

Article

Preliminary Analysis of the Behaviour of Monopiles for Offshore Wind Turbines Founded on Calcareous Sand Profiles of the Ceará Coast Through Numerical Modelling

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Abstract

Offshore wind can diversify the Brazilian electricity matrix, but foundation design on the Ceará continental shelf must account for carbonate sands whose stiffness, crushability and stress-dependent response differ from those of quartz sands. This study contribution is the use of carbonate-sand parameters calibrated from consolidated-drained triaxial tests on Ceará-shelf sediments and the direct comparison of two sands with contrasting carbonate contents. The measured responses calibrated the Hardening Soil model in PLAXIS 2D, and the resulting parameters drove PLAXIS 3D simulations of monopiles with different diameters, embedment lengths and tower heights under monotonic lateral loading. The more calcareous sand showed higher frictional strength but lower stiffness—as a result, it mobilised larger mudline displacements, making the Serviceability Limit State more restrictive than the Ultimate Limit State. Pile diameter controlled lateral capacity, whereas reduced L/D lowered system stiffness and serviceability performance. Within the monotonic, homogeneous-profile scope adopted here, M3 (D = 9 m, L = 36 m, L/D = 4) gave the most favourable response for the medium-rated turbine class. The results provide screening-level comparative evidence, not a design-ready proof of feasibility.

Keywords: offshore wind energy; carbonate sand; monopiles; numerical analysis; Ceará

1. Introduction

The shift from fossil-fuel generation to low-carbon electricity has become a practical design problem, not only an energy-policy target. Offshore wind is central to that shift because marine winds are stronger and steadier than onshore winds, but the gain in energy yield transfers much of the engineering uncertainty to the support structure and to the seabed that resists lateral loading.

Brazil has developed a large onshore wind sector and now ranks wind power as the second-largest source in its electricity matrix [1,2]. The offshore resource is also favourable: the Atlas of Brazilian Wind Potential reports stronger and more consistent winds offshore than onshore [3]. Yet Brazil still operates no offshore wind farm. As of March 2025, more than one hundred offshore projects were under environmental licencing at IBAMA, with a marked concentration along the North-East, South and South-East coasts [4].



Academic Editor: Haixiao Liu

Received: 28 May 2026

Revised: 30 June 2026

Accepted: 15 July 2026

Published: 17 July 2026

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That pipeline cannot be judged on wind resource alone. Along much of the Brazilian littoral, and particularly on the Ceará shelf, the founding soils are carbonate-rich sediments. These materials combine high void ratios, high porosity and grain-crushing susceptibility [5,6]. Their response is non-linear and stress dependent, which makes lateral stiffness, accumulated displacement and shaft resistance harder to predict than in conventional quartz sands.

The issue is acute for driven piles and monopiles. Grain crushing can reduce shaft friction, while cyclic wind and wave actions can degrade stiffness and accumulate rotation [7]. Numerical modelling calibrated with laboratory data offers a practical way to test the soil–structure response before field-scale evidence becomes available. Existing monopile studies [8–15] provide useful benchmarks, but most use quartz or generic granular parameters drawn from the literature. Site-specific calibrated evidence for biogenic carbonate sands, especially from the Brazilian continental shelf, remains limited.

This study addresses that gap by analysing monopile behaviour in Ceará carbonate sands through calibrated numerical modelling. Its novelty lies in the soil data and in the controlled comparison, not in the finite-element method itself. Two carbonate sands are contrasted: CS1, with approximately 49% CaCO_3 , and CS2, with approximately 89% CaCO_3 . The constitutive parameters are fitted to consolidated-drained triaxial tests on Ceará-shelf sediments, and the resulting monopile response is interpreted against SLS and ULS screening criteria. This structure links laboratory behaviour to foundation performance and gives early design guidance for the Brazilian carbonate margin.

1.1. Carbonate Sands

In offshore settings, carbonate sediments derive predominantly from the skeletal fragments of shallow-water marine organisms—chiefly molluscs, algae, foraminifera, sponges, echinoderms and crinoids in coastal regions, and planktonic foraminifera, pteropods and coccoliths in deeper water [16]. Such biogenic deposits are composed of accumulated animal and plant remains, as exemplified by the shell- and coral-derived carbonates [17].

Carbonate sand is a granular material dominated by fragments of high carbonate content, sourced mainly from the skeletal remains of marine organisms [6,18]. These fragments may form in situ, as for benthic organisms, be reworked by currents, waves and wind, or precipitate from the water column [5]; weathering of CaCO_3 -rich sedimentary rock provides a further source, yielding soils of distinctive mineralogy and geotechnical character.

Their biogenic origin gives carbonate sands a geotechnical response markedly different from that of quartz sands: high porosity and void ratio, low particle hardness that promotes fragmentation under load, and depositional or cemented internal structures [19,20]. Unlike the predominantly continental quartz sands routinely tested in the laboratory, biogenic carbonate sands occur chiefly in offshore marine environments, where relevant data remain comparatively scarce.

Carbonate Sands of the Coast of Ceará

The Brazilian continental shelf hosts the largest continuous cover of carbonate sediment on the globe [21], and its composition, distribution and physical properties have been examined in successive studies along the continental margin.

Early surveys identified extensive calcareous-algae banks on the north–north-eastern shelf [22], and later sedimentological work attributed their distribution to the high biogenic productivity of tropical waters [23].

On the Ceará shelf, previous work has focused on facies characterisation and local sedimentary processes. Sediments are classified by provenance as lithoclastic (continental or terrigenous) or bioclastic (marine or organic), a distinction based on CaCO_3 content

and material genesis [24,25]. Dias (1996) [25] classifies shelf sediments as lithoclastic (<30% CaCO_3), transitional (30–70%) or bioclastic (>70% CaCO_3). The two sands studied here therefore represent distinct sedimentological classes: CS1 (~49% CaCO_3) lies in the transitional range, whereas CS2 (~89% CaCO_3) is bioclastic. This mineralogical contrast is expected to control the geomechanical response. Figure 1 maps these facies across the shelf.

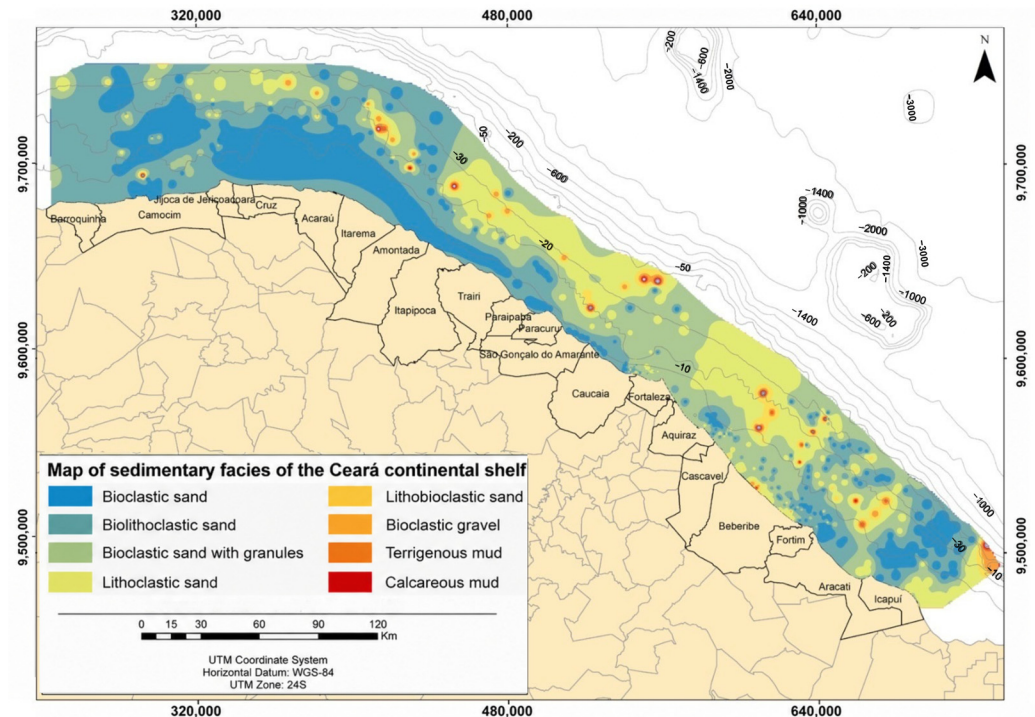


Figure 1. Facies map of the continental shelf of Ceará (adapted from Maia de Almeida [24]).

