

Comprehensive site response analysis for offshore foundations: evaluating soil-structure interaction and constitutive model effects under seismic loading

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ABSTRACT: This paper describes a site response analysis (SRA) conducted by WSP to support offshore foundation design under seismic (i.e. earthquake) loading conditions. It includes a useful comparison of simple one-dimensional free-field simulations using the popular NERA program with more sophisticated two-dimensional and three-dimensional numerical model simulations using PLAXIS. The soil-structure interaction (SSI) effects of two ISO design level earthquakes on a subsea foundation were evaluated. Two foundation types were compared, comprising a conventional (mudmat) shallow foundation, and a more innovative hybrid foundation with piles. Three constitutive models – Isotropic Hardening Soil (HS), Hardening Soil with small-strain stiffness (HSsmall), and UBC3D-PLM – were employed to represent a range of geotechnical behaviours, including liquefaction potential, which was considered important given the calcareous nature of the seabed sediments. This paper highlights the critical role of modelling dimensionality and the importance of constitutive model selection. Convenient, but simplified 1D analyses like NERA, lacking strength degradation and accurate pore-pressure considerations, can be shown to overestimate spectral accelerations compared to FE methods employing advanced constitutive models. The 3D simulations demonstrate advantages in capturing multi-directional wave scattering and energy redistribution, affecting the predicted seismic demands. Modelling SSI effects enables the seismic response to be better captured, to improve the reliability and enable optimisation of foundation systems for seabed infrastructure.

KEYWORDS: offshore seismic design, SRA, finite element, dynamic analysis, SSI, constitutive modelling, liquefaction.

1 INTRODUCTION

Seismic site response analysis (SRA) is critical for offshore foundation design, especially in seismically active regions where ground shaking poses a significant threat to the stability and integrity of marine structures (Kontoe et al., 2024; Ngo and Kim, 2024). This is particularly the case for subsea structures that are relied upon to ensure hydrocarbon containment of the attached pipelines. Offshore installations are often exposed to complex loading conditions that necessitate detailed modelling approaches capable of capturing key geotechnical and structural phenomena. Key factors include spatial soil heterogeneity (Laib et al., 2015), non-linear constitutive soil behaviour under cyclic loading (Soleimanian et al., 2023), and dynamic soil-structure interactions (SSI) effects (Ju and Huang, 2019). Among these, SSI is intrinsically linked to structural dynamic response – either by reducing or exacerbating seismic demands, depending on the interplay between soil properties and structural stiffness (Romero-Sánchez and Padrón, 2024).

Traditional one-dimensional (1D) analyses, while computationally efficient, typically assume vertically propagating shear waves in horizontally layered soils and neglect key elements such as lateral heterogeneity and kinematic interaction effects. These simplifications may either result in overly conservative or potentially unsafe designs, particularly for offshore structures with complex foundation systems. Despite advancements in 1D simulation methods – such as enhanced nonlinear representation in the software NERA (Nonlinear Earthquake Site Response Analysis, Bardet and Tobita, 2001) – their capability remains limited when representing near-field interactions, spatial variability, and multi-directional seismic effects. In contrast, two-dimensional (2D) and three-dimensional (3D) numerical analyses can better capture these complexities, enabling a more accurate assessment of both ground motions and foundation response (Kramer and Stewart, 2024). This represents a value-adding complementary additional approach.

This paper seeks to explain, by example, the advantages of this approach, by describing an actual study that employs a comparative multi-dimensional modelling approach: the nonlinear 1D software NERA for baseline site response evaluations, and advanced 1D, 2D and 3D numerical analyses using PLAXIS. The primary objectives of the study are to

systematically investigate the impact of model dimensionality, soil constitutive modelling, and explicit SSI representation on predicted seismic responses. By evaluating these aspects, the study aims to enhance seismic assessment methodologies, providing practical insights and guidance for safer, more resilient offshore structural designs.

