile [17–19], bucket–monopile [20–22], wing–monopile [23–25] and helical–monopile configurations [26–28], etc. These systems are typically formed by integrating additional structural components with the monopile to enhance load-bearing capacity and improve overall stability, thereby reducing the reliance on large-diameter monopiles. Among the various types of hybrid foundations, the plate–monopile has attracted considerable attention in recent years due to its favourable overall performance in balancing structural complexity, constructability, and economic efficiency. Compared with other hybrid configurations, the plate–monopile can effectively enhance foundation performance by incorporating relatively simple auxiliary components, without significantly increasing structural complexity or construction difficulty. In addition, as the plate is typically installed at the mudline level without requiring embedment into the seabed, the engineering adaptability is enhanced, enabling more flexible design and application under different geological conditions.

Recent studies have been conducted to investigate the load-bearing advantages of plate–monopile hybrid foundations. Ma et al. [17] employed numerical simulations to evaluate the bearing performance of plate–monopile hybrid foundations and showed that, under sand conditions with a relative density of 50%, such foundations exhibit a significant advantage in reducing the required embedded depth of the monopile. Aleem et al. [26] conducted 1 g model tests on plate–monopile hybrid foundations with different geometric configurations in Red Hill 110 silica sand, demonstrating that the hybrid system can effectively restrain pile head rotation. Li et al. [20] combined 1 g model tests with numerical simulations to analyse the migration characteristics of sand particles around hybrid foundations under cyclic lateral loading. Lehané et al. [27] carried out centrifuge tests and numerical simulations in sand to compare the mechanical performance of hybrid foundations and monopiles, and found that the inclusion of a plate significantly increases the seabed contact area and enhances the overall bending stiffness. Wang et al. [28] performed centrifuge tests in clay, showing that the introduction of a plate leads to a significant increase in stiffness and load-bearing capacity compared with monopiles, but the bearing capacity is lower than the sum of the individual capacities of the monopile and the plate.

Overall, existing studies on plate–monopile hybrid foundations have mainly focused on sand conditions and have achieved relatively comprehensive research progress. Numerical simulations, 1 g model tests, and centrifuge tests have demonstrated that the introduction of a plate can effectively enhance the lateral bearing capacity, reduce pile head rotation, improve overall stiffness, and mitigate deformation under both monotonic and cyclic loading. These improvements are mainly attributed to the enlarged seabed contact area, enhanced soil confinement, increased frictional resistance, and more favourable load

transfer mechanisms. Furthermore, several studies have investigated soil particle migration and stress distribution, providing valuable insights into the interaction between the foundation and surrounding sand. Compared with sand, studies on plate–monopile hybrid foundations in clay remain very limited [29–31]. Existing research has mainly evaluated the overall bearing capacity and stiffness through centrifuge tests, demonstrating that the inclusion of a plate can significantly improve foundation performance. However, owing to the fundamentally different mechanical behaviour of clay, the load-bearing mechanism is governed primarily by undrained shear resistance and stress redistribution rather than particle friction and rearrangement. Consequently, the pile–soil interaction, load transfer path, and collaborative mechanism between the plate and monopile differ substantially from those in sand. Therefore, conclusions obtained under sand conditions cannot be directly extended to clay. However, systematic investigations into the pile–soil interaction, internal force transfer, and load redistribution mechanisms within plate–monopile hybrid foundations in clay remain insufficient. This limited understanding restricts both the interpretation of the working mechanism and the optimisation of foundation design under complex offshore clay conditions.

In this study, numerical models of plate–monopile hybrid foundations and monopiles are established in clay under different undrained shear strengths (S_u). A systematic comparative analysis of the load-bearing performance of hybrid foundations is conducted under varying pile diameters (D) and plate-to-pile diameter ratios (R). Furthermore, the pile–soil interaction characteristics and the mechanisms of load transfer and redistribution within the foundation system are investigated. The study provides a useful reference for understanding the load-bearing mechanisms and for the design optimisation of plate–monopile hybrid foundations in clay.

2. Numerical Methodology

2.1. Constitutive Model

The Norwegian Geotechnical Institute–Anisotropic Deformation Plasticity (NGI–ADP) yield criterion is an anisotropic elastoplastic constitutive model developed based on the conventional Tresca yield criterion and is specifically designed to describe the undrained mechanical behaviour of clay [32–34]. As illustrated in Figure 1, the NGI–ADP model employs an anisotropic yield surface in the π -plane. Unlike the conventional Tresca yield criterion, which assumes a uniform yield strength in all loading directions, the NGI–ADP model assigns different undrained shear strengths to different loading directions. Consequently, the yield surface exhibits direction-dependent strength characteristics under different stress paths, enabling the model to capture the inherent anisotropy of natural clays arising from their depositional history and subsequent consolidation processes.

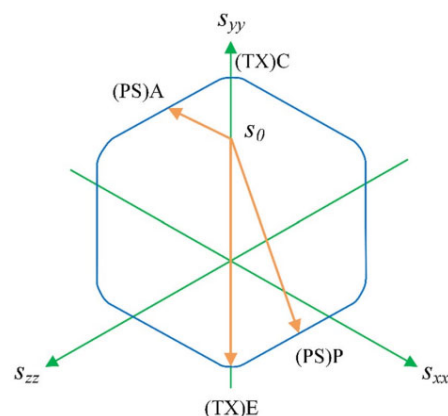


Figure 1. Failure criterion of the NGI–ADP model in the π plane [35].

Prior to yielding, the NGI–ADP model adopts an isotropic linear elastic assumption to describe the elastic response of the soil, in which the elastic shear modulus and Poisson's ratio are assumed to be identical in all directions, thereby neglecting the directional dependence of the elastic properties. Consequently, the anisotropy incorporated in the NGI–ADP model is manifested in the plastic response rather than the elastic response. During the plastic stage, the model defines the undrained shear strengths corresponding to three representative loading modes, namely Active, Direct Simple Shear, and Passive, which are used to construct a direction-dependent anisotropic yield surface and characterise the plastic response under different stress paths. Specifically, the undrained shear strength is expressed as a continuous function of the Lode angle in the π -plane, allowing a smooth transition of shear strength between different loading paths. This formulation provides a unified description of the nonlinear mechanical response under typical loading conditions, including triaxial compression, triaxial extension, and Direct Simple Shear. Compared with conventional isotropic constitutive models, the NGI–ADP model more realistically captures the inherent strength anisotropy of natural clays and improves the prediction of stress–strain behaviour and deformation characteristics under complex stress paths.

The NGI–ADP model is particularly suitable for the numerical analysis of laterally loaded pile foundations because the surrounding soil is subjected to multiple stress paths during loading rather than remaining in a single stress state. In general, the soil behind the pile predominantly experiences a triaxial compression stress state, whereas the shallow soil in front of the pile is mainly subjected to triaxial extension. Near the pile rotation point, the soil response is dominated by a Direct Simple Shear stress state. At greater depths, however, the combined effects of multiple stress paths result in a more complex stress response, as illustrated in Figure 2. By assigning distinct undrained shear strengths to these representative loading modes, the NGI–ADP model is capable of capturing the direction-dependent strength characteristics of the surrounding soil, thereby providing a more accurate description of the deformation behaviour and strength evolution of the soil around laterally loaded piles.

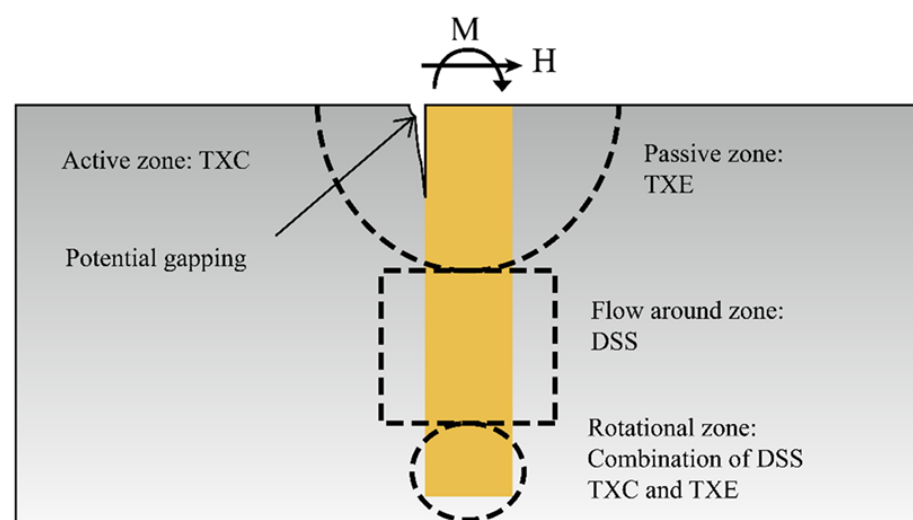


Figure 2. Stress path of the soil surrounding the pile [36]: Triaxial Compression (TXC), Triaxial Extension (TXE), Direct Simple Shear (DSS).

The input parameters required for the NGI–ADP constitutive model can be broadly classified into two categories: stiffness parameters and strength parameters. The stiffness-related parameters are used to characterise the elastic response of the soil and its deformation behaviour under different stress paths. These parameters include the ratio of the

unload/reload shear modulus to the Active undrained shear strength (G_{ur}/S_U^A), Poisson's ratio (ν), and the shear strains corresponding to the peak strength under triaxial compression, triaxial extension, and Direct Simple Shear, denoted by γ_f^C , γ_f^E and γ_f^{DSS} , respectively. Among these parameters, G_{ur}/S_U^A and ν govern the initial elastic stiffness of the soil, whereas the characteristic shear strains control the evolution of the stress–strain response and the development of plastic deformation under different loading modes.

The strength parameters are primarily used to define the yield characteristics and the anisotropic strength distribution of the soil. These parameters include the ratio of the Passive to Active undrained shear strength (S_U^P/S_U^A), the ratio of the Direct Simple Shear to Active undrained shear strength (S_U^{DSS}/S_U^A), the initial mobilisation ratio (τ_0/S_U^A), the ratio of the triaxial compression strength to the Active undrained shear strength ($S_U^{C,TX}/S_U^A$), and the soil unit weight (γ). Among these parameters, (S_U^P/S_U^A) and (S_U^{DSS}/S_U^A) play a key role in defining the strength anisotropy of the soil by specifying the differences in undrained shear strength associated with different loading directions. When the undrained shear strengths are identical in all loading directions, the yield surface reduces to an isotropic form. As the differences between these directional strength ratios increase, the degree of anisotropy becomes more pronounced, enabling the model to more realistically represent the inherent directional strength characteristics of naturally deposited clays.

Furthermore, to ensure a smooth transition of the mechanical response between different stress paths, the NGI–ADP model employs an elliptic interpolation scheme to continuously interpolate the characteristic shear strains between the Active, Direct Simple Shear, and Passive loading modes. This approach eliminates abrupt changes in the constitutive response between different loading conditions and enhances the model's capability to simulate complex stress paths. In addition, the model is capable of accounting for non-homogeneous clay deposits by assuming that the undrained shear strengths corresponding to the Active, DSS, and Passive loading modes increase linearly with depth. This assumption enables the model to realistically represent the strength profile of marine soft clays, in which the undrained shear strength generally increases with depth, thereby improving its applicability to offshore soft clay deposits.