Geotechnical data for the carbonate sands of north-eastern Brazil remain limited, but the triaxial programme of [26] supplies directly relevant evidence: their biogenic structure raises shear strength relative to comparable quartz soils, yet this gain is offset by greater compressibility (lower stiffness), with particle degradation not dominant at the stress levels tested.

1.2. Wind Energy in Brazil

Onshore wind has become one of the main sources of growth in the Brazilian electricity matrix, supported by an extensive coastal wind corridor [3]. In 2023, it contributed 9.3 GW, or 51.73%, of the annual increase in installed capacity [1]; by the end of that year, Brazil had reached 30.45 GW distributed across more than one thousand onshore wind farms [2].

Offshore exploitation remains at the pre-operational stage, constrained by regulation, installation costs and foundation uncertainty. The licencing pipeline nevertheless indicates a likely expansion [4,27]. Ceará is strategically relevant within this pipeline because it combines high wind potential, shallow continental-shelf areas and carbonate seabed deposits that require soil-specific design checks.

1.3. Offshore Wind Turbines

Most offshore wind farms occupy the continental shelf, often around 10 km or more from the shoreline, where marine winds face less surface obstruction and can yield higher capacity factors [28].

Offshore turbines impose more demanding foundation requirements than onshore turbines. The support system must transfer self-weight and overturning moments while

resisting time-varying actions from wind, rotor vibration, waves and currents (Figure 2). These actions interact; the foundation response must therefore be assessed as a coupled structural and geotechnical problem [12].

Among current fixed-bottom foundation options, the monopile is the most widely used: a single large-diameter steel pile connected to the tower base. Typical diameters range from 3 to 12 m and depend on water depth, embedment, tower height and soil stiffness.

Monopiles dominate because they are simpler to fabricate and install than many alternatives and remain viable in water depths of roughly 30 to 70 m, which can reduce execution costs [29].

As projects move farther offshore and into deeper water, monopile design must satisfy both Serviceability Limit State (SLS) and Ultimate Limit State (ULS) requirements. Diameter and embedded length must resist cyclic environmental actions transmitted mainly as bending moments, horizontal loads and axial forces.

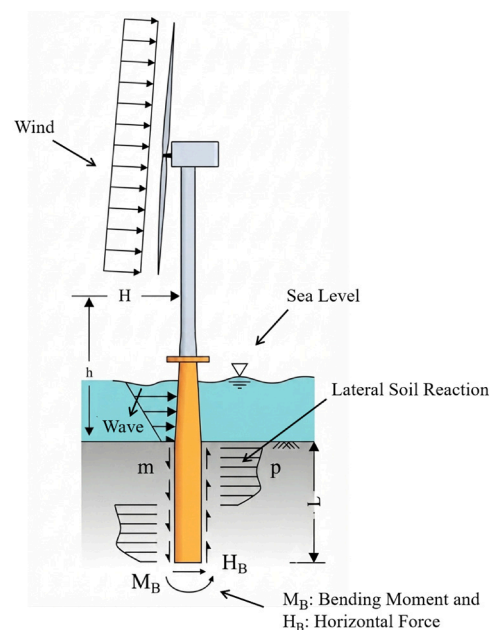


Figure 2. Characteristic loads on an offshore wind turbine [30].

2. Materials and Methods

The workflow links three operations: geotechnical parameter selection, finite-element calibration and limit-state screening. Literature data and triaxial tests on Cear  carbonate sands first defined the input range. The numerical models then reproduced the measured stress–strain response and applied the calibrated parameters to monopiles with controlled changes in diameter, embedment and tower height.

Each configuration was loaded laterally and evaluated against SLS and ULS thresholds from the governing design framework. Only monotonic lateral loading was imposed in the PLAXIS 3D (v2024.2) models: vertical loads were not applied and bending moments were not prescribed as independent load components. The bending demand was generated indirectly by the eccentricity of the lateral action, governed by the tower height h , so that the ratio h/L controls the load eccentricity. The vertical loads and bending moments later reported in this study therefore serve only as literature context, fixing the order of magnitude of offshore-turbine actions and supporting the lateral-load screening threshold; they are not independent load components applied in the numerical model. The procedure does not substitute for site-specific certification; it ranks geometries under comparable

carbonate-sand conditions and identifies which variables most strongly affect serviceability and capacity.

2.1. Numerical Model of the Triaxial Compression Test

The triaxial simulations calibrated the Hardening Soil (HS) parameters against the measured response of the carbonate sands. The HS model was selected because it represents stress-dependent stiffness and separates the E_{50} , E_{oed} and E_{ur} moduli. Unlike the perfectly plastic Mohr–Coulomb (MC) model, HS uses strain hardening: the yield surface evolves during loading [31], and stiffness varies with stress through a power law, allowing different loading paths to develop different stiffness levels [32].

The choice of the Hardening Soil model nonetheless carries known limitations for carbonate sands. As a hardening-only formulation, it reproduces stress-dependent stiffness and pre-peak non-linearity but does not represent post-peak strain softening, particle breakage or the progressive loss of stiffness associated with crushing of the biogenic skeleton. Constitutive laws that incorporate a softening branch or an explicit breakage mechanism (for example, bounding-surface or breakage-mechanics formulations) would capture the dense-state response more faithfully; the present model was retained because it is well established, numerically stable and adequate for the stress range and for the preliminary, comparative scope of this study, in which the relative ranking of geometries—rather than absolute post-peak strength—is of primary interest. Particle morphology, cementation and crushability therefore enter the analysis implicitly, through the back-analysed stiffness and strength parameters, rather than being modelled explicitly; this should be borne in mind when extrapolating the results to higher stress levels or to driven-installation conditions, where grain crushing becomes more pronounced.

Calibration was carried out in PLAXIS 2D (v22.01) following Surarak et al. (2012) [33], using an axisymmetric formulation with 15-node triangular elements. The domain was discretised with a medium-density mesh, locally refined where stress gradients were high and near the boundaries (Figure 3), balancing accuracy against computational cost. This procedure was not intended to reproduce an independent physical case, but rather to verify the numerical consistency of the constitutive model implemented within the finite element framework.

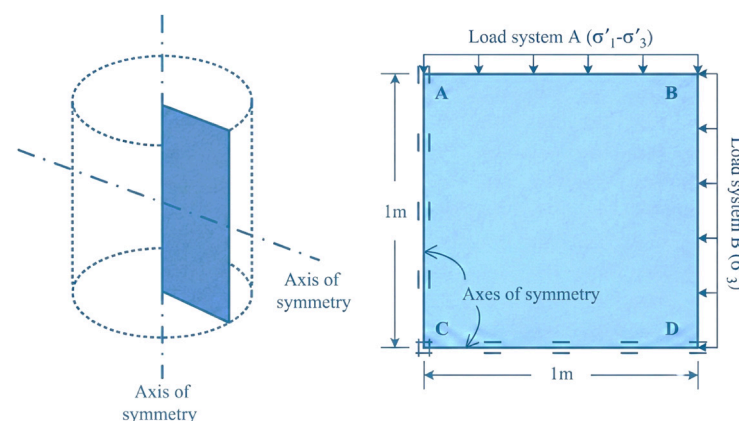


Figure 3. Simplified axisymmetric model for the triaxial compression test (adapted from Surarak [33]).

A unit specimen was used to isolate gravity effects and remove scale dependence. The boundary conditions reproduced the laboratory triaxial test: fixed base support and free top and lateral boundaries, allowing volumetric expansion. The calculation included consolidation under confining pressure followed by shearing under imposed axial strain to failure, following [33] (Figure 4). Because this step reused the same numerical setup

and input parameters reported in the reference, it is a benchmark reproduction, not an independent validation of either the constitutive model or the calibrated Ceará parameters. The stress-displacement curves coincided, giving an RMS error of 0. This confirms correct numerical implementation; it does not add independent predictive evidence.

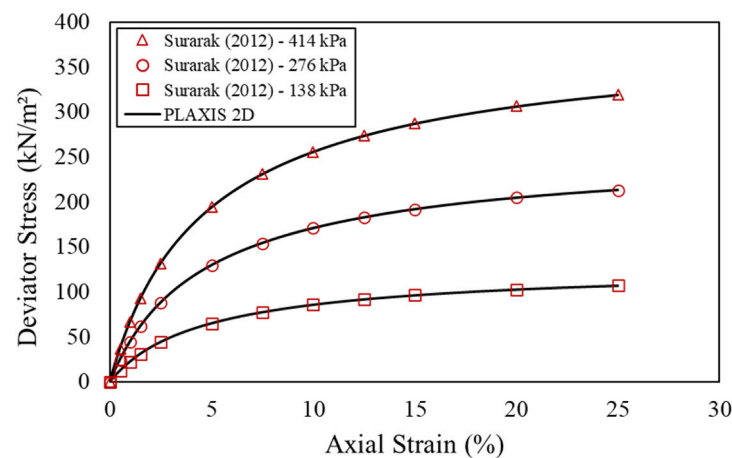


Figure 4. Benchmark of the triaxial compression test model [33].