2 GEOTECHNICAL SITE CHARACTERIZATION

The study site is in an offshore region, within a seismically active tectonic setting characterized by subduction interface and regional crustal fault systems. Regional seismicity is characterized by frequent moderate-to-large magnitude events ($M_w > 6.5$), with plate convergence rates across the trench exceeding 85 mm/year.

A Probabilistic Seismic Hazard Assessment (PSHA) was performed to define appropriate input motions for dynamic analyses. The hazard modelling incorporated Next Generation Attenuation (NGA) ground motion prediction models applicable to both active shallow crustal sources and subduction sources, selected for their regional relevance and wide acceptance in offshore seismic design. Vertical-to-horizontal spectral ratios (V/H) of 0.5 were adopted across all structural periods, with values up to 0.7 observed for stiff structures with periods < 0.2 s.

Based on a time-averaged shear wave velocity (V_{s30}) of 265 m/s, the site is classified as Site Class D in accordance with ISO 19901-2 (2022), reflecting a profile that is moderately stiff in a seismic classification context, yet dominated by soft, fine marine sediments that are prone to dynamic amplification.

Despite the elevated carbonate content (ranging from 20% to 60%), no evidence of cementation, structured fabric, or hard crusts was observed. In terms of consistency, the soil remains soft to firm, with low sensitivity (generally < 5), and the mechanical behaviour is broadly comparable to that of natural marine clays and silty sands under low-to-moderate stress conditions. These geotechnical characteristics necessitate a robust site response analysis (SRA) to ensure reliable performance of the various offshore foundation systems built on the seabed.

Seismic wave propagation at the site is governed by depth-dependent variations in undrained shear strength (s_u), dynamic shear stiffness (G_{max}), and material damping (ξ), all of which

are sensitive to strain levels. These properties were characterized through a geotechnical investigation that combined in-situ testing methods (e.g., SCPT) and laboratory-based dynamic testing (e.g., resonant column, cyclic SS). At each depth, three different soil parameter profile scenarios – Low Estimate (LE), Best Estimate (BE), and High Estimate (HE) – were defined to account for variability and uncertainty in soil properties. The geotechnical profiles, as illustrated in Figure 1, form the basis for dynamic response simulations conducted across 1D, 2D, and 3D modelling frameworks.

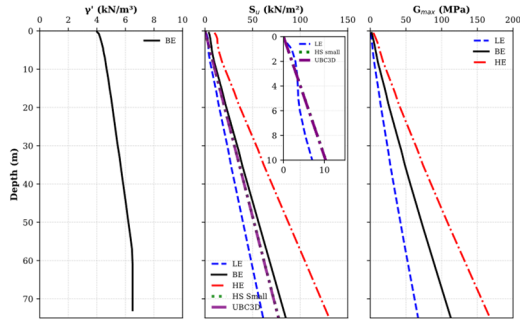


Figure 1. Soil profiles for three different parameter scenarios: (LE) low estimate, (BE) best estimate, and (HE) high estimate.

3 SEISMIC LOADING CONDITIONS

The seismic loading conditions used in this study are derived from the site-specific PHSA and are associated with the following probabilistic scenarios, defined in alignment with ISO 19901-2 (2022) offshore seismic design guidelines:

- Extreme Level Earthquake (ELE): 170-year return period.
- Accidental Level Earthquake (ALE): 1,400-year return period.

Ground motion input was based on spectrally matched records consistent with the target response spectra developed for a V_{s30} of 265 m/s. The selected reference motion produced an earthquake with $M_w = 7.1$, scaled to match the response spectrum at 5% structural damping for each scenario. The resulting input peak ground accelerations (PGA) are 0.05 g for ELE and 0.14 g for ALE (Figure 2).

In accordance with ISO design guidance, the following combinations were evaluated:

- ELE LE and ELE HE: representing the Extreme Level Earthquake under Low and High Estimate soil conditions.
- ALE BE: representing the Abnormal Level Earthquake under Best Estimate conditions.