This study considers three representative clay types: soft clay, medium clay, and stiff clay. To characterise the mechanical properties of clays with different strength levels and enable a quantitative comparison, the undrained shear strength is adopted as the key controlling parameter, with values of 30 kPa, 60 kPa, and 90 kPa assigned to represent different clay strength categories. During the comparative analysis, all other soil parameters are kept constant. The detailed soil properties are listed in Table 1. It should be noted that this approach represents a simplified assumption, as in reality, soil properties vary with depth and multiple parameters may change simultaneously. Therefore, the results are intended to provide a comparative reference rather than site-specific predictions.

Table 1. NGI–ADP constitutive model parameters.

Parameter	Description	Unit	Value
γ'	Soil submerged unit weight	kN/m ³	8.5
K_0	Lateral earth pressure coefficient	-	1
$S_U^{A,ref}$	Undrained shear strength	kPa	30, 60, 90
$S_U^{A,inc}$	Variation in undrained shear strength with depth	kPa/m	3.1
G_{ur}/S_U^A	Ratio of soil shear modulus to S_U^A	-	600
$S_U^{C,TX}/S_U^A$	Ratio of undrained shear strength in triaxial compression to S_U^A	-	0.99
S_U^P/S_U^A	Ratio of undrained shear strength in triaxial extension to S_U^A	-	0.5

S_U^{DSS}/S_U^A	Ratio of undrained shear strength in Direct Simple Shear to S_U^A	-	0.75
τ_0/S_U^A	Ratio of Initial in situ maximum shear stress to S_U^A	-	0
γ_f^C	Ultimate plastic strain for compression	%	10
γ_f^E	Ultimate plastic strain for extension	%	20
γ_f^{DSS}	Ultimate plastic strain for direct shear	%	15

2.2. Finite Element Model

The FEM models are developed using Plaxis 3D 2025.1. It consists of two main components: the soil domain and the foundation structure. In previous monopile studies, the soil domain is typically required to extend at least ten times the pile diameter to minimise boundary effects [37–40]. However, for plate–monopile hybrid foundations, the plate dimension also significantly influences boundary effects. Therefore, considering the maximum plate size in this study, the soil domain is uniformly defined as $300 \times 300 \times 70$ (length $\times$ width $\times$ height). This configuration ensures that, within the parameter range considered, the soil domain dimensions are no less than ten times the plate diameter, and the depth direction satisfies the requirement of being larger than twice the embedment depth, thereby effectively minimising boundary effects on the simulation results. The top surface of the soil domain is defined as a free boundary, while the lateral boundaries are constrained in the horizontal directions, and the bottom boundary is fully fixed. The plate–monopile hybrid foundation model consists of two components: a monopile with a wall thickness of 0.07 m and a plate with a thickness of 0.2 m. Both components are modelled using steel, with a Young's modulus of 210 GPa and a unit weight of 78.5 kN/m³, as shown in Figure 3.

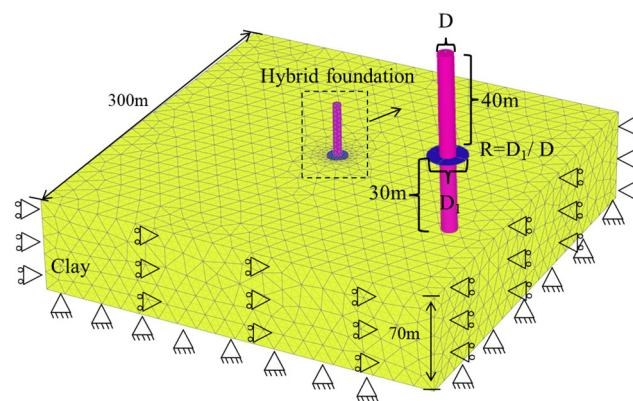


Figure 3. Hybrid foundation FEM model.

The monopile is modelled using linear elastic solid elements, with material properties determined based on the principles of equivalent stiffness and equivalent density [41]. The corresponding formulations are provided in Equations (1) and (2). In this study, monopiles with diameters of 4 m, 6 m, and 8 m are considered, and their equivalent density and stiffness parameters are summarised in Table 2. This simplification neglects the soil plugging effect within the monopile. Its validity has been confirmed in the study by Achmus et al. [42]. Furthermore, Page et al. [43], Gao [44] and Li et al. [45] have verified the applicability of this approach for monopiles with diameters ranging from 4 m to 9 m. The embedded depth of the hybrid foundation is set to 30 m, with a structural height of 40 m above the mudline. The lateral load is applied at a height of 40 m above the mudline.

$$E_{eq} = \frac{64E_p I_p}{\pi D^4} \quad (1)$$

$$\rho_{eq} = \frac{A\rho}{A_{eq}} \quad (2)$$

where E_{eq} is the equivalent flexural stiffness; $E_p I_p$ is the original flexural stiffness; D is the structure diameter; ρ_{eq} is the equivalent density; A is the original structure cross-section area; ρ is the original density; A_{eq} is the equivalent structure cross-section area.

Table 2. Equivalent parameters of monopile.

D	Equivalent Density	Equivalent Stiffness
4 m	549.43 kg/m ³	27.89 GPa
6 m	369.38 kg/m ³	18.92 GPa
8 m	277.26 kg/m ³	14.31 GPa

To accurately capture potential separation and slippage at the pile–soil interface, interface elements are introduced between the monopile and the surrounding soil, as well as between the plate and the soil. To facilitate tensile cut-off, the interface is modelled using the Mohr–Coulomb model. The stiffness parameters of the interface are determined based on the stiffness characteristics of the adjacent soil, whereas the interface strength parameters are reduced according to the material combination at the steel–clay interface. Following the recommendation for steel–clay interfaces provided in [46,47], an interface strength reduction factor of 0.65 is adopted, such that the interface shear strength is taken as 65% of the corresponding strength of the adjacent soil. It should be noted that the reduction factor reflects the effects of the contacting material combination and interface roughness on the interface strength; therefore, different reduction factors may be adopted for different material combinations. To achieve an optimal balance between computational accuracy and efficiency, a local mesh refinement strategy is adopted. Centred on the model, a refined zone with a radius of 1.5 times the plate diameter is defined, while the remaining domain is assigned a standard mesh density. A refinement factor of 0.5 is applied within the refined region. Two representative foundation configurations are selected to perform the mesh sensitivity analysis for the standard density region. The first case consists of a monopile diameter of 6 m and a plate diameter of 15 m, while the second case is only a monopile with a 6 m diameter. Four mesh densities, coarse, medium, fine, and very fine, are considered. By applying loads to the monopile and comparing the resulting deformation at the mudline, it is observed that the difference between the fine and very fine mesh results for these two cases is within an acceptable range (less than 4%). Therefore, the fine mesh is adopted for the standard density region in subsequent analyses.

2.3. Verification

Before conducting the numerical simulations, the modelling approach and constitutive model are validated. The CL2 pile test conducted at the Cowden site during the PISA project is selected as the benchmark case. The CL2 pile has a diameter of 2 m, an embedded length of 10.6 m, and a wall thickness of 25 mm. The lateral load is applied at a height of 10.9 m above the mudline level. The ground profile consists of approximately 40 m of over-consolidated clay till underlain by chalk, with the groundwater table located 1 m below the ground surface. The saturated unit weight of the soil is taken as 21.19 kN/m³. Based on the site investigation, the variation with depth of the small-strain shear modulus, G_0 , the undrained shear strength in compression, S_u , the lateral earth pressure coefficient in terms of effective stress, K_0 , and the measured pore water pressure, u , are shown in Figure 4.

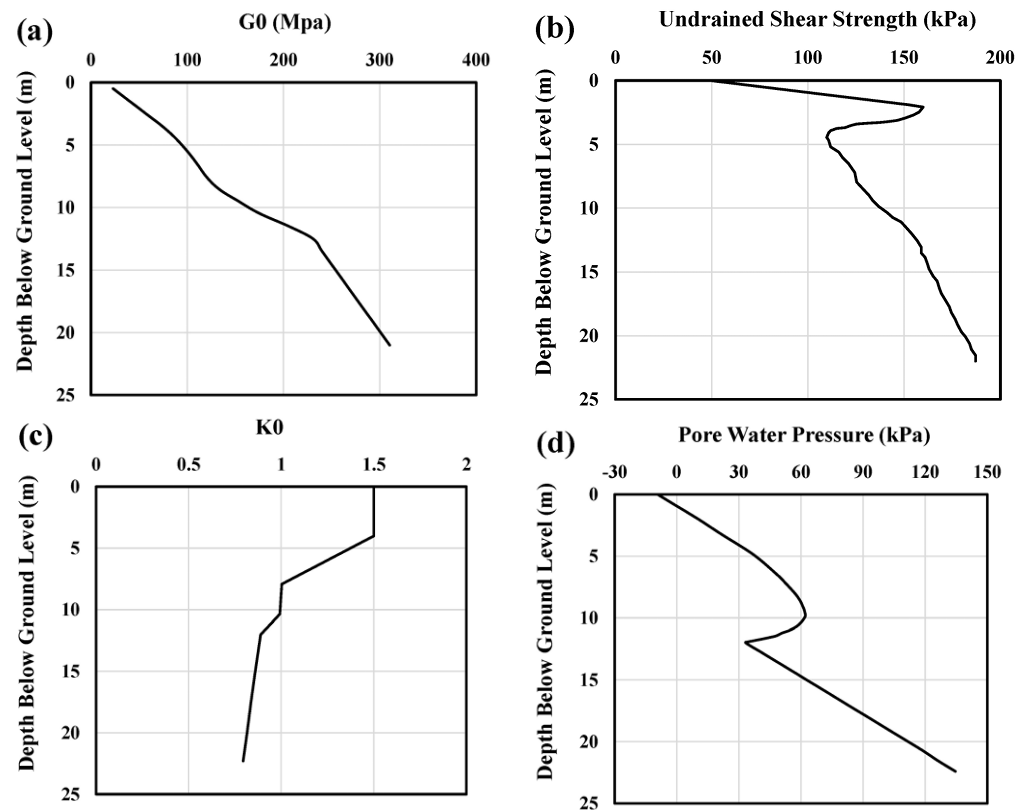


Figure 4. Cowden site soil parameters: (a) G_0 ; (b) S_u ; (c) K_0 ; (d) u [48].

The validation model is established with dimensions of 30 m × 30 m × 20 m (length × width × depth). To accurately reproduce the field test results, the soil profile is divided into ten layers with distinct mechanical properties based on the measured geological data shown in Figure 4. The undrained shear strength S_u^A is assumed to vary linearly with depth (z) as it is given by:

$$S_u^A = 5.2 + 3.1z \quad (3)$$

where z is the depth. Thus the $S_{u,inc}^A$ is 3.1 kPa/m. In each layer, the $S_u^{A,ref}$ can be calculated using

$$S_{u,ref}^A = S_u^{top} \quad (4)$$

where S_u^{top} is the undrained shear strength at the top of each soil layer. G_{ur}/S_u^A is defined by:

$$G_{ur}/S_u^A = G_0 / ((S_u^{top} + S_u^{bottom})/2) \quad (5)$$

where S_u^{bottom} is the undrained shear strength at the bottom of each soil layer. G_0 is the soil shear modulus, which is taken as the average value for each soil layer. S_u^P/S_u^A is taken as a constant value of 0.5, while S_u^{DSS}/S_u^A is defined by

$$S_u^{DSS}/S_u^A = \frac{1 + S_u^P/S_u^A}{2} \quad (6)$$

The τ_0/S_u^A is obtained using:

$$\tau_0/S_u^A = 0.5(1 - K_0)\sigma'_{v0}/S_u^A \geq 0 \quad (7)$$

where τ_0/S_u^A represents the initial vertical effective stress. The relationship between the ultimate plastic strains for compression, extension, and direct shear is defined as follows:

$$\gamma_f^c = \frac{6000}{G_{ur}/S_u^A} \quad (8)$$

$$\gamma_f^C = \gamma_f^E / 2 = \gamma_f^{DSS} / 1.5 \quad (9)$$

Therefore, the NGI–ADP model parameters assigned to each layer are listed in Table 3.