2.2. Three-Dimensional Model of Offshore Wind Turbines

The soil–structure system was modelled in PLAXIS 3D. The parametric analysis combined structural variables and calibrated carbonate-sand parameters to assess how soil stiffness, carbonate content and monopile geometry affect lateral performance.

Following Brinkgreve et al. (2020) [8], the model used a symmetry plane and represented only half of the domain, reducing computational cost without changing the physical response. The turbine was represented as a monopile-tower assembly with 10-node structural elements.

The benchmark soil domain measured 120 m × 40 m × 60 m (Figure 5). These dimensions were selected to keep the lateral boundaries far enough from the large-diameter monopiles and to minimise boundary effects.

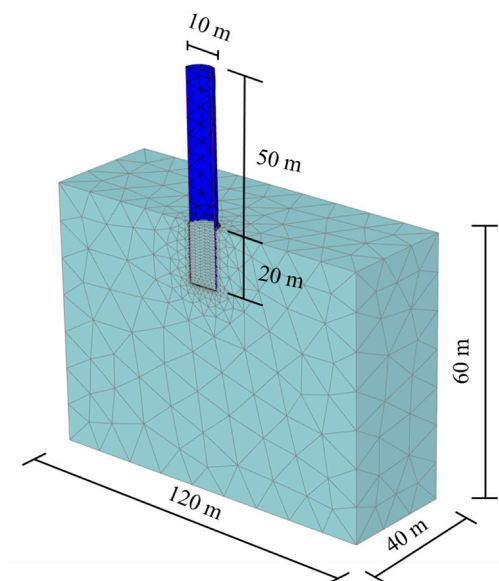


Figure 5. Dimensions of the soil domain and the monopile used in the model. The blue shape represents the monopile–tower assembly above the seabed; the pink color represents the beam element; the green color represents the interface between the soil and the monopile; and the light-blue region represents the soil discretized using finite elements.

Soil, interface and monopile inputs followed Brinkgreve et al. (2020) [8]. The interface reduction factor was inherited from that benchmark rather than fitted to Cear  data; $R_{inter} = 0.90$ was adopted by analogy with offshore monopile practice, in which the pile–sand contact is represented by interface elements weaker than the surrounding soil [8]. Salem et al. (2021) [9] used an interface strength factor of 0.7 for piles in calcareous sand, which supports the need for a reduced contact strength. A sensitivity check of ± 0.1 in R_{inter} produced only minor changes in mudline response, and the load–displacement curves remained nearly coincident at both the SLS and 0.1D thresholds. The pile–soil interface therefore did not control the comparative ranking; pile diameter did. Figure 6 reports the sensitivity analysis, and Figure 7 identifies the mudline node used to extract displacement.

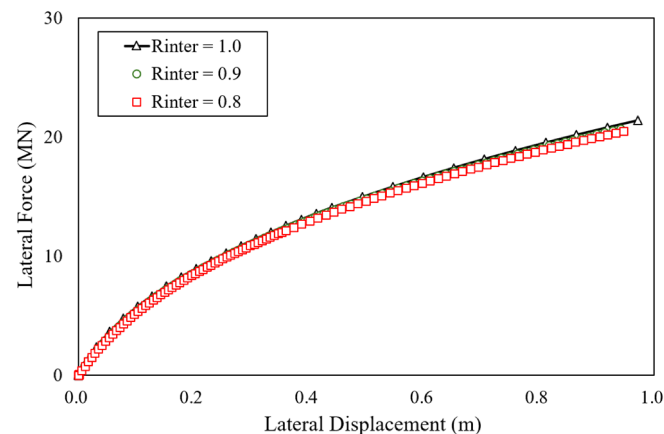


Figure 6. Sensitivity analysis of the interface reduction factor (R_{inter}).

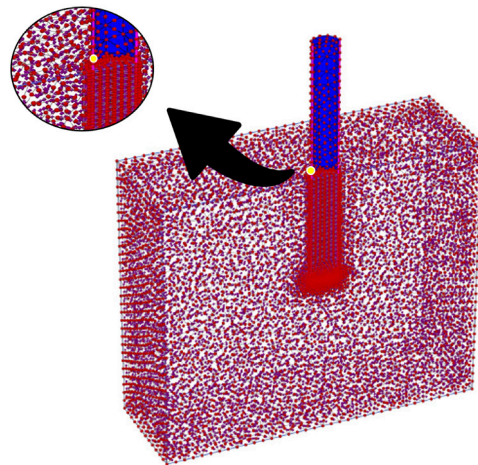


Figure 7. Node corresponding to the mesh sensitivity analysis point. The blue shape represents the monopile–tower assembly above the mudline, while the red shape represents the nodes located between the elements and the stress points. The yellow point indicates the mudline node at which the lateral displacement was extracted.

The software-generated mesh used the Fine refinement level. Because PLAXIS generated the element sizes automatically, the comparison used the lateral force at 0.6 m mudline displacement as the reference response. The Fine mesh differed from the Very Fine mesh by 0.69%, which was the best compromise between numerical accuracy and computational cost.

The lateral boundaries were normally fixed, the base was fully fixed, and the top boundary was free. These conditions prevented artificial rigid-body motion while allowing the soil surface to deform.

The analysis included three stages: geostatic stress generation, monopile installation and prescribed lateral displacement following Brinkgreve et al. (2020) [8]. All stages used the standard PLAXIS 3D Newton–Raphson scheme with a tolerated global error of 0.01. Results were extracted at the mudline node shown in Figure 7, where the SLS and ULS screening criteria were applied.

The 3D model reproduced the benchmark of [8] (Figure 8). Because the soil, interface, material and loading parameters were taken directly from the reference, the comparison checks implementation fidelity rather than validating the model against field data. Three steps are kept distinct: benchmark reproduction verifies that the numerical setup reproduces published results; calibration fits the constitutive parameters to consolidated-drained triaxial data for the Ceará carbonate sands [26]; and parametric application uses those calibrated parameters in the CS1 and CS2 monopile simulations. The 0.865 MN RMS error confirms close agreement with the benchmark response, but the subsequent Ceará-sand results remain calibrated predictions.

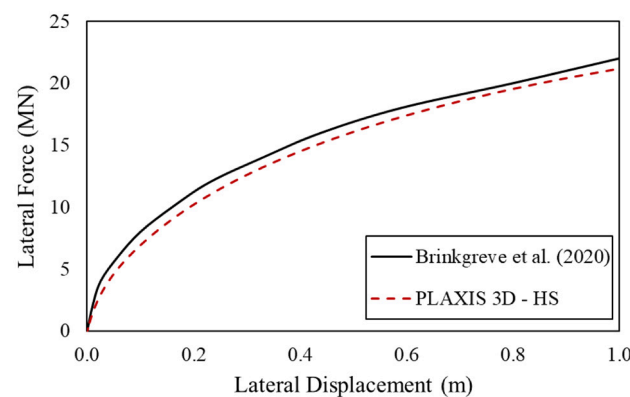


Figure 8. Benchmark reproduction of the numerical model for offshore wind turbine monopile foundations [8].

In the parametric runs, only D , L and h were varied; all other benchmark parameters remained fixed. The load–displacement curves agree across the full displacement range in Figure 8, not only at one point, which is consistent with PISA-based benchmarking practice [8].

The turbine geometries were derived from [8] by varying external diameter (D), embedded length (L) and tower height (h) (Figure 9). The ratios L/D and h/L were used to isolate the effects of embedment and load eccentricity on lateral response.