For the sake of brevity in this paper, only the detailed analysis for the ELE LE case is presented. This scenario was selected because low-stiffness soils (LE profile) are expected to exhibit greater seismic wave amplification and stronger nonlinearity, making it a critical case for evaluating modelling approach differences.

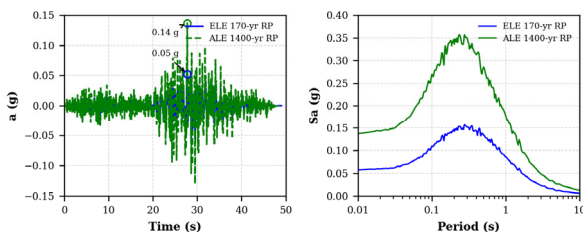


Figure 2. Input time histories and response spectra.

4 MODELLING TOOLS

As described previously, the study compares 1D, 2D, and 3D models to assess offshore site response under complex soil-structure interaction.

4.1 One-dimensional Models: Finite Difference Method (FDM)

The 1D site response analysis was conducted using NERA, a well-established finite difference-based code (FDM) for modelling wave propagation through soil layers under free-field conditions, excluding structural influences. NERA, an evolution of the “Equivalent Linear” EERA framework (Bardet et al., 2000), enhances seismic analysis by integrating FDM with nonlinear and hysteretic soil behaviour, thereby enabling time-domain simulations that capture key phenomena such as stiffness degradation and increased damping at high strain levels during seismic events. NERA simulates soil degradation under cyclic loading, with incorporation of the modified Masing criterion, which tends to overestimate the damping at high strain levels, potentially leading to discrepancies in the predicted dynamic response (Yamamoto et al., 2015). Because NERA is applied in a purely free-field context, any potential soil–structure interaction (SSI) effects cannot be considered in the 1D analysis. Another limitation of NERA is its inability to simulate ground profiles with non-horizontal layering stratigraphy or predict excess pore pressure accurately.

In this study, the nonlinear behaviour of soft marine sediments was characterized in NERA using modulus reduction and damping curves derived from laboratory cyclic tests, which were calibrated against the in-situ shear wave velocity measurements. Figure 3 presents the resulting normalized shear modulus (G/G_{max}) and damping ratio (ξ) as functions of shear strain for each stratigraphic sub-layer. The soil column was discretized into sub-layers with thicknesses ranging from 0.5 m in the near-surface zone to 4 m at greater depths, to account for the heterogeneity of the site profile in terms of stiffness and strength. While most of the available geotechnical data are limited to the upper strata – coverage typically sufficient for offshore foundation design – this study extends the depth of analysis to 75 m below mudline (BML) to meet the requirements of seismic site response modelling and to ensure compatibility with the characteristics of the selected input motions.

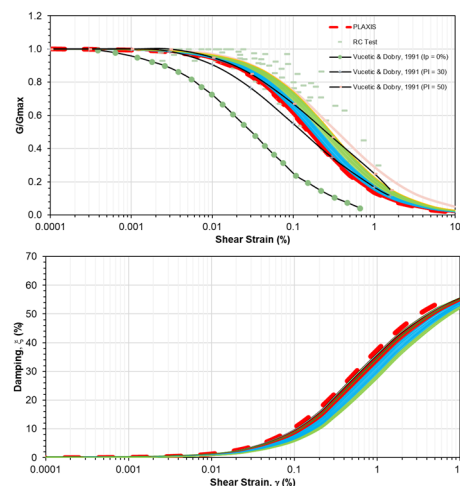


Figure 3. Normalized shear stiffness G/G_{max} (upper) and damping ratio (ξ) (lower) employed in NERA and PLAXIS.