Table 3. The NGI–ADP model parameters for the verification model.

Layer	z^{top}	z^{bottom}	K_0	σ'_{v0}	S_u^{top}	S_u^{bottom}	G_0	G_{ur}/S_u^A	S_u^A, ref	S_u^A, inc	γ_f^C	γ_f^E	γ_f^{DSS}	τ_0/S_u^A
1	0	2	1.5	20.5	20	130	25	333.3	20	55	18	36	27	0
2	2	4	1.5	44	130	110	65.5	545.8	130	−10	10.9	21.9	16.4	0
3	4	6	1.5	68.5	110	118.8	94.1	822.5	110	4.4	7.2	14.5	10.9	0
4	6	8	1.12	97	118.8	127	114.7	933.2	118.8	4.1	6.4	12.8	9.6	0
5	8	10	1	130.5	127	138.3	142.7	1075.7	127	5.65	5.5	11.1	8.3	0
6	10	12	0.95	180	138.30	154.9	190.55	1299.7	138.3	8.3	4.6	9.2	6.9	0.032
7	12	14	0.87	232.5	154.9	161.4	235.35	1488.1	154.9	3.25	4.0	8.0	6.0	0.097
8	14	16	0.86	256	161.4	167.8	258	1567.4	161.4	3.2	3.8	7.6	5.7	0.111
9	16	18	0.84	279	167.8	174.1	276.8	1619.1	167.8	3.15	3.7	7.4	5.5	0.133
10	18	20		301	174.1	181.4	293	1648.3	174.1	3.65	3.6	7.2	5.4	0.155

As shown in Figure 5, the numerical results are generally in good agreement with both the FEM results and the PISA field test data in terms of the load–displacement response. Both the initial stiffness and the overall nonlinear behaviour are well captured. A slight overestimation of the lateral resistance is observed at large displacements (approximately 150–200 mm). This discrepancy may be attributed to the discretised representation of the continuously varying soil profile, the idealised modelling assumptions adopted in the numerical model, and the increased sensitivity of soil behaviour under large deformation. In addition, the PISA field test may also be influenced by construction disturbance, loading eccentricity, natural spatial variability of the soil, and measurement uncertainties, which may further contribute to the observed differences. Nevertheless, compared with the previously published PISA FEM results, the present model follows the field-test trend more closely over most of the displacement range. Therefore, the proposed numerical model is considered capable of reliably reproducing the overall soil–structure interaction behaviour observed in practice, thereby validating its applicability for the subsequent analyses.

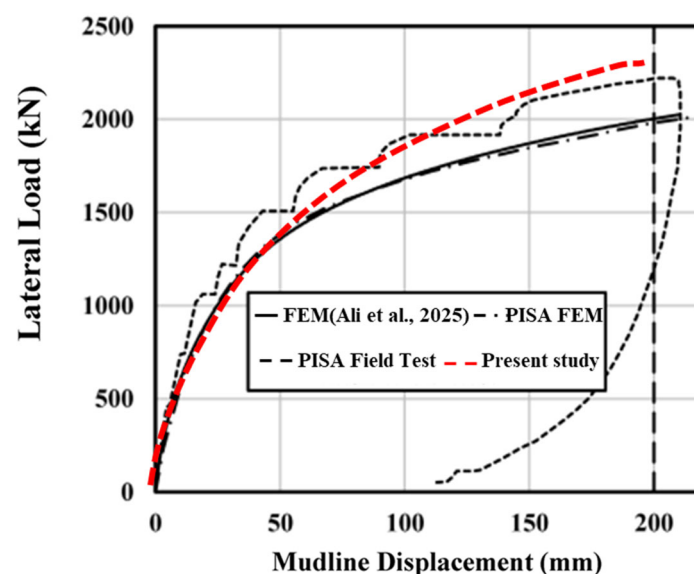


Figure 5. Comparison of the verification results and reference data [46].

It should be noted that the numerical modelling strategy for the plate–monopile hybrid foundation has been validated against experimental results under sand conditions in our previous study [15,17]. Therefore, the present validation primarily aims to verify the applicability of the adopted NGI–ADP constitutive model and the numerical modelling framework for marine clay. Together with the previous validation of the hybrid-foundation modelling approach, these results provide confidence in the proposed numerical methodology for analysing plate–monopile hybrid foundations in clay.

2.4. Simulation Schemes

The undrained shear strength, monopile diameter, and plate–pile diameter ratio are selected as the key influencing parameters in this study. Two groups of numerical simulation schemes are established, comprising a total of 108 cases, as summarised in Table 4. The first scheme is designed to evaluate the bearing capacity of the hybrid foundation under different parameter combination, thereby enabling an analysis of the influence mechanisms of these parameters and their quantitative relationships with the bearing capacity. Based on the obtained bearing capacity for each case, the second scheme applies the corresponding load to each model to bring it to its limit state, with the aim of investigating the effects of different parameters on the pile–soil interaction behaviour under this condition. It should be noted that a homogeneous clay profile was assumed in the present study to isolate the effects of representative soil strength and foundation geometry. In natural marine clay deposits, the undrained shear strength generally increases with depth. Such a strength gradient may modify the stress distribution around the foundation, alter the load transfer mechanism between the monopile and the plate, and consequently influence the bearing capacity, deformation response, and load-sharing behaviour of the hybrid foundation. Therefore, the results should be interpreted within the assumptions adopted in this study. A systematic investigation of different strength gradients will be considered in future work.

Table 4. FEM schemes.

	S_u	D	R	Load
I	30	4	0 (Monopile), 1.5, 2, 2.5, 3, 3.5	15 MN
		6		
		8		
	60	4		20 MN
		6		
		8		
	90	4		30 MN
		6		
		8		
	30, 60, 90	4, 6, 8		Corresponding bearing capacity

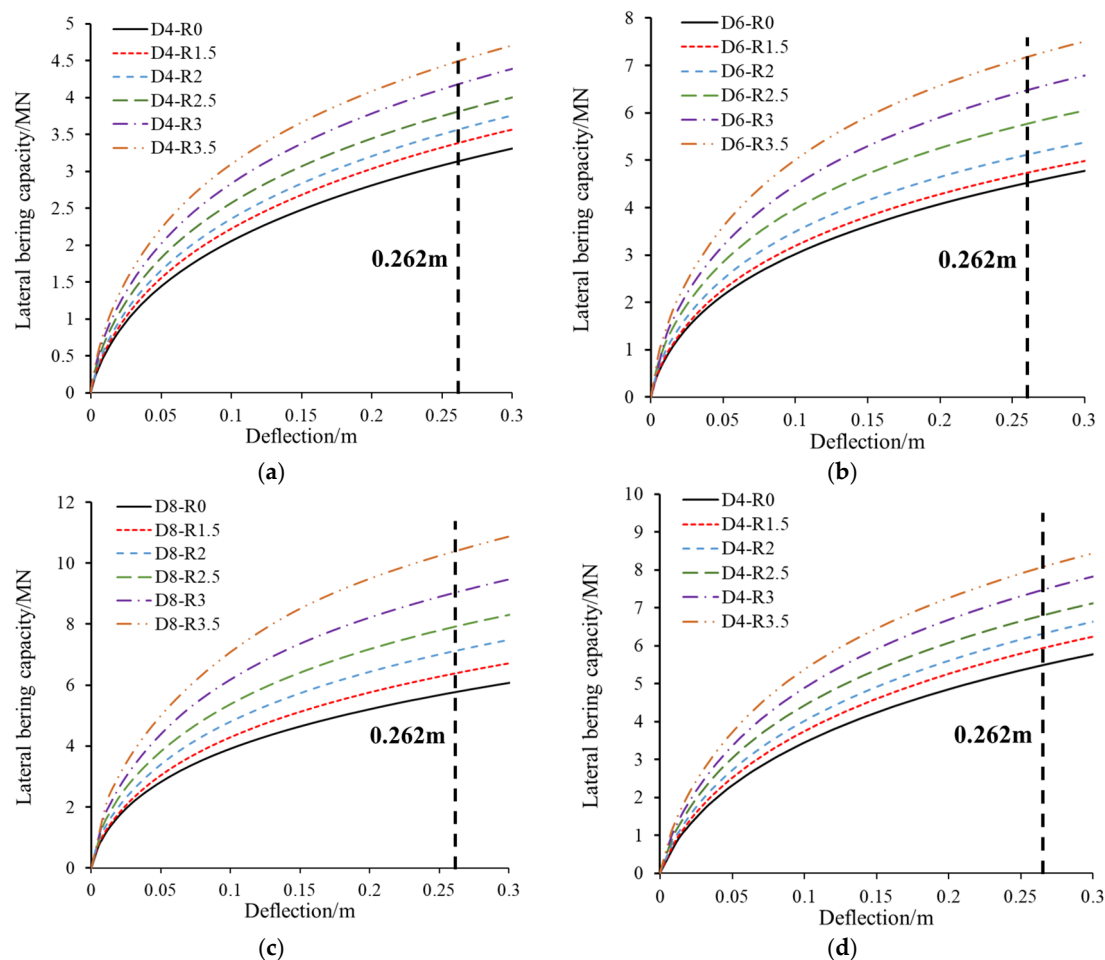
3. Results Analysis

3.1. Lateral Bearing Capacity

The bearing capacity is evaluated based on the Serviceability Limit State (SLS). According to DNV-GL-ST-0126 (2016) [49], the mudline rotation is limited to 0.5° . Assuming the rotation centre at the pile toe, the corresponding lateral deflection at the mudline is derived from the embedment depth, consistent with the approach in Achmus et al. [42] and Bhattacharya et al. [50]. For an embedded depth of 30 m, the maximum allowable mudline deflection is 0.262 m. It should be noted that the pile tip is adopted as a reference rotation centre solely for the definition of the SLS criterion, providing a unified basis for comparing

different foundation configurations, rather than representing the actual pivot point during loading.

Figure 6 presents the load–deflection responses of the foundation under different undrained shear strengths, pile diameters, and plate–pile diameter ratios. It can be observed that the overall shapes of the curves are similar across all cases. In the initial loading stage, the curves exhibit a relatively steep slope, indicating that the deformation increases slowly with increasing load and that the system possesses a high initial stiffness. As the load increases further, the curves gradually flatten and the rate of deformation accelerates, suggesting a reduction in system stiffness and the progressive development of soil nonlinearity. Under different S_u conditions, both the R and the D show a positive correlation with the bearing capacity. This can be attributed to the increase in the plate diameter, which enlarges the plate–soil contact area and mobilises a larger volume of surrounding soil, thereby enhancing the pile–soil interaction. Meanwhile, the plate structure is able to carry a larger proportion of the applied load, effectively reducing the rotation at the mudline and improving overall stability. In addition, increasing the pile diameter not only enhances the structural bending stiffness but also strengthens the confinement effect of the surrounding soil, leading to a significant improvement in the overall load-bearing capacity. Furthermore, the bearing capacity also increases with increasing undrained shear strength. This is because a higher S_u directly enhances the shear resistance of the soil, enabling the surrounding soil to provide larger lateral resistance. As a result, the onset of soil yielding and the development of plastic zones are delayed, contributing to an increase in the overall capacity of the foundation.