Table 1 lists the models, organised by the geometric ratios investigated. The pairs (M1, M5), (M2, M6), (M3, M7) and (M4, M8) share tower height and external diameter, differing only in embedded length. A constant wall thickness of $t = 0.08$ m was assigned to all models ($t = 0.091$ m in the validation model), adopted from the linear-elastic steel monopile of the Brinkgreve et al. (2020) benchmark [8] ($E = 200$ GPa, $\nu = 0.3$). The resulting diameter-to-thickness ratios ($D/t \approx 75$ – 112) reflect this benchmark-based simplification: because the present study is a comparative parametric analysis of soil–structure interaction rather than a detailed steel-section design, a uniform wall thickness was retained to isolate the effects of diameter, embedded length and tower height. Detailed wall sizing (typically $D/t < 50$) would be undertaken at a subsequent design stage. The constant wall thickness does not bias this comparative ranking. With $L/D = 2$ and 4 the monopiles fall in the rigid to near-rigid regime, where the lateral response is governed by the mobilised soil reaction and the frontal projected area rather than by the flexural rigidity of the steel section. For a thin-walled tube, the flexural rigidity scales as $EI \propto D^3t$, so that at constant t the $D = 9$ m models are stiffer in bending than the $D = 6$ m models by a factor of about $(9/6)^3 \approx 3.4$; holding t constant therefore reinforces, rather than reverses, the superiority of the larger-

diameter configurations. The stiffness contrast between the two sands is of comparable magnitude—the reference secant stiffness E_{50}^{ref} falls from 15,000 to 11,000 kN/m² in the loose state and from 55,000 to 30,000 kN/m² in the dense state—so the geometry ranking is controlled by the soil–structure interaction and by diameter, not by the simplified wall thickness. A full wall-thickness sensitivity study, with $D/t < 50$ sections, is reserved for the detailed-design stage.

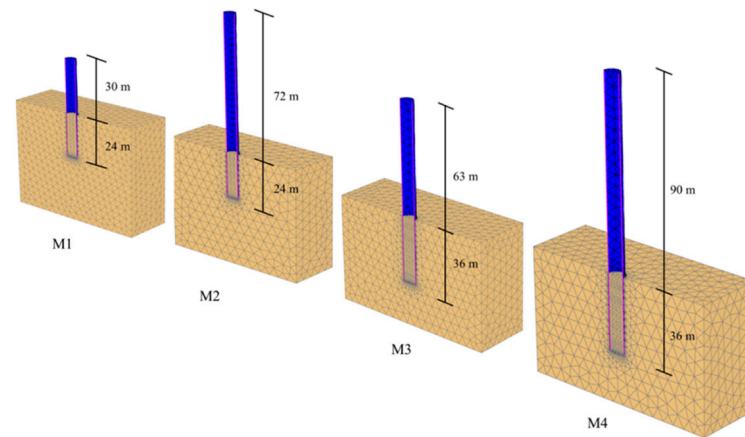


Figure 9. Schematic representation of the offshore wind turbine configurations corresponding to Models 1–4. In each configuration the blue shape is the monopile–tower above the seabed, the grey shape is the embedded length of the pile, and the surrounding block is the finite-element soil domain; the red lines outline the embedded pile within the soil.

Table 1. Geometric parameters of the monopile models considered in the analysis.

Model	h (m)	L (m)	D (m)	t (m)	h/L	L/D
M1	30	24	6	0.08	1.25	4.00
M2	72	24	6	0.08	3.00	4.00
M3	63	36	9	0.08	1.75	4.00
M4	90	36	9	0.08	2.50	4.00
M5	30	12	6	0.08	2.50	2.00
M6	72	12	6	0.08	6.00	2.00
M7	63	18	9	0.08	3.50	2.00
M8	90	18	9	0.08	5.00	2.00

The mesh sensitivity study used M1 because its smaller geometry reduces secondary geometric effects and makes the discretisation comparison clearer. Figure 10 and Table 2 show that refinement increased the number of nodes and stress points, which in turn increased computational time.

Table 2. Mesh sensitivity analysis presenting the number of nodes, stress points, lateral load, and the relative error calculated with respect to the reference mesh.

Mesh	Number of Nodes	Number of Stress Point	Lateral Force (MN)	Relative Error (%)
Very Coarse	8391	17,740	16.98	2.80%
Coarse	9552	20,332	16.97	2.74%
Medium	14,462	32,020	16.75	1.38%
Fine	25,581	60,028	16.63	0.69%
Very Fine	46,021	112,592	16.52	0.00%

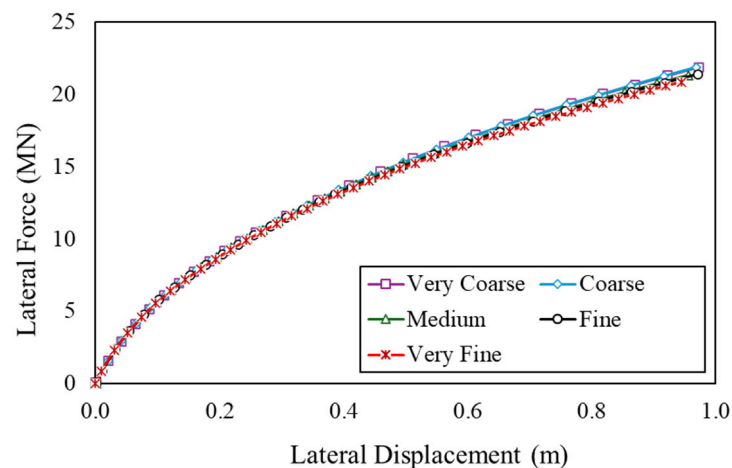


Figure 10. Influence of mesh refinement on the numerical response of the model.

Mesh influence was quantified using the lateral force at 0.6 m displacement. Taking the Very Fine mesh as the reference, all coarser meshes differed by less than 3%, indicating that further refinement changed the response only marginally.

The Fine mesh was therefore retained. It differed from the Very Fine mesh by only 0.69% while requiring substantially fewer nodes and stress points.

SLS and ULS were evaluated from the lateral force-displacement curves following [34]. ULS was defined at a lateral displacement of 0.1D. SLS was defined by a mudline rotation of 0.25° (0.004363 rad). Here θ was estimated as $\theta \approx u/L$, where u is the lateral displacement extracted at the mudline node and L is the embedded length of the monopile; this ratio is adopted as an engineering approximation suitable for preliminary screening, rather than a rotation read directly from the PLAXIS structural pile elements. Using $\theta \approx u/L$ gives $u_{SLS}/D = 0.0175$ for $L/D = 4$ and 0.0087 for $L/D = 2$, values well below the 0.1D ULS displacement. The two checks therefore interrogate different parts of the response: initial stiffness and compressibility at SLS, and progressive mobilisation of lateral resistance at ULS. The $\theta \approx u/L$ estimate is appropriate for the rigid to near-rigid monopiles considered here ($L/D = 2$ and 4), which rotate close to the toe rather than deflecting as slender flexible members. At 0.25° , $\tan \theta \approx \theta$ introduces negligible error. Direct extraction of structural-element rotation would be preferable in detailed design, but it would not alter the comparative ranking sought in this screening study. These thresholds are used only as screening-level allowable-deformation criteria: they provide a consistent basis for comparing the analysed geometries, but they do not replace a certification-level assessment. Final design would still require turbine-specific load cases, complete wind-wave-current combinations, partial safety factors, fatigue verification, dynamic soil-structure interaction, scour assessment and the acceptability criteria of the governing design standard and project certification body [34].

3. Results and Discussion

3.1. Calibration of Geotechnical Parameters of Carbonate Sands

Calibration used the triaxial compression data of [26] for carbonate sediments from the Ceará coast. Parameters were fitted at a reference stress of 100 kPa to reproduce the measured stress and strain paths. Four cases were analysed: CS1 (~49% CaCO_3) and CS2 (~89% CaCO_3), each in loose and dense states, so that compactness and carbonate content could be separated.

The measured and simulated curves were compared quantitatively, not only by visual agreement. For each confining pressure, the error over the common strain range was

assessed through RMSE, MAE and percentage-error indicators where paired numerical and experimental points were available. Errors were lower in the loose states and increased after peak in the dense states, as expected for an HS model without post-peak softening. This confirms that the fitted parameters are most reliable in the pre-peak and serviceability-relevant range used for the monopile comparisons.

For loose CS1 (Figure 11a), the simulation reproduced the strain-hardening trend. The largest mismatch occurred at 50 kPa confining stress, where the experimental curve showed a higher initial stiffness than the other confining pressures, possibly because of seating effects or local inter-granular cementation early in the test.