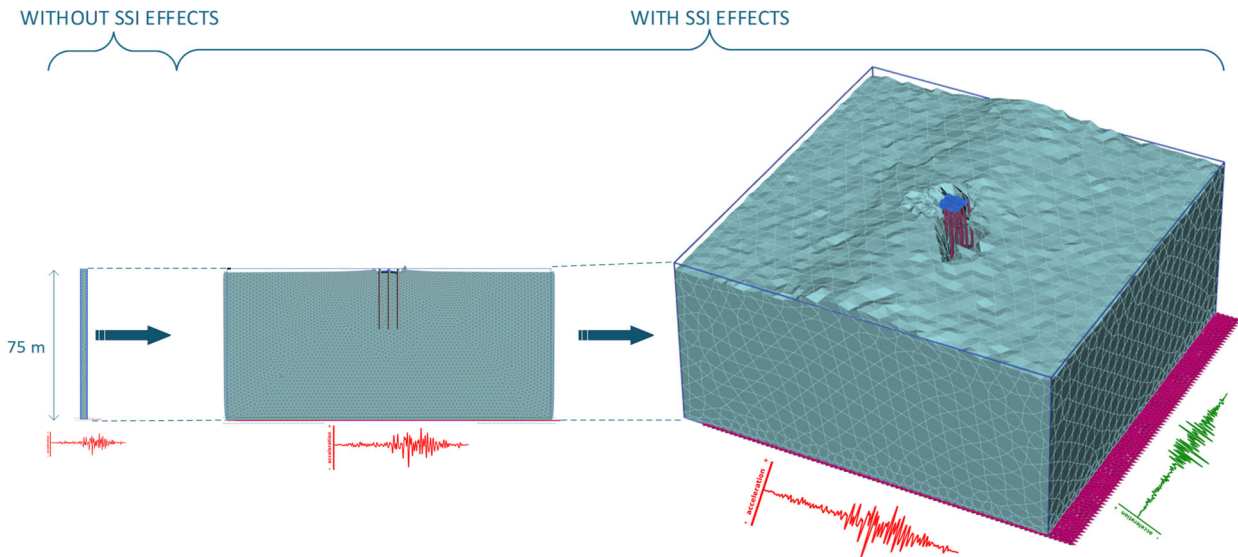


Figure 4. Illustration of SRA models in PLAXIS, featuring 1D SRA without SSI effects and 2D and 3D analyses with SSI effects.

4.2 Finite Element Method (FEM): One-, Two- and Three-Dimensional Models

PLAXIS (Bentley Systems, 2024) was employed to conduct the 1D (free field), 2D and 3D seismic analyses to account for soil-structure interaction (SSI) effects. In the 2D analysis, plane strain conditions were adopted to simulate the dynamic response of the layered marine sediments. Advanced constitutive models were adopted to capture the nonlinear stress-strain behaviour and strain-dependent damping characteristics of the soils (see Section 4.3.2). These features enabled a more realistic representation of geometric effects and near-field effects, which are beyond the capability of simplified 1D models.

The 3D analyses provided a fully spatial representation, allowing for the simulation of additional phenomena such as wave scattering, multidirectional stress redistribution, and complex dynamic SSI responses. Together, the 2D and 3D models offer valuable insight into the influence of model dimensionality on the predicted seismic response, thereby complementing the findings from the 1D analysis.

4.3 Model Setup and Boundary Conditions

The geometries of the FE models used in the SRA analysis are illustrated in Figure 4. The FE domain was discretized using 6-noded quadratic triangular elements for the plain strain 2D (and 1D) analysis, and 10-noded tetrahedron elements for the 3D model.

In the 1D PLAXIS SRA model, the soil column was modelled as a domain measuring 1.0 m in width by 75 m in depth. In the 2D PLAXIS model, the width was increased to 160 m, while the 3D PLAXIS model covered 160 m (width) \times 160 m (length) \times 75 m (depth). In all cases, the average element size was maintained smaller than or equal to one-eighth of the wavelength associated with the maximum frequency component of the input wave, which is sufficiently small to ensure accurate wave simulation and numerical stability (Kuhlemeyer and Lymer, 1973). A time step of 0.01 s was adopted in the calculation to ensure that, during a single increment, a wave does not move a distance greater than the minimum dimension of one element.