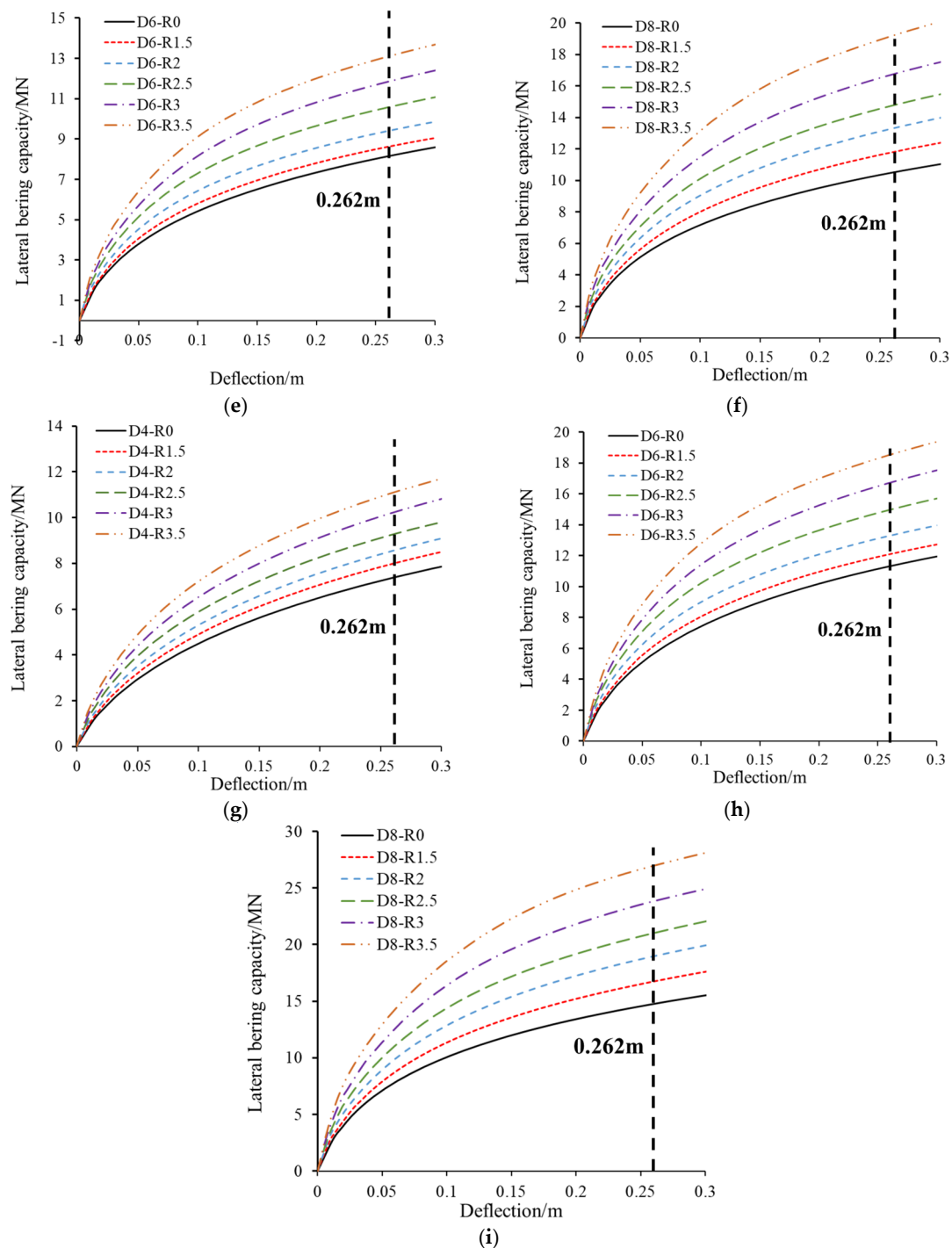


Figure 6. Load–deflection curves: (a) $S_u = 30$ kPa, $D = 4$ m; (b) $S_u = 60$ kPa, $D = 4$ m; (c) $S_u = 90$ kPa, $D = 4$ m; (d) $S_u = 30$ kPa, $D = 6$ m; (e) $S_u = 60$ kPa, $D = 6$ m; (f) $S_u = 90$ kPa, $D = 6$ m; (g) $S_u = 30$ kPa, $D = 8$ m; (h) $S_u = 60$ kPa, $D = 8$ m; (i) $S_u = 90$ kPa, $D = 8$ m (D4-R0 denotes the D is 4 m, the R is 0).

The bearing capacity calculation methods for monopiles have been extensively studied. On the basis of monopile bearing capacity calculations, this paper further investigates the composition and prediction methods of the bearing capacity of hybrid foundations. Considering that a hybrid foundation consists of a monopile and a plate, its overall bearing capacity can be reasonably decomposed into two components: one arising from the load-bearing capacity of the monopile itself, and the other from the additional contribution of the plate structure to the surrounding soil. Accordingly, to quantitatively evaluate the effect of the plate, the total bearing capacity of the hybrid foundation is reduced by the corresponding monopile bearing capacity. The resulting difference is defined as the

additional bearing capacity, which can be interpreted as the additional load-bearing capacity provided by the plate. Figure 7 illustrates the variation in the additional bearing capacity under different parameter combinations. The results indicate that the additional bearing capacity exhibits a pronounced nonlinear increasing trend with the increase in the plate-to-pile diameter ratio. Through curve fitting of the data under various conditions, it is found that the relationship between the additional bearing capacity and R can be well described by a quadratic function. This nonlinear growth primarily originates from the geometric scaling effect of the plate. As R increases, the effective area of the plate grows with the square of its characteristic dimension, thereby mobilising a larger volume of surrounding soil in the load transfer process. This enhanced soil mobilisation strengthens the pile–soil interaction and improves the overall bearing capacity of the system. Since the rate of increase in area exceeds the linear variation in R , the soil reaction and the additional resistance provided by the plate exhibit an accelerated growth trend, ultimately resulting in a pronounced nonlinear increase in the additional bearing capacity.

Based on the above analysis, a predictive expression for the bearing capacity of the hybrid foundation is proposed, as shown in Equation (10), in which the total bearing capacity is formulated as the sum of the monopile capacity and the additional contribution provided by the plate. It should be noted that this decomposition assumes the superposition of the two contributions at the macroscopic level. Although a certain degree of coupling exists between the monopile and the plate, such interaction effects can be reasonably captured by introducing a fitting function based on R . For the contribution provided by plate, only its numerical superposition with the monopile bearing capacity is considered, i.e., while the monopile capacity has a unit of MN, the value obtained from the quadratic function of R is inherently dimensionless; therefore, it is empirically assigned a consistent unit in the model to ensure dimensional compatibility. Meanwhile, the effects of the S_u and the D on the magnitude of the additional contribution are implicitly embedded in the fitted parameters. The relationship between the fitting parameters and the R and S_u as illustrated in Figure 8.

$$F_{Hybrid} = F_{Monopile} + aR^2 + bR + c \quad (10)$$

where F_{Hybrid} and $F_{Monopile}$ denote the bearing capacity of plate–monopile hybrid foundation and monopile, respectively, under SLS. Parameters a , b and c are fitting coefficients.

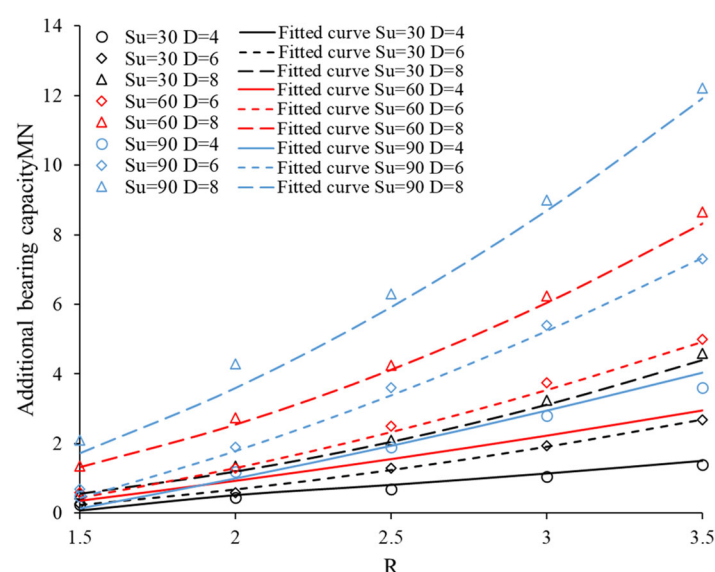


Figure 7. Additional bearing capacity.

It should be noted that the proposed empirical equation does not explicitly include interaction terms between the monopile and the plate. The coupling effect is implicitly embedded in the fitting coefficients because the regression database is generated from numerical simulations considering the complete soil–foundation interaction. Although the proposed equation does not explicitly describe the coupling mechanism between the monopile and the plate, it still provides a useful quantitative reference for engineering applications within the investigated parameter range. Furthermore, the proposed empirical equation is established based on the assumptions adopted in the present study, including a homogeneous clay layer, idealised soil properties, and static loading conditions. Therefore, it is primarily applicable to the preliminary prediction of the ultimate lateral bearing capacity of plate–monopile hybrid foundations under the conditions considered in this study and should be interpreted within the framework of these assumptions and the investigated parameter range. For layered soil profiles or other more complex engineering conditions, the soil–foundation interaction mechanism may differ from that considered herein. Therefore, the proposed empirical equation should be applied with caution under such conditions and in conjunction with site-specific engineering judgement. Future work will incorporate more realistic soil profiles and more complex loading conditions to further improve the applicability of the proposed prediction model.

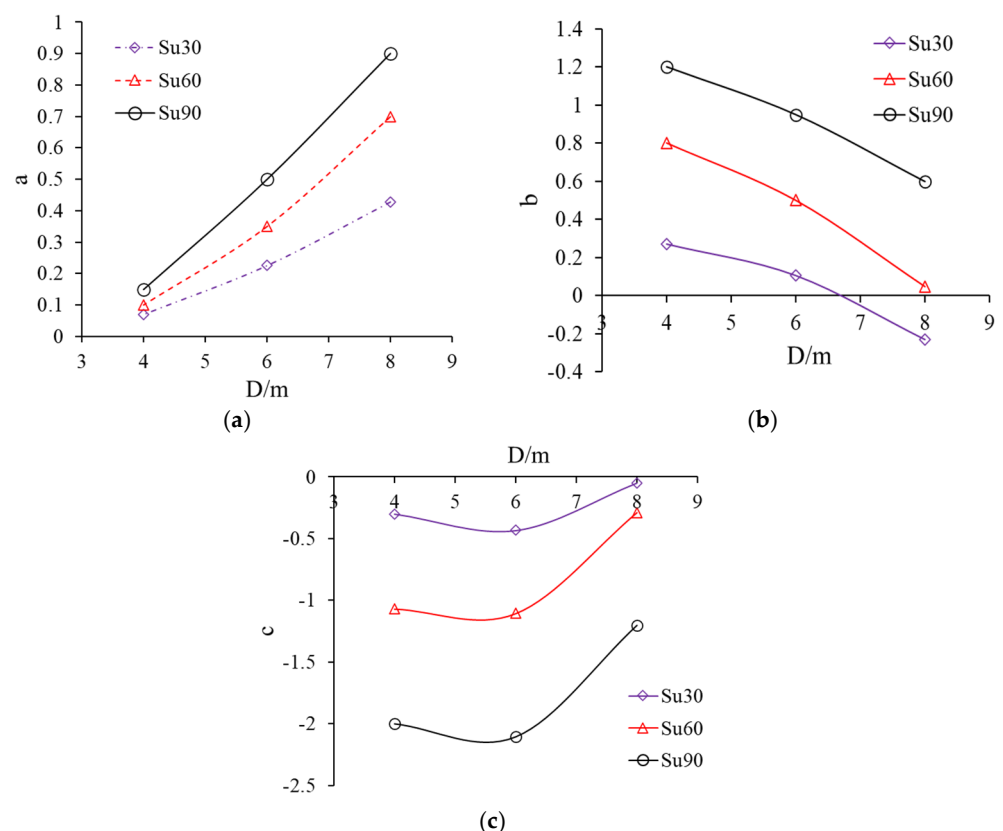


Figure 8. Fitting parameter curves: (a) fitting parameter a ; (b) fitting parameter b ; (c) fitting parameter c .