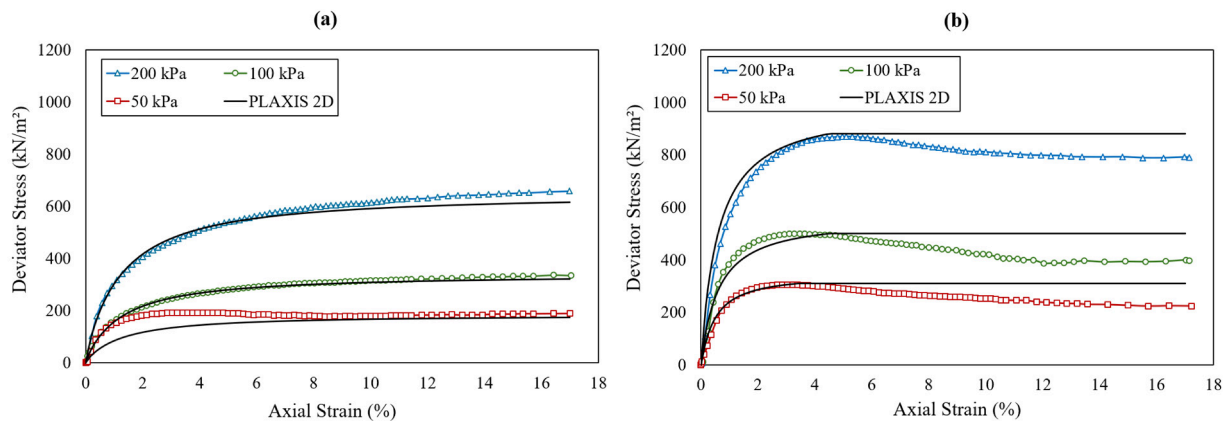


Figure 11. Experimental and numerical stress–strain curves obtained using PLAXIS 2D for carbonate sand CS1 under: (a) loose state and (b) dense state.

For dense CS1 (Figure 11b), agreement remained acceptable up to peak strength. After peak, the HS model could not reproduce strain softening, which is a known limitation of a hardening-only formulation.

For loose CS2 (Figure 12a), the calibrated model reproduced the initial stiffness and progressive strength mobilisation across the confining stresses. The fitting prioritised the early strain range and the secant modulus at 50% of failure stress (E_{50}), which are the quantities most relevant to serviceability response.

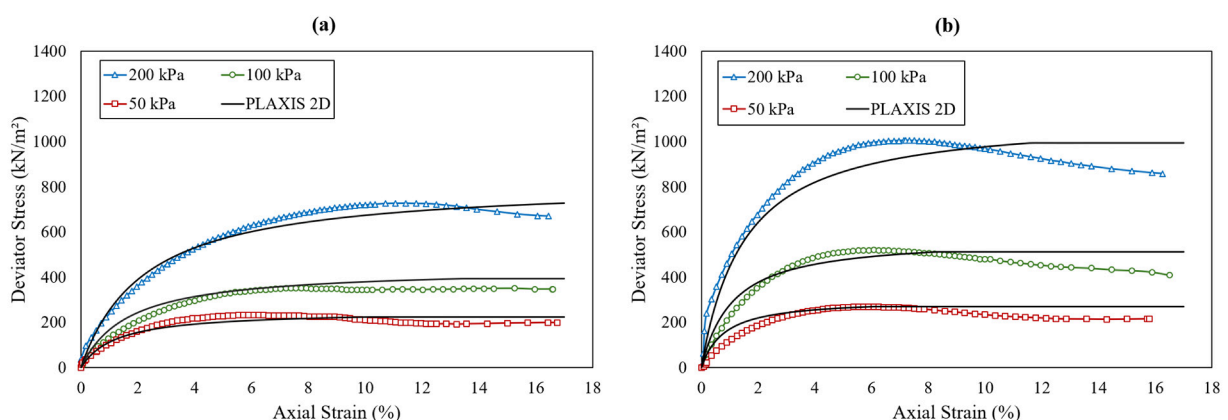


Figure 12. Experimental and numerical stress–strain curves obtained using PLAXIS 2D for carbonate sand CS2 under: (a) loose state and (b) dense state.

For dense CS2 (Figure 12b), the experimental curves developed sharper peaks, indicating stronger mobilisation of the granular skeleton. The same fitting criterion was retained to avoid overfitting the post-peak branch that the HS model cannot represent.

The calibrated parameters in Table 3 translate the laboratory response into the monopile analyses. CS2 is less stiff than CS1 despite its higher friction angle, showing that carbonate content affects stiffness and strength in different directions. The calibrated strength parameters remain consistent with reported ranges for comparable compactness states, while the stiffness contrast between CS1 and CS2 explains the serviceability differences discussed below.

Table 3. Strength and stiffness parameters obtained from the back-analysis of triaxial compression tests.

Samples	CS1—L	CS1—D	CS2—L	CS2—D
γ (kN/m ³)	15.2	17.6	14.3	17.0
φ' (°)	37.5	41.0	39.0	45.0
ψ' (°)	0.0	5.0	4.0	4.0
c' (kN/m ²)	7.5	27.0	13.0	6.0
E_{50}^{ref} (kN/m ²)	15,000	55,000	11,000	30,000
$E_{\text{oed}}^{\text{ref}}$ (kN/m ²)	15,000	55,000	11,000	30,000
$E_{\text{ur}}^{\text{ref}}$ (kN/m ²)	45,000	165,000	33,000	90,000
R_f	1.0	0.9	0.9	0.9
m	1.0	1.0	0.5	0.5
K_0^{nc}	0.42	0.39	0.35	0.35
ν_{ur}	0.2	0.2	0.2	0.2

The main contribution of this study is therefore the calibrated parameter set for Ceará carbonate sands and its use in a controlled monopile comparison. Lateral capacity, mudline displacement and rotation all derive from the soil stress–strain response; generic sand parameters would mask the stiffness penalty associated with higher carbonate content. The fitted parameters provide an initial numerical basis for comparing carbonate deposits along the Ceará coast under consistent geotechnical scenarios.

3.2. Analysis of the Behaviour of Monopiles

The simulations used the calibrated parameters to isolate soil effects. A homogeneous layer was assumed in all cases so that CS1 and CS2, in loose and dense states, could be compared without stratigraphic interference. This is a controlled baseline, not a site-specific seabed model. In natural carbonate deposits, cemented crusts, soft biogenic interlayers or hard lenses would redistribute soil reaction around the pile: a stiff upper crust would raise initial lateral stiffness and reduce mudline rotation, whereas a soft-over-stiff sequence would concentrate deformation near the mudline and reduce the SLS load.

Because site-specific stratigraphy was unavailable for the target areas, the homogeneous assumption is kept as an explicit limitation. The resulting geometry ranking remains informative for early screening, but the absolute SLS and ULS margins remain conditional on the actual vertical soil sequence. Layered finite-element reactions should be introduced once the seabed profile is known [8].

Mudline lateral force-displacement curves were extracted for $L/D = 4$ (Figures 13 and 14) and $L/D = 2$ (Figures 15 and 16), covering both carbonate contents and both compactness states.

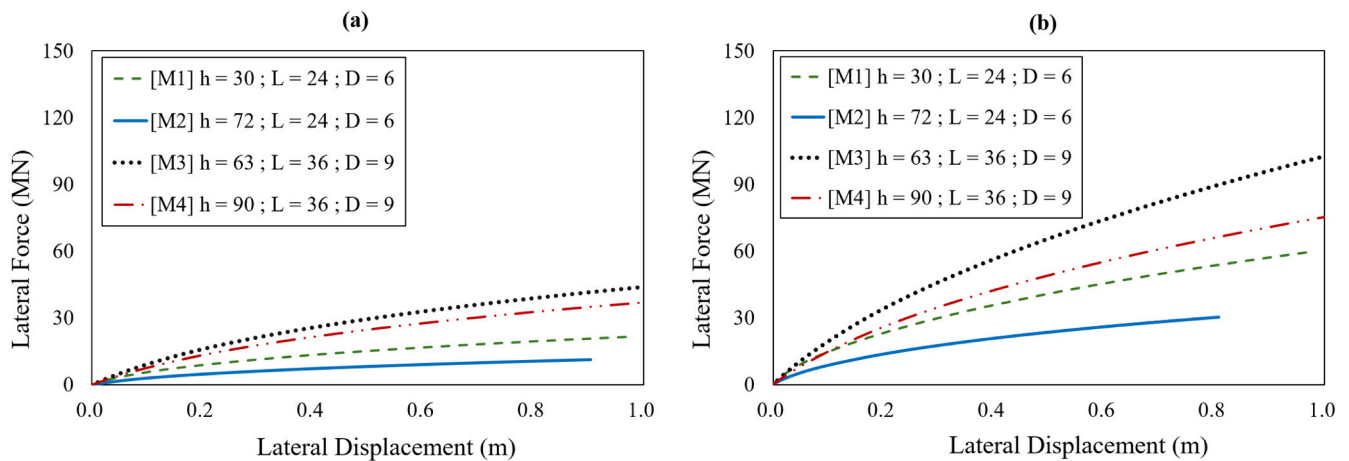


Figure 13. Lateral load–displacement response at the mudline for monopiles with $L/D = 4$ in sample CS1 (49% CaCO_3): (a) loose state and (b) dense state.