As in the 1D NERA SRA, the seismic loading was applied at the base of the PLAXIS model. A prescribed boundary displacement was then introduced, including compliant base boundary conditions to represent the dynamic source. This boundary and loading conditions were uniformly applied across all three modelling approaches for consistency.

4.3.1 Foundation Configurations

The foundation considered in this study included a 10 m \times 10 m mudmat with and without piles. In the piled configuration, the mudmat was supported by nine steel tubular piles, each 0.9 m in diameter, 20 mm wall thickness, and 30 m in embedded length. In the unpiled scenario, the mudmat was equipped with a peripheral steel skirt 30 mm thick, extending 350 mm below its base to enhance embedment depth and stability under seismic loading. These configurations were chosen to simulate typical offshore foundation designs and evaluate their impact on the seismic response. The inclusion of both piled and skirted foundations allowed assessing the potential benefits gained during seismic events.

4.3.2 Constitutive Models

Accurate prediction of seismic site response hinges on having constitutive models capable of simulating strain-dependent stiffness degradation, hysteretic and viscous damping, and pore pressure generation during cyclic loading. Conventional isotropic hardening models that lack strain-dependent modulus reduction and pore-pressure formulation become unreliable once earthquake-induced shear strains exceed the small-strain range; under such conditions, soils may experience marked modulus degradation and, in saturated sands and silts, cyclic mobility or liquefaction. Consequently, more advanced constitutive formulations – such as those documented by Brinkgreve (2021) – are required to simulate the full nonlinear soil response under dynamic loading. To explore these mechanisms, the analysis employed three PLAXIS material models, each selected to capture a distinct aspect of soil behaviour under seismic loading (Table 1) and show potential contrasting behaviour: the isotropic Hardening Soil model (HS), the small-strain Hardening Soil model (HSsmall) and the kinematic hardening UBC3D-PLM model (UBC3D). Their calibrated parameters are summarized in Table 2.

Table 1. Capabilities and limitations of the constitutive models adopted for seismic site response analysis

Models	Key Parameters	G/G _{max} Degradation	Hysteretic Damping	Key Capabilities	Main Limitations
Isotropic Hardening Soil (HS)	E_{50}, c', ϕ'	Good beyond the small zone	None	Stress-path-dependent stiffness and strength	No small-strain stiffness; purely effective-stress elastic-plastic, therefore no cyclic pore-pressure build-up
Hardening Soil with small-strain stiffness (HSsmall)	$G_0^{ref}, \gamma_{0.7}$	Good	Moderate	Adds G_0^{ref} and $\gamma_{0.7}$ to reproduce shear-modulus reduction and hysteretic damping at very small strains (and beyond)	Unable to account for the cumulative nature of excess pore pressure with shaking
UBC3D-PLM (UBC3D)	k^*, m, f	Good	Good	Kinematic hardening; excess pore-pressure generation and dissipation; liquefaction triggering	Requires rigorous calibration; originally formulated for granular soils. Therefore, applicability to silts requires careful consideration.

Table 2. Material properties resulting from calibration.