In the process of engineering design, the extent to which material properties are effectively mobilised plays a critical role in determining the structural feasibility. Therefore, it is essential to quantitatively assess the contribution of unit material area to the enhancement of bearing capacity. Accordingly, a unit material bearing capacity contribution index, denoted as γ , is introduced, as expressed in Equation (11), representing the ratio of the additional bearing capacity, $F_{\text{additional}}$, to the corresponding plate area, A_{plate} , and corresponding variation presented in Figure 9.

$$\gamma = F_{\text{additional}}/A_{\text{plate}} \quad (11)$$

It can be observed that, under different S_u conditions, the hybrid foundation incorporating a monopile with a diameter of 4 m exhibits the highest bearing capacity contribution per unit plate area; however, this value gradually decreases with increasing R . This behaviour can be attributed to the fact that, as the plate radius increases, the additional plate area is primarily located in the outer region farther away from the monopile. Owing to stress diffusion within the surrounding soil, the contact pressure acting on the outer plate region progressively decreases, resulting in a lower mobilisation of the surrounding soil resistance. Consequently, although enlarging the plate size further enhances the overall bearing capacity of the hybrid foundation, the increase in bearing capacity is smaller than the corresponding increase in plate area. As a result, the bearing capacity contribution per unit plate area gradually decreases, indicating a diminishing marginal contribution of the additional plate area.

For hybrid foundations with monopile diameters of 6 m and 8 m, the bearing capacity contribution per unit plate area is consistently lower than that of the 4 m monopile case, while its variation with increasing R remains relatively limited. This is because larger-diameter monopiles possess a higher inherent lateral bearing capacity and therefore carry a greater proportion of the applied lateral load. Consequently, the relative contribution of the plate to the overall bearing capacity is reduced, resulting in a lower bearing capacity contribution per unit plate area than that observed for the smaller-diameter monopile.

Furthermore, compared with the 4 m monopile case, the differences in the bearing capacity contribution per unit plate area for the 6 m and 8 m monopile cases gradually decrease as R increases. This indicates that, as the plate size continues to increase, the marginal bearing capacity contribution of the newly added outer plate area gradually converges, and the influence of monopile diameter on the bearing capacity contribution per unit plate area gradually reduces.

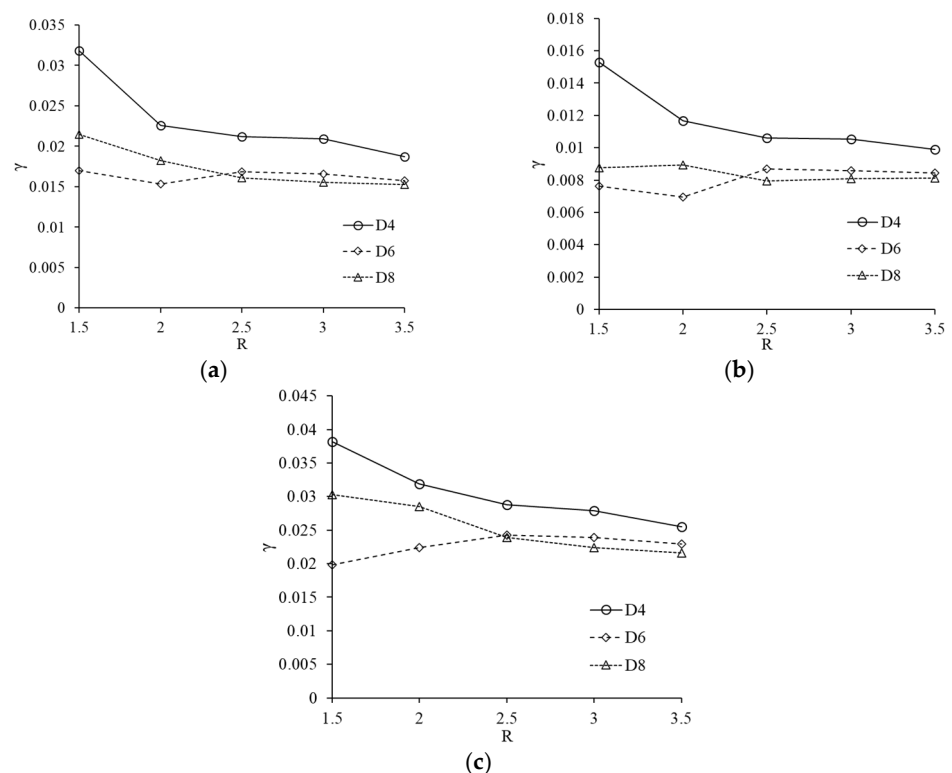


Figure 9. The unit material bearing capacity contribution index (a) $S_u = 30$ kPa (b) $S_u = 60$ kPa (c) $S_u = 90$ kPa.

3.2. Pile Deflection and Pivot Point

Under external loading, the pile exhibits varying degrees of bending deformation. Figure 10 presents the deflection profiles of the pile in hybrid foundations under different parameter combinations at the SLS condition. It can be observed that, under different S_u , a smaller D results in more pronounced pile bending. As R increases, under the same SLS condition (i.e., identical mudline deflection), the reverse displacement at the pile toe location increases. This is primarily because a larger plate mobilises a larger volume of surrounding soil, thereby enhancing the overall system resistance. Consequently, a higher external load is required to reach the same mudline deflection, which intensifies the pile bending and local rotational effects, ultimately leading to a larger reverse displacement at the tie location.

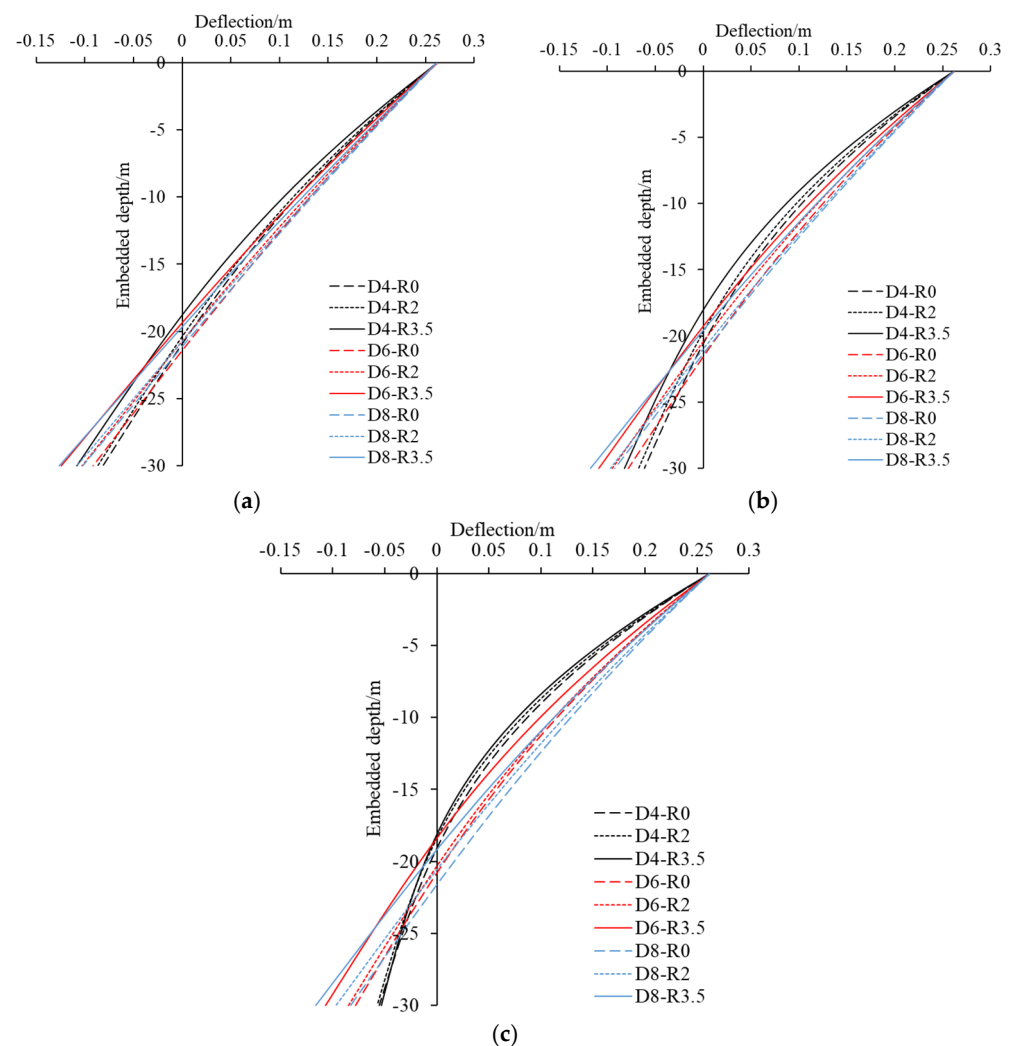


Figure 10. Pile deflection along embedded depth: (a) $S_u = 30$ kPa; (b) $S_u = 60$ kPa; (c) $S_u = 90$ kPa.

The pivot point represents the equivalent centre of rotation of the pile during deflection and divides the pile into two regions with opposite displacement directions. The location of the pivot point varies with changes in system parameters under different conditions. Figure 11 presents a statistical analysis of the pivot point locations for the hybrid foundation.

It can be observed that, under different S_u , the pivot point exhibits a gradual upward shift with increasing R . Specifically, increasing R from 1.5 to 3.5 results in an upward movement of the pivot point by approximately 1.0–1.9 m, depending on the pile diameter. This indicates that the rotation centre of the pile moves closer to the ground surface, and the deformation mechanism of the system transitions from being governed by deeper soil

layers to being dominated by shallower soil. As the plate size increases, a larger portion of the shallow soil is mobilised, allowing it to play a more significant role in load transfer, while the contribution of deeper soil correspondingly diminishes. In addition, when S_u is 30 kPa, the lateral confinement provided by the soil is relatively weak. As the pile diameter D increases, a larger external load is required to reach the SLS condition, leading to a larger reliance on deeper soil layers for load transfer. As a result, the pivot point shifts progressively deeper, with a more pronounced variation. When D increases from 4 m to 8 m, the pivot point moves downward by approximately 0.7–1.5 m, indicating a substantial increase in the participation of deeper soil in load transfer. In contrast, when S_u is 60 kPa and 90 kPa, the pivot point still shows a noticeable downward shift as D increases from 4 m to 6 m; however, when D further increases to 8 m, the change becomes much less significant. The additional downward movement of the pivot point from $D = 6$ m to $D = 8$ m is generally less than approximately 0.5–0.8 m. This suggests that, in stronger soils, the need for deeper soil participation is reduced. Consequently, further increases in pile diameter have a limited influence on the overall load transfer mechanism, and the pivot point tends to stabilise.