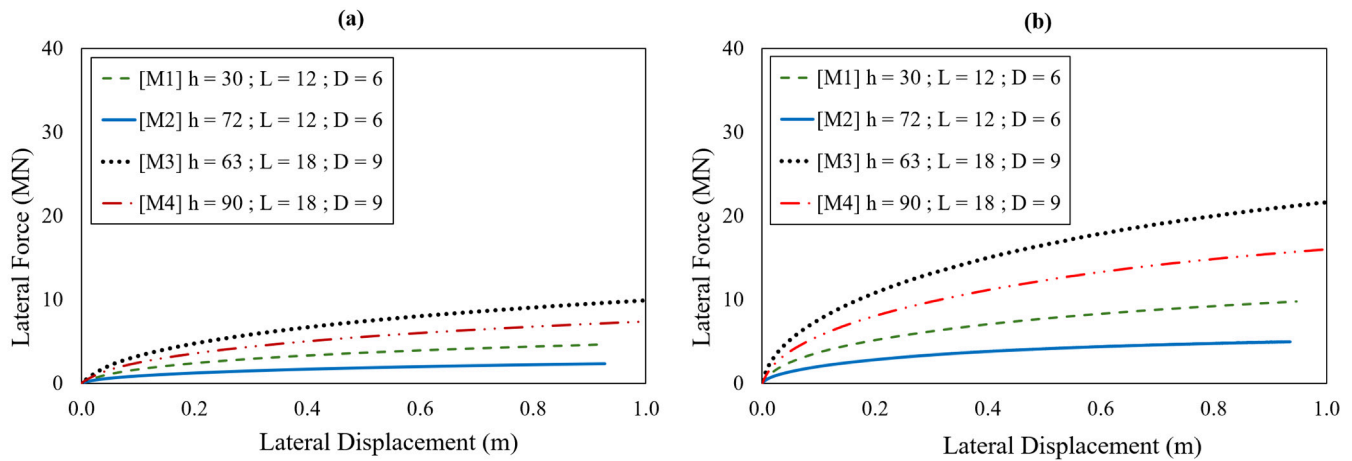


Figure 14. Lateral load–displacement response at the mudline for monopiles with $L/D = 4$ in sample CS2 (~89% CaCO_3): (a) loose state and (b) dense state.

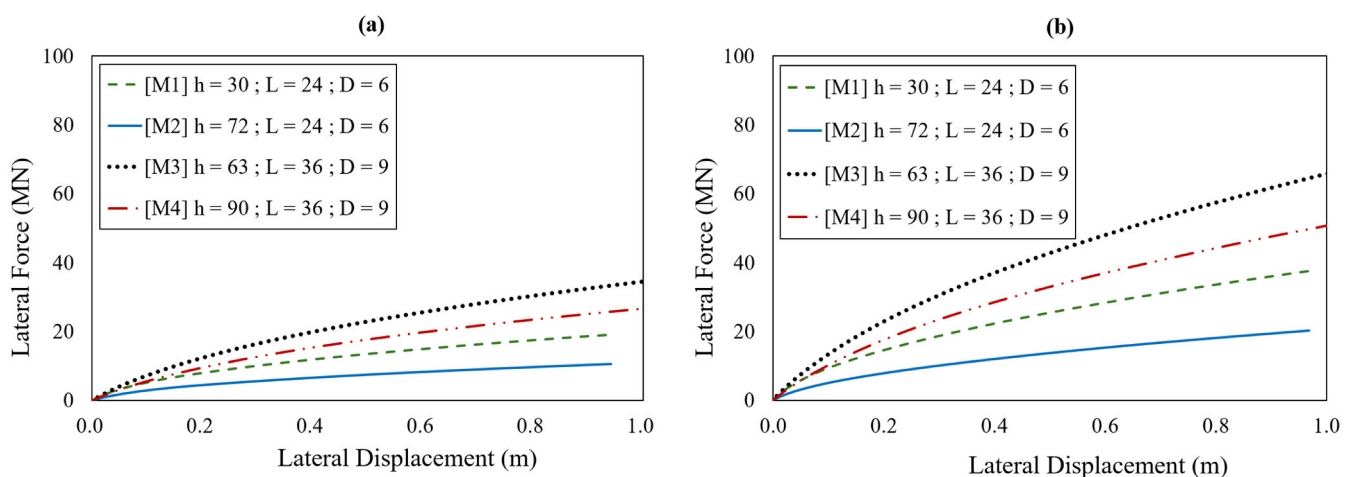


Figure 15. Lateral load–displacement response at the mudline for monopiles with $L/D = 2$ in sample CS1 (49% CaCO_3): (a) loose state and (b) dense state.

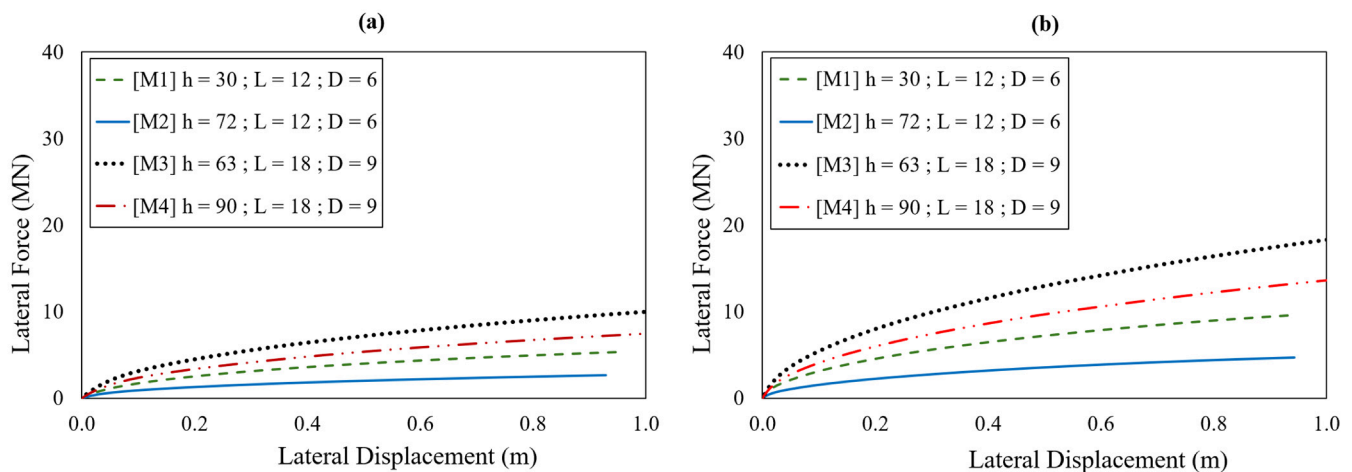


Figure 16. Lateral load–displacement response at the mudline for monopiles with $L/D = 2$ in sample CS2 (~89% CaCO_3): (a) loose state and (b) dense state.

Larger diameters mobilised higher lateral loads at the same displacement because they engage a larger soil volume and increase the frontal projected area resisting horizontal movement.

At low strain, M1 was stiffer than M4 and, in some cases, M3, despite its smaller diameter (Figures 13 and 15). The load application height explains this pattern: the taller M4 produces a larger base moment and a larger initial rotation. Serviceability stiffness is therefore controlled not only by pile diameter but also by superstructure geometry.

At larger displacements, geometry dominated. The order $M3 > M4 > M1 > M2$, and similarly $M7 > M8 > M5 > M6$, shows that larger-diameter piles mobilise more soil resistance once lateral strength is progressively activated.

Changing L/D exposed the role of embedment. Lower L/D reduced initial stiffness and changed the in-service ranking: M5, the $L/D = 2$ counterpart of M1, lost its initial advantage and was overtaken by M8 and matched by M7. With less embedded length, lateral resistance becomes more dependent on the frontal projected area, so diameter overtakes tower-height moment as the governing variable.