Soil Model	Drainage Type	γ_{sat} [kN/m ³]	c' [kPa]	ϕ' [°]	ϕ'_{cv}, ϕ'_p [°]	OCR [-]	E_{50}^{ref} [kPa]	E_{ur}^{ref} [kPa]	k_{Be}^* [-]	k_{Ge}^* [-]	k_{Gp}^* [-]	m_e, n_e, n_p [-]	m [-]	p_{ref} [kPa]	f_{dens} [-]	f_{Epost} [-]	R_f [-]	ν [-]
HS / HS small	Undr. A	15	0.5	14	-	1.0	2,600	8,000	-	-	-	-	0.9	30	-	-	-	0.2
UBC3D	Undr. A	15	0.5	-	14	-	-	-	770	810	770	0.85	-	30	1.0	0.5	0.9	-

Notes: γ_{sat} = saturated unit weight; c' = effective cohesion; ϕ' = effective friction angle; ϕ'_{cv} = constant volume effective friction angle; OCR = over-consolidation ratio; E_{50}^{ref} = secant vertical modulus in standard drained triaxial test at 50% strength; E_{ur}^{ref} = unloading / reloading stiffness; k_{Be}^* = elastic bulk modulus factor; k_{Ge}^* = elastic shear modulus factor; k_{Gp}^* = plastic shear modulus factor; m_e = rate of stress-dependency of elastic bulk modulus; n_e = rate of stress-dependency of elastic shear modulus; n_p = rate of stress-dependency of plastic shear modulus; m = power for stress-level dependency of stiffness; p_{ref} = reference pressure; f_{dens} = densification factor; f_{Epost} = post-liquefaction factor of; R_f = failure ratio; ν = Poisson's ratio.

The HSsmall model parameters G_0^{ref} and $\gamma_{0.7}$ were calibrated against the site-specific modulus reduction and damping curves that were derived from resonant column and cyclic triaxial testing (Figure 3). The selected parameters, $G_0^{ref} = 10$ MPa and $\gamma_{0.7} = 6$ %, reproduce the small-strain stiffness and hysteretic damping behaviour used in the NERA profiles, ensuring consistency between 1D and FE formulations. Rayleigh damping was incorporated with the HS model, which does not have hysteretic damping. By contrast, HSsmall intrinsically generates hysteretic damping at small strains (and beyond); therefore, no Rayleigh damping was applied in the base case, except in the sensitivity analyses discussed later.

For the UBC3D model, the elastic bulk and shear stiffness factors (k_{Be}^*, k_{Ge}^*) and plastic shear modulus factor (k_{Gp}^*) were calibrated using virtual cyclic DSS simulations (Figure 5), matched to the same monotonic backbone curve used for HSsmall. Parameters controlling cyclic mobility (m, m_e, n_e, n_p) were adjusted to reproduce the observed accumulation of pore pressure and stiffness degradation under undrained cyclic loading. Although UBC3D was originally formulated for granular soils, its cyclic-mobility formulation potentially makes it suitable for representing the behaviour of the site's carbonate-rich marine sediments, which is investigated here.

The influence of constitutive model selection on predicted acceleration spectra is examined in Section 5.2.

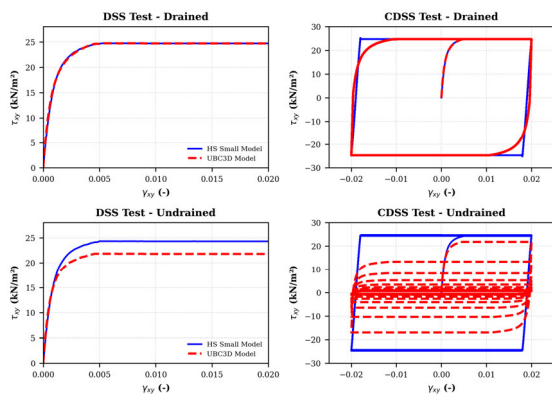


Figure 5. Calibration results of HSsmall and UBC3D models under both drained and undrained conditions.

5 RESULTS AND DISCUSSIONS

5.1 Comparative Free-Field Analysis of 1D, 2D, and 3D Approaches

Figure 6 compares free-field spectral accelerations at bedrock and mudline obtained from the 1D, 2D, and 3D analyses. All results shown here employ the HSsmall constitutive model (with no Rayleigh damping) and the ELE input motion (Figure 2). As expected, all modelling approaches indicate amplification from bedrock to mudline, a behaviour that is also evident in the mudline acceleration time histories (Figure 7); however, the magnitude and frequency content of this amplification differ notably with model dimensionality. In the mid-frequency range, the 1D and 2D models tend to predict higher amplification than the 3D model.