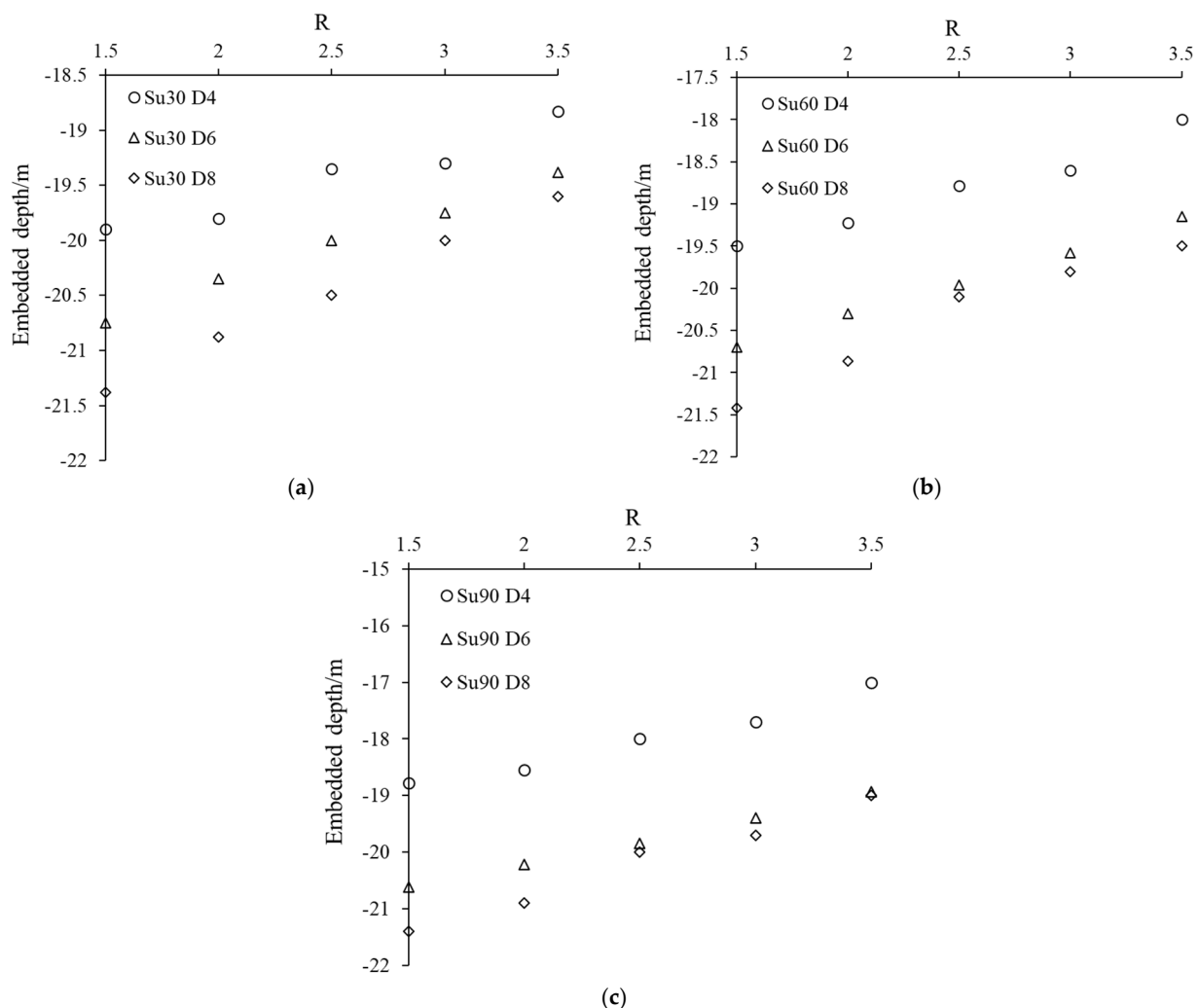


Figure 11. Pivot point position: (a) $S_u = 30$ kPa; (b) $S_u = 60$ kPa; (c) $S_u = 90$ kPa.

3.3. Pile Bending Moment

To further understand the load transfer mechanism of the pile–soil system, it is necessary to analyse the bending moment distribution along the pile. The bending moment, as a key parameter characterising both the load response and deformation behaviour of

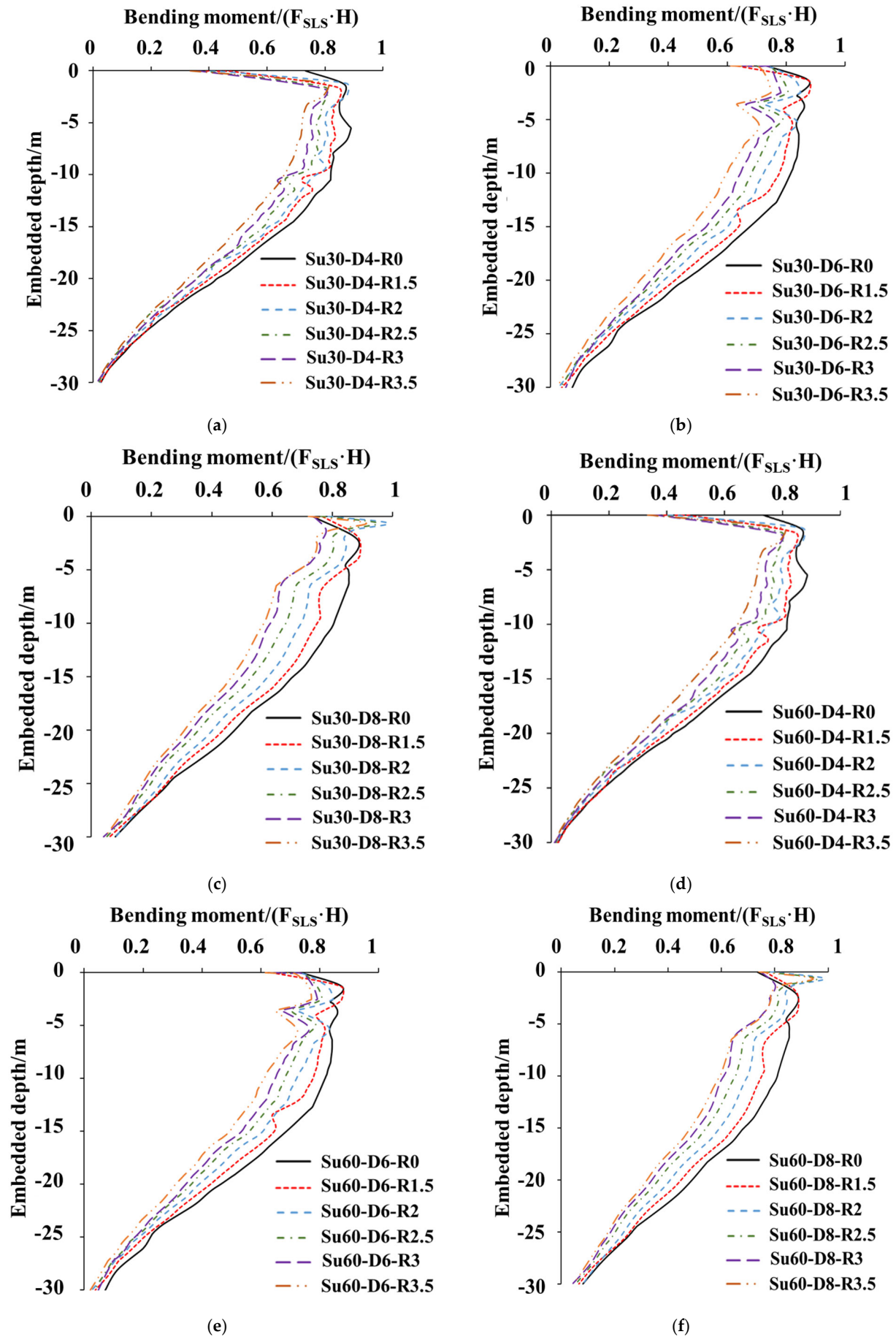
the pile, plays an important role in evaluating structural performance. To facilitate comparison among different cases, a normalised bending moment is adopted, defined as the bending moment divided by the product of the bearing capacity and the distance from the load application point to the mudline. The distribution of normalised bending moment along the embedded depth is shown in Figure 12.

The results indicate that, for $D = 4$ m and $D = 6$ m, the normalised bending moment distributions exhibit similar overall shapes across different S_u for various R , with the maximum values occurring at approximately 2.5 m below the mudline. As R increases, the normalised bending moment along the pile gradually decreases, indicating that the plate takes on an increasing proportion of the load and bending moment within the system. However, when $D = 8$ m, a different behaviour is observed under all S_u . At $R = 2$, the maximum normalised bending moment occurs near the mudline and exhibits a sudden increase, with its peak value significantly higher than those for $R = 1.5$ and $R = 0$. As R increases further, the maximum normalised bending moment gradually decreases while its location remains essentially unchanged; nevertheless, the peak value still remains relatively high. It is worth noting that this phenomenon is confined to the region of the maximum bending moment, whereas the normalised bending moment along the rest of the pile follows the same general trend as observed for $D = 4$ m and $D = 6$ m, namely a gradual decrease with increasing R . This indicates that, for the case of $D = 8$ m and $R = 2$, a local moment concentration effect near the mudline is intensified. For $D = 4$ m and $D = 6$ m, the pile is relatively flexible, allowing local load effects to be redistributed along the pile through continuous bending deformation with depth. Consequently, as R increases, the role of the plate is primarily to progressively share the external load and reduce the bending demand on the pile, leading to an overall decrease in the normalised bending moment without the formation of pronounced local peak amplification. In contrast, when $D = 8$ m, the bending stiffness of the pile is significantly higher, and its deformation capacity is reduced. At $R = 2$, the plate reaches a size sufficient to noticeably influence the confinement of shallow soil and the load transfer interaction at the pile–plate connection. However, its effect is not yet strong enough to redistribute the bending moment smoothly over a wider depth range. As a result, this leads to the occurrence of a local moment concentration near the mudline. As R increases further, the plate contributes more effectively to load and moment sharing, and the overall load distribution within the system becomes more dispersed, causing the maximum normalised bending moment to decrease again.

Therefore, it can be concluded that, for monopiles with smaller diameters, an increase in plate size primarily leads to a continuous reduction in the bending moment along the pile. In contrast, for larger-diameter monopiles, the influence of the plate exhibits a more pronounced stage-dependent behaviour. Specifically, within a certain range of R , the plate not only contributes to sharing the overall load but may also alter the local load transfer mechanism near the mudline, thereby inducing a local amplification of the peak bending moment.