The h/L ratio did not provide a transferable predictor across different diameters. Within equal-diameter pairs, however, higher h/L reduced the mobilised lateral load because the longer lever arm increased the base moment and mudline displacement.

Relative density had a clear effect. Dense samples required higher lateral forces to reach the same displacement because they were both stiffer and stronger. The monopile ranking nevertheless remained stable across densities, confirming that larger diameters mobilise higher lateral loads regardless of compactness.

The response separates into two regimes. At large strains, diameter and embedment control ULS capacity. At small strains, tower height and load eccentricity affect initial stiffness and SLS rotation. Foundation optimisation must treat these regimes separately rather than relying on a single geometric ratio.

3.3. Normative Assessment

The SLS assessment targets the low-strain range that controls operational tolerance. Figure 17 and Table 4 report the horizontal loads mobilised at the SLS rotation limit for CS1 and CS2 in loose and dense states.

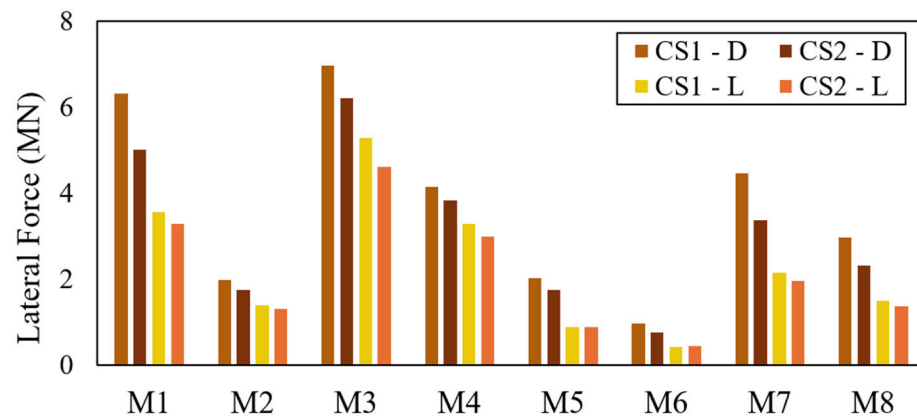


Figure 17. Lateral loads at the Serviceability Limit State (SLS) for CS1 and CS2 sands in loose and dense states across different models.

Table 4. Lateral loads corresponding to the Serviceability Limit State (SLS) for the different models, considering CS1 and CS2 in their loose and dense states.

Model	Lateral Force (MN)			
	CS1—L	CS1—D	CS2—L	CS2—D
M1	3.58	6.33	3.31	5.03
M2	1.42	2.00	1.33	1.78
M3	5.30	6.98	4.64	6.23
M4	3.30	4.16	3.02	3.86
M5	0.92	2.05	0.91	1.77
M6	0.46	0.99	0.47	0.78
M7	2.18	4.50	1.99	3.40
M8	1.53	3.00	1.40	2.33

CS1 generally mobilised higher SLS loads than CS2. Although CS2 has higher frictional strength, its lower stiffness controls the serviceability response: at the same load, CS2 develops larger mudline displacements. Density increased SLS load for both sands, but mineralogy altered the magnitude of that gain. The mechanism is particle-scale: the more calcareous CS2 grains are angular, intra-porous and thin-walled, so they rearrange and locally crush more readily under load than the denser, more rounded particles in CS1. The quantitative contrast is clear. Using $K_p = (1 + \sin\phi') / (1 - \sin\phi')$, the passive coefficient rises from 4.11 to 4.40 in the loose state and from 4.81 to 5.83 in the dense state between CS1 and CS2, whereas E_{50}^{ref} falls from 15,000 to 11,000 kN/m² in the loose state and from 55,000 to 30,000 kN/m² in the dense state. The frictional-strength gain is therefore outweighed by stiffness loss at serviceability strains. This explains why peak friction angle is an unsafe proxy for monopile performance in highly calcareous sands.

The 3.5–5.0 MW turbine class provides the load context for this screening exercise. The compiled cases span shallow-water sites (5–20 m depth) to deeper North Sea environments (25–50 m), with extreme wave heights of up to 22 m and horizontal loads applied roughly 35 m above the seabed; these details are summarised here only to bracket the order of magnitude of offshore-turbine actions rather than to reproduce each design context in full. Literature values indicate characteristic horizontal loads of roughly 4 MN for this class [35–38], and Table 5 reports representative vertical loads, horizontal loads and bending moments. The 4 MN threshold is used only for concept-level comparison; larger turbines or different metocean conditions would require turbine-specific load assessment, as simplified estimates can diverge from OpenFAST results. Under this threshold, the loose states were highly restrictive: only M3 satisfied the requirement for both sands.

Table 5. Characteristic loads acting on offshore wind turbines reported in the literature.

Reference	Turbine Power (MW)	Vertical Load V (MN)	Horizontal Load H (MN)	Bending Moment M (MN·m)	Remarks
Byrne & Houlsby (2003) [37]	3.5	6	4	120	3.5 MW shallow-water reference case.
Lesny & Wiemann (2006) [38]	5	35	16	560	5 MW North Sea case under severe wave conditions.
Doherty & Gavin (2012) [29]	–	6	2	110	Response governed mainly by H and M.
Byrne & Houlsby (2015) [36]	3.5–5	6–10	4–8	–	3.5–5 MW design range for offshore foundations.
Brinkgreve et al. (2020) [8]	11	–	15	525	11 MW PLAXIS monopile benchmark.
Jawalageri et al. (2024)—NREL 5 [35]	–	–	4.33–6.45	165.6–374.0	OpenFAST-based load assessment, NREL 5 MW.
Jawalageri et al. (2024)—DTU 10 [35]	–	–	5.4–11.6	154.9–850.3	OpenFAST-based load assessment, DTU 10 MW.
Jawalageri et al. (2024)—IEA 15 [35]	–	–	11.9–19.1	757.2–1823.8	OpenFAST-based load assessment, IEA 15 MW.

In the dense state, the number of acceptable configurations increased. M1 and M3 satisfied the 4 MN screening threshold for both sands, while M4 and M7 also satisfied it for CS1.

The ULS results (Table 6 and Figure 18) follow the large-strain force-displacement curves. Diameter again controls capacity: larger cross-sections mobilise more surrounding soil, whereas lower L/D reduces lateral stability and failure load. The design trade-off therefore lies between diameter, which raises lateral resistance efficiently, and embedment length, which increases shaft engagement but may introduce installation penalties in dense carbonate profiles.

Table 6. Lateral loads corresponding to the Ultimate Limit State (ULS) for the different models, considering CS1 and CS2 in their loose and dense states.

Model	Lateral Force (MN)			
	CS1—L	CS1—D	CS2—L	CS2—D
M1	16.23	45.27	14.54	28.09
M2	8.88	25.87	7.96	15.12
M3	41.41	95.40	32.02	60.70
M4	34.80	70.43	25.01	46.97
M5	3.92	8.28	4.29	7.81
M6	1.96	4.37	2.17	3.86
M7	9.47	20.84	9.48	17.21
M8	7.00	15.44	7.00	12.82

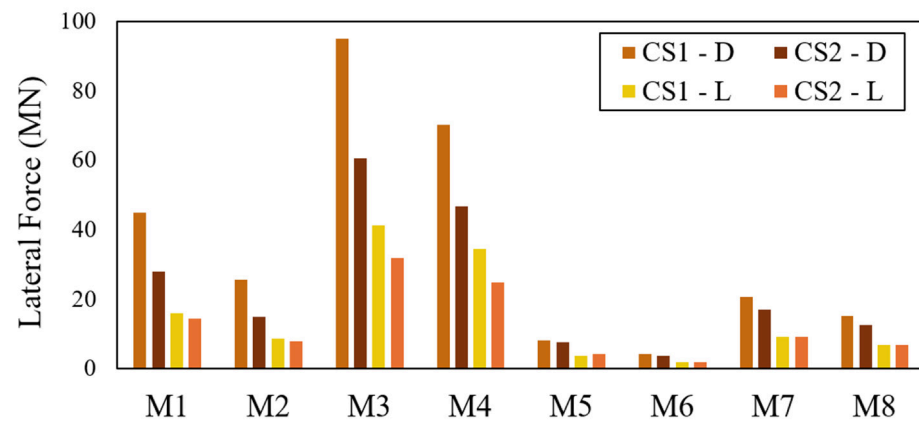


Figure 18. Ultimate lateral load capacity at ULS for CS1 and CS2 sands under loose (L) and dense (D) conditions.