3.4. Pile Shear Force

The shear force provides a direct representation of the pile response under applied loading, and its distribution along the pile is of critical importance for assessing the safety and stability of the foundation. To facilitate comparison among different schemes, the shear force is also normalised by the corresponding bearing capacity in each case. The distribution of normalised shear force along the embedded depth is shown in Figure 13.



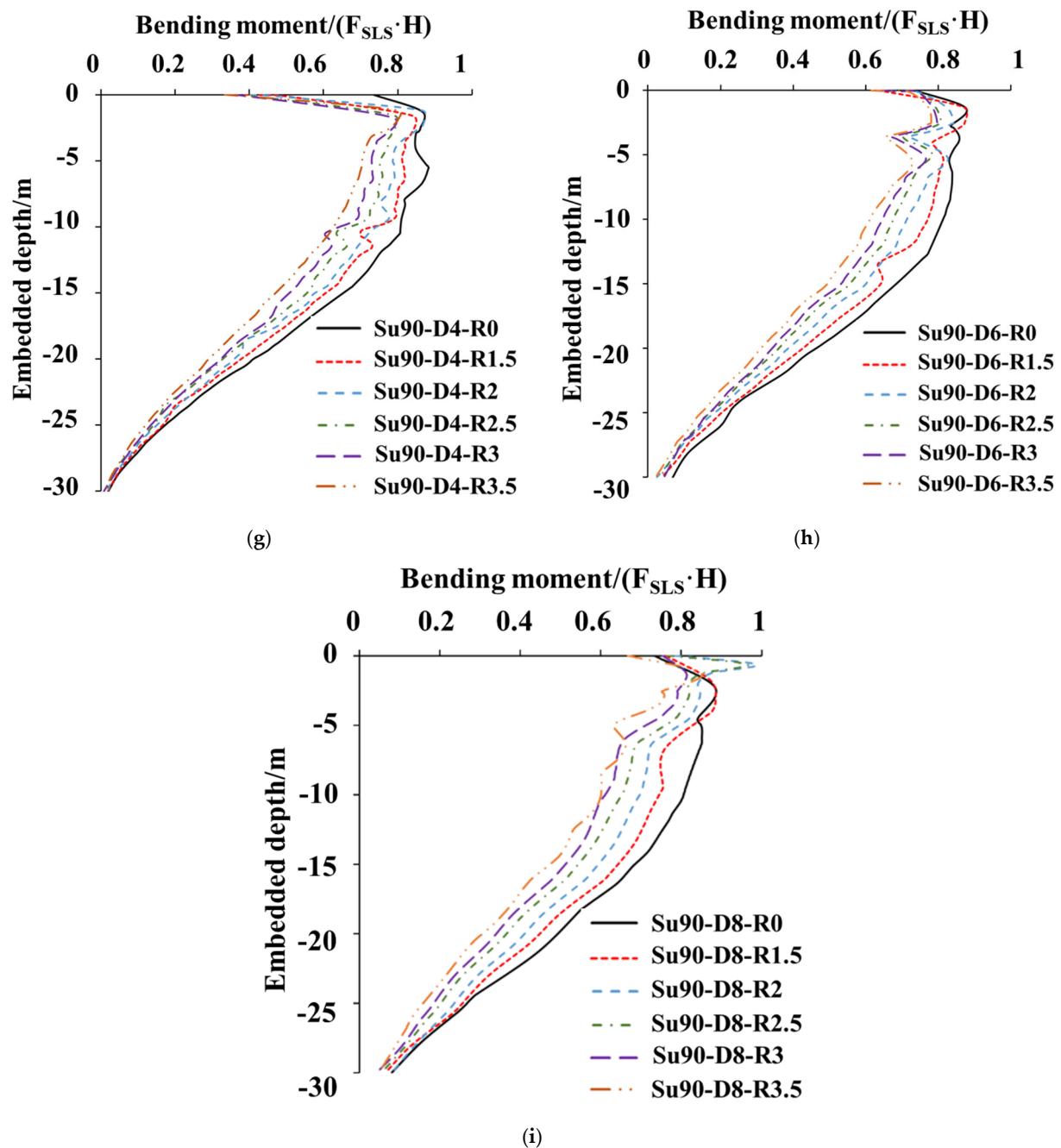


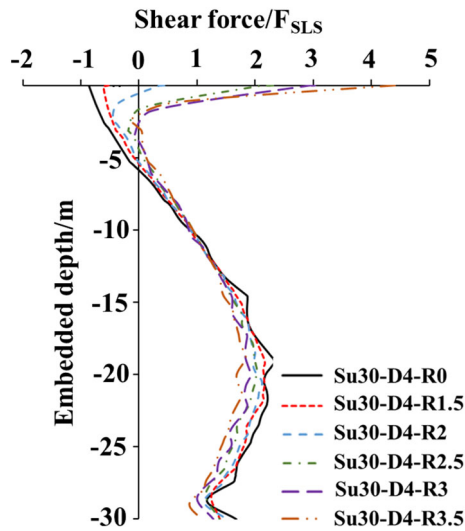
Figure 12. Normalised bending moment curves: (a) $S_u = 30$ kPa, $D = 4$ m; (b) $S_u = 30$ kPa, $D = 6$ m; (c) $S_u = 30$ kPa, $D = 8$ m; (d) $S_u = 60$ kPa, $D = 4$ m; (e) $S_u = 60$ kPa, $D = 6$ m; (f) $S_u = 60$ kPa, $D = 8$ m; (g) $S_u = 90$ kPa, $D = 4$ m; (h) $S_u = 90$ kPa, $D = 6$ m; (i) $S_u = 90$ kPa, $D = 8$ m.

Under different S_u , when the pile diameter D remains constant, the shear force distribution of the hybrid foundation generally exhibits similar variation patterns. When $D = 4$ m and $R = 1.5$, the shear force distribution of the hybrid foundation shows only minor differences compared to that of the monopile. The maximum shear force occurs at approximately 20 m below the mudline, indicating that the influence of the plate on the overall load-bearing mechanism is relatively limited at this stage. When $R = 2$, the direction of the shear force at the mudline reverses. This is primarily attributed to the increased plate size, which begins to bear and redistribute part of the external load near the mudline, thereby altering the local force equilibrium and resulting in a reversal of the shear force direction. However, the primary load-bearing characteristics of the pile are still governed by the deeper soil layers, and thus the maximum shear force remains located at approximately

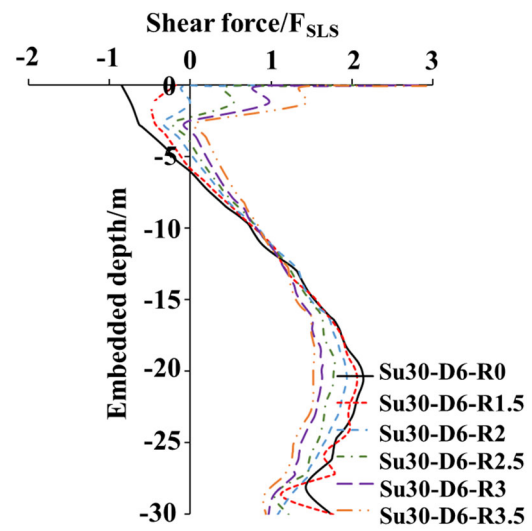
20 m below the mudline. As R further increases, the shear force at the mudline increases significantly and gradually becomes the location of the maximum shear force along the pile. This indicates that the plate participates more actively in load transfer, leading to a more concentrated load distribution near the mudline and more pronounced internal force variations in this region, thereby causing a substantial increase in shear force. Meanwhile, a shear force peak still exists at approximately 20 m below the mudline, but its magnitude gradually decreases with increasing R . This is because part of the load originally carried by the deeper sections of the pile is progressively transferred due to the enlargement of the plate.

When $D = 6$ m, the dispersion of shear force distribution near the mudline becomes significantly more pronounced. This is attributed to the increased pile stiffness, which reduces its ability to redistribute localised loads through bending deformation, thereby causing the load redistribution induced by the plate to become more concentrated in the shallow region. At $R = 1.5$, a trend of shear force direction reversal is already observed at the mudline, indicating that the influence of the plate on the local load-bearing mechanism has begun to emerge. As R increases, the differences in shear force near the mudline become further amplified. At this stage, the shear force peaks are mainly located at two positions: near the mudline and approximately 21 m below the mudline. The former results from the concentrated load distribution in the shallow soil, while the latter reflects the dominant load-bearing response of the deeper pile section. As R exceeds 1.5, the shear force at the mudline gradually increases and becomes dominant, whereas the deep-seated peak progressively decreases. This indicates a transfer of load from the deeper regions of the pile to the shallow layers.

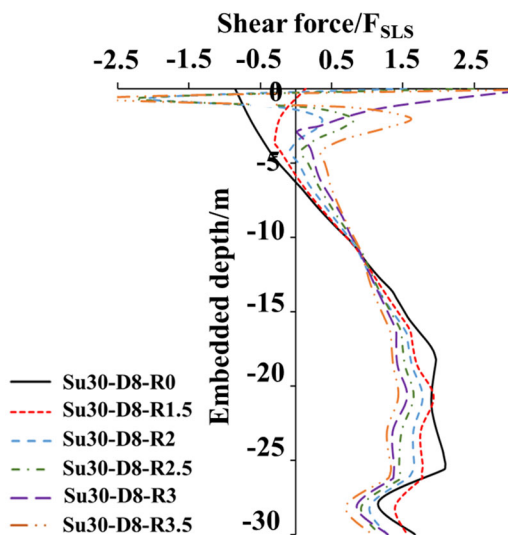
When the pile diameter increases to $D = 8$ m, the shear force distribution exhibits a more pronounced localisation effect, with significantly larger divergence among the curves near the mudline under different R . This also indicates that, as pile stiffness increases, localised loads can no longer be effectively redistributed to deeper regions through bending deformation. Consequently, the load redistribution induced by the plate becomes more concentrated in the vicinity of the mudline. At $R = 1.5$, a clear reversal in the shear force direction is already observed near the mudline. As R increases, the shear force at the mudline rises rapidly and gradually becomes the governing location, indicating that the load is more intensively redistributed in this region, resulting in more pronounced internal force variations. Meanwhile, a distinct shear force peak still exists at approximately 22–26 m below the mudline, reflecting the load-bearing response of the deeper pile section. As R increases, this deep-seated peak follows the same trend observed for $D = 4$ m and $D = 6$ m, gradually decreasing in magnitude. This also suggests that part of the load is progressively transferred from the deeper regions to the shallow layers. Notably, within the range of approximately 0–5 m below the mudline, the shear force exhibits the trend of multiple directional reversals, indicating continuous adjustment of internal forces over a short depth. This behaviour primarily results from the high stiffness of the large-diameter pile, which limits its bending deformation capacity and prevents the transfer of load-induced variations to deeper regions, thereby confining them to the shallow layer. At the same time, the presence of the plate alters the load transfer path near the mudline, causing load dispersion and redistribution within the shallow soil. The combined effects of these mechanisms ultimately lead to the trend of multiple directional changes in shear force within the near-mudline region.