To facilitate comparison, Table 7 consolidates the SLS and ULS lateral loads from Tables 4 and 6 together with the dense-to-loose (D/L) ratio for each sand.

Table 7. Summary of SLS and ULS lateral loads (MN) with the dense-to-loose (D/L) ratio for CS1 and CS2. L = loose, D = dense.

Model	L/D	D (m)	SLS CS1-L	SLS CS1-D	SLS CS2-L	SLS CS2-D	ULS CS1-L	ULS CS1-D	D/L CS1	ULS CS2-L	ULS CS2-D	D/L CS2
M1	4	6	3.58	6.33	3.31	5.03	16.23	45.27	2.79	14.54	28.09	1.93
M2	4	6	1.42	2.00	1.33	1.78	8.88	25.87	2.91	7.96	15.12	1.90
M3	4	9	5.30	6.98	4.64	6.23	41.41	95.40	2.30	32.02	60.70	1.90
M4	4	9	3.30	4.16	3.02	3.86	34.80	70.43	2.02	25.01	46.97	1.88
M5	2	6	0.92	2.05	0.91	1.77	3.92	8.28	2.11	4.29	7.81	1.82
M6	2	6	0.46	0.99	0.47	0.78	1.96	4.37	2.23	2.17	3.86	1.78
M7	2	9	2.18	4.50	1.99	3.40	9.47	20.84	2.20	9.48	17.21	1.82
M8	2	9	1.53	3.00	1.40	2.33	7.00	15.44	2.21	7.00	12.82	1.83

Relative density and mineralogy control the soil–structure interaction, but geometry determines whether the foundation can use the available resistance. Increasing diameter and embedment raises lateral stiffness and capacity, which improves both SLS and ULS response. Within the medium-rated turbine class considered here, the screened configurations carry the reference horizontal loads with varying margins; this is early-planning evidence for ranking geometries, not proof of installation feasibility. Full feasibility would require cyclic loading, time-dependent soil–structure response, layered profiles, scour checks and complete load combinations.

The results are based on monotonic lateral loading. Offshore service involves repeated wind–wave–current actions and occasional extreme combinations, under which stiffness can degrade and rotation can accumulate. Because cyclic effects were not modelled, the serviceability margins reported here may be higher than those expected in operation. Scour, liquefaction susceptibility and long-term stiffness degradation also remain outside the present scope. Current monopile workflows treat monotonic finite-element results as screening information and address repeated loading through dedicated cyclic procedures or equivalent degradation frameworks when serviceability rotation governs [8,34].

For prospective Ceará developments, the practical message is clear. Diameter governs ULS capacity and, through system stiffness, SLS performance; increasing diameter is therefore usually more effective than lengthening the embedment for satisfying the rotation limit. The cost and installation consequences differ. Larger diameters increase steel tonnage, fabrication effort and driving demand, whereas longer embedment increases material

length and may raise the risk of refusal in dense carbonate layers. Subject to vessel and fabrication constraints, diameter increase is geotechnically preferable in this screening set because it raises capacity at both small and large displacements, while embedment gains tend to saturate beyond moderate slenderness [8,30].

4. Conclusions

This study evaluated monopile foundations in two Brazilian carbonate sands with contrasting calcium carbonate contents: CS1 (~49% CaCO_3) and CS2 (~89% CaCO_3). The analysis linked laboratory-calibrated Hardening Soil parameters to PLAXIS 3D monopile simulations and interpreted the results against SLS and ULS screening thresholds.

The calibration reproduced the pre-peak stress–strain response with the closest agreement in the loose states. Dense-state post-peak softening was not captured, as expected for the HS model, but the calibrated parameters were adequate for the serviceability-relevant range used in the comparative analyses. The eight monopile configurations led to the following findings:

- For a given L/D ratio, the largest diameters gave the highest lateral capacities: M3 and M4 for $L/D = 4$, and M7 and M8 for $L/D = 2$;
- Reducing L/D lowered system stiffness because shorter embedment mobilised less lateral resistance along the shaft;
- h/L alone was a weak predictor of lateral resistance. Its effect became meaningful only within equal-diameter pairs, where greater load eccentricity reduced the mobilised load;
- At ULS, the dense-to-loose load ratio ranged from 2.02 to 2.91 for CS1 and from 1.78 to 1.93 for CS2, indicating a narrower compactness effect in the more calcareous sand;
- M3 ($D = 9$ m, $L = 36$ m, $L/D = 4$) gave the highest SLS and ULS loads for both sands and both compactness states;
- Reducing L/D caused larger SLS-load losses in the loose states, while dense states were less sensitive to the same geometric change.

Carbonate content controlled the balance between strength and stiffness. CS2 reached higher frictional strength but was more compressible than CS1; that stiffness loss reduced the load available at the permissible serviceability displacement. In these sands, compressibility can control design before peak strength is mobilised.

Overall, monopile performance was governed by pile diameter and by the carbonate-content-dependent trade-off between frictional strength and deformability. M3 was the most favourable configuration in the screening set, and M1 ($D = 6$ m, $L = 24$ m, $L/D = 4$) was the next most favourable, satisfying the serviceability threshold for both sands only in the dense state. These results identify numerical trends under monotonic loading in a homogeneous profile. They do not demonstrate full feasibility, which would require cyclic and time-dependent analyses, site-specific stratigraphy, full environmental load combinations, scour assessment and installation checks.

Limitations and Future Work

The findings are bounded by four main limitations. First, the Hardening Soil model does not reproduce post-peak softening, particle crushing or cementation, all of which are relevant to carbonate sands. Future analyses should test constitutive laws with explicit breakage or softening mechanisms. Second, the monotonic and homogeneous-profile assumptions should be replaced by cyclic loading, accumulated-rotation checks, stiffness-degradation assessment and layered site-specific stratigraphies. Third, interface-strength sensitivity, installation effects, scour and field validation must be added before any configuration is advanced to detailed design. Fourth, the experimental database should be

expanded to include carbonate sands from different sectors of the Ceará shelf and a wider range of CaCO_3 contents, allowing spatial variability to be quantified rather than assumed.

Author Contributions: Conceptualization—original draft preparation, J.C.d.N.S., G.F.A. and C.G.d.S.P.; formal analysis, J.C.d.N.S.; experimental procedures, J.C.d.N.S.; methodology and supervision, J.C.d.N.S., G.F.A., C.G.d.S.P. and A.S.M.; writing—review and editing, project administration, and funding acquisition, A.S.M. All authors have read and agreed to the published version of the manuscript.

Funding: This research was funded by PETROBRAS, made effective through the P&D cooperation agreement under the scope of PD-00553-0069/2021 and CNPq for funding the project under process number 446341/2024-0.

Data Availability Statement: The data supporting the results of this study are openly available in the Institutional Repository of the Federal University of Ceará at the following address: <https://repositorio.ufc.br/handle/riufc/84961> (accessed on 15 May 2026). The dataset is associated with the undergraduate thesis entitled “Analysis of the Performance of Offshore Monopile Foundations in Carbonate Sandy Soils of the Ceará Coast Through Numerical Models”, by Sales (2026) advised by Prof. Alfran Sampaio Moura.

Acknowledgments: The authors are grateful for the financial support of PETROBRAS, made possible through the R&D cooperation agreement (within the scope of PD-00553-0069/2021), which meets ANEEL’s requirements. J.C.d.N.S. thanks the Coordenação de Aperfeiçoamento de Pessoal de Nível Superior (CAPES) for grant number 88887.271412/2026-00. G.F.A. and C.G.d.S.P. thank the National Council for Scientific and Technological Development (CNPq) for grants 380588/2026-9 and 314814/2025-6, respectively. C.G.d.S.P. also acknowledges CNPq funding under process number 446341/2024-0.

Conflicts of Interest: This study used data generated within a project funded by PETROBRAS through the R&D cooperation agreement PD-00553-0069/2021. The conception of the study, data analysis and interpretation, manuscript preparation, and submission were carried out exclusively by the authors of this manuscript.

Abbreviations

The following abbreviations are used in this manuscript:

ABEEólica	Brazilian Wind Energy Association
CS1	Carbonate Sand 1 ($\approx 49\%$ carbonate content)
CS2	Carbonate Sand 2 ($\approx 89\%$ carbonate content)
HS	Hardening Soil (constitutive model)
MC	Mohr–Coulomb (constitutive model)
ULS	Ultimate Limit State
SLS	Serviceability Limit State
UFC	Federal University of Ceará

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