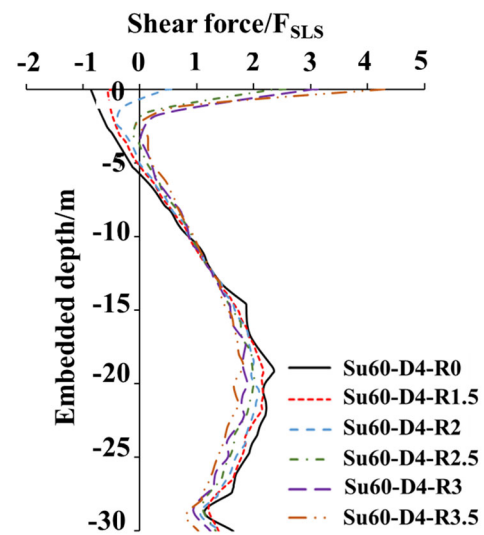
(a)



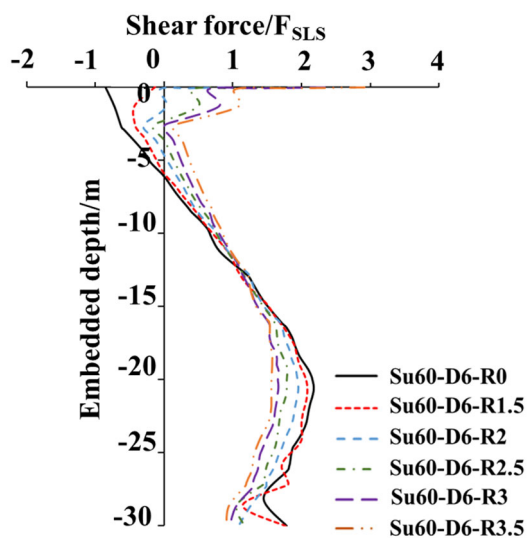
(b)



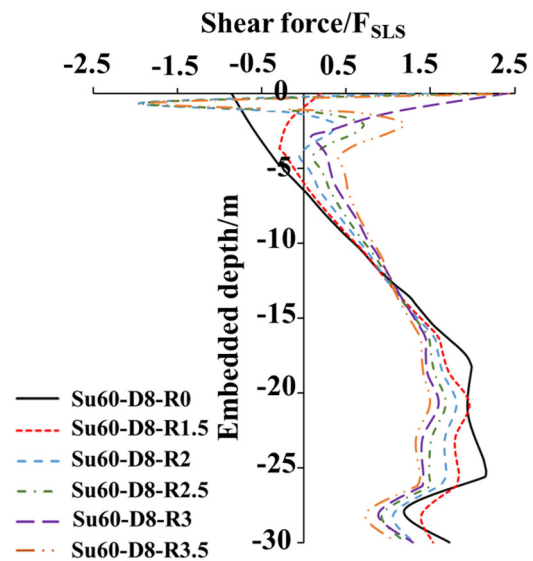
(c)



(d)



(e)



(f)

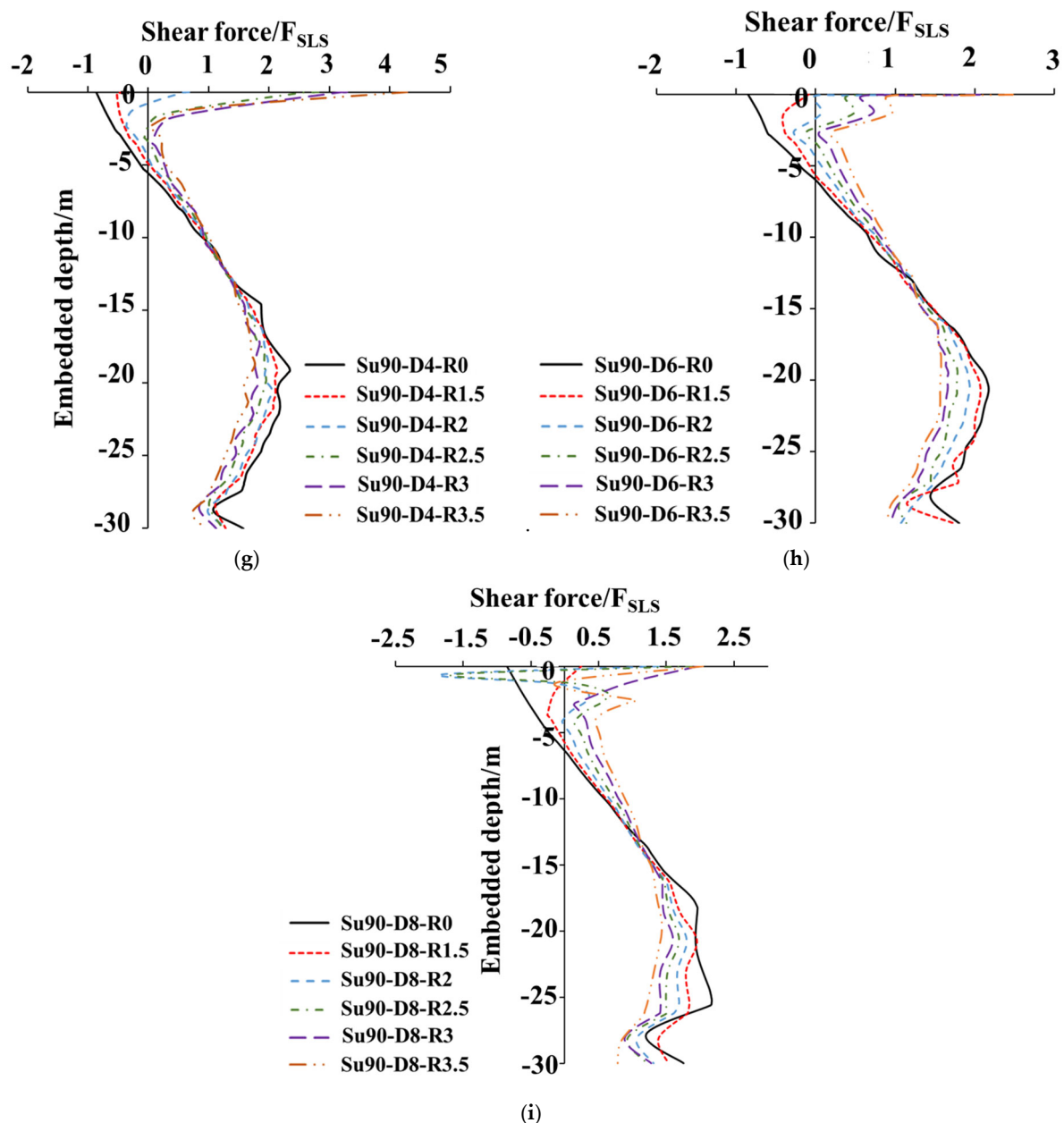


Figure 13. Normalised shear force curves: (a) $S_u = 30$ kPa, $D = 4$ m; (b) $S_u = 30$ kPa, $D = 6$ m; (c) $S_u = 30$ kPa, $D = 8$ m; (d) $S_u = 60$ kPa, $D = 4$ m; (e) $S_u = 60$ kPa, $D = 6$ m; (f) $S_u = 60$ kPa, $D = 8$ m; (g) $S_u = 90$ kPa, $D = 4$ m; (h) $S_u = 90$ kPa, $D = 6$ m; (i) $S_u = 90$ kPa, $D = 8$ m.

3.5. Design Implications for Offshore Wind Foundations

The findings of this study provide useful guidance for the preliminary design of plate–monopile hybrid foundations for offshore wind turbines installed in clay. Based on the numerical results, the selection of the R should not only aim to maximise the lateral bearing capacity but also consider the efficiency of material utilisation. Although increasing R consistently improves the overall bearing capacity of the hybrid foundation, the bearing capacity contribution per unit plate area gradually decreases with increasing R , indicating diminishing marginal returns associated with further enlargement of the plate. Therefore, excessively large plates may increase material consumption and construction costs without providing proportional improvements in structural performance.

The effectiveness of increasing the plate size also depends on the soil strength. Under relatively weak clay conditions, enlarging the plate mobilises a larger volume of shallow soil and produces a more pronounced improvement in lateral bearing capacity and system

stability. In contrast, for stronger clay deposits, the additional benefit provided by further increasing R becomes less significant because the monopile itself is capable of mobilising sufficient soil resistance. Consequently, the plate dimensions should be selected according to site-specific soil conditions rather than simply adopting the largest possible plate.

The present results further indicate that increasing the monopile diameter enhances both the structural stiffness and the contribution of deeper soil layers to load transfer. Therefore, for offshore wind foundations subjected to relatively large lateral loads, improving pile diameter and plate dimensions simultaneously may provide a more balanced enhancement of structural performance than increasing either parameter alone.

Furthermore, the empirical bearing-capacity prediction equation proposed in this study provides a practical tool for preliminary design. Instead of performing time-consuming three-dimensional finite element analyses during the conceptual design stage, engineers may rapidly estimate the lateral bearing capacity of plate–monopile hybrid foundations using the proposed equation under different combinations of D , R , and S_u . Detailed numerical analyses and site-specific investigations are still recommended for final engineering design, particularly for layered marine deposits, cyclic loading, and other complex geological conditions.

4. Conclusions

In this study, numerical models of plate–monopile hybrid foundations are developed in clay using Plaxis 3D, considering different S_u , D , and R . Through systematic comparative analyses, the load-bearing advantages of the hybrid foundation are quantified. Furthermore, the effects of key parameters on foundation performance, together with the pile–soil interaction characteristics and the underlying load transfer mechanisms, are comprehensively investigated. The main conclusions are summarised as follows:

- (1) S_u , D , and R all show positive correlations with the load-bearing capacity of the hybrid foundation. Furthermore, the load-bearing capacity of the hybrid system can be expressed as the superposition of the monopile capacity and a quadratic function of R , reflecting the nonlinear enhancement in foundation performance induced by the addition of the plate.
- (2) The location of the pivot point is jointly influenced by R , D and S_u . As R increases, the pivot point shifts upward, indicating a transition of the system from deep-soil-dominated behaviour to shallow-soil-dominated behaviour. In contrast, increasing D leads to a downward shift in the rotation centre, a trend that is more pronounced in low-strength soils.
- (3) For small-diameter plate–monopile hybrid foundations, the bending moment along the pile decreases progressively with increasing R . For large-diameter hybrid foundations, the bending moment response exhibits a stage-dependent behaviour, where a local concentration of bending moment occurs near the mudline within a certain range of R , followed by a gradual reduction as R further increases.
- (4) The shear force distribution exhibits a pronounced double-peak characteristic, with peaks near the mudline and in the deeper pile section. As R increases, the mudline shear force is enhanced while the deep-seated peak decreases; meanwhile, increasing D leads to a transition from a smooth to a localised distribution, with evident near-mudline intensification and multiple directional reversals for large diameters.

Despite the systematic analysis conducted in this study, several limitations should be acknowledged. First, the soil profile is simplified as a single layer, and different values of S_u are adopted to represent different clay types. However, in reality, soil properties typically vary with depth and multi-layers. Therefore, the present results should be regarded as comparative rather than site-specific predictions. Future work will incorporate depth-

dependent soil profiles to achieve more accurate and realistic assessments. In addition, this study considers three key parameters, namely the S_u , D , and R . Although the obtained results reveal the influence of these key parameters, other potentially important factors, such as the embedment depth, interface reduction factor, and loading height, have not been considered. Furthermore, the present study focuses exclusively on the ultimate lateral bearing capacity under static loading conditions. The mechanical behaviour of plate–monopile hybrid foundations under cyclic loading conditions may differ significantly due to the accumulation of deformation and stiffness degradation. These aspects will be systematically investigated in future work to further improve the understanding of the long-term performance of hybrid foundations in marine clay.

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