Across all modelling approaches, the mudline spectral acceleration S_a peaks at $\approx 0.4g$, whereas the corresponding input spectral value is about $0.15g$ for this particular earthquake, yielding an amplification factor of roughly 2.7 (Figure 6).

Interestingly, the two 1D methods (PLAXIS-FEM and NERA-FDM) produce nearly identical spectra at the fundamental period of 0.2s, with amplification ratio differing by less than 5 %, demonstrating that PLAXIS provides a consistent preliminary site-response analysis. In contrast, the 2D plain-strain model exhibits a similar peak but with a modest shift following 0.1 s later in the spectral shape. The 3D model shows an amplification level about 15% lower than those obtained from the 1D and 2D approaches, which lies within expected range of variation. It turns out that the reduced amplification in the 3D model stems from its ability to simulate multi-directional wave scattering, radial energy dispersion and spatial variability; phenomena that are absent in 1D or only partially captured in 2D.

Depth-wise comparisons of maximum shear strain (γ_{max}), maximum shear stress (τ_{max}), and peak ground acceleration (PGA) are given in Figure 8. Values at each depth correspond to the peak response recorded during the wave train. PLAXIS (1D) agrees well with NERA, although PLAXIS slightly underestimates γ_{max} between 5 m and 20 m depth and overestimates it above 5 m. A similar pattern is observed for

τ_{max} . The 3D simulation produces modestly lower τ_{max} (with a maximum difference of 30 %), reflecting a more realistic redistribution of stresses. PGA profiles are consistent among all three methods.

Overall, these comparisons highlight the trade-off between computational efficiency (1D/2D) and physical fidelity (3D). While lower-dimensional models remain useful for preliminary design, 3D analysis offers a more comprehensive representation of wave propagation and soil behaviour, particularly when soil–structure interaction and spatial variability are critical. The influence of constitutive choice and SSI effects is examined in the following sections.

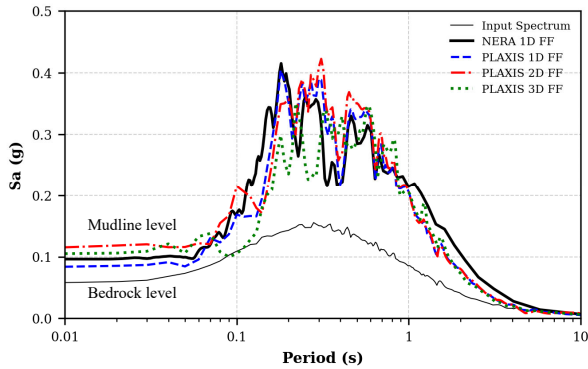


Figure 6. Comparison of spectral acceleration at mudline and bedrock levels from 1D, 2D, and 3D analyses, highlighting amplification trends and dimensional effects on shear wave propagation (FF = Farfield).

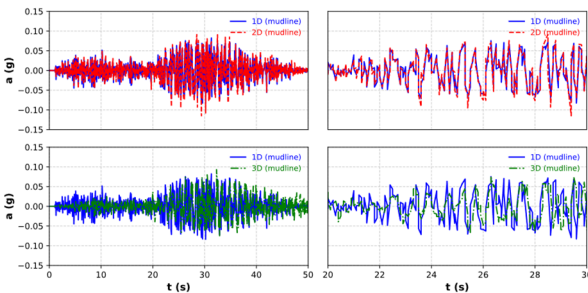


Figure 7. Time histories at the mudline for 1D, 2D, and 3D analyses: (left) full time series, and (right) 20–30 s window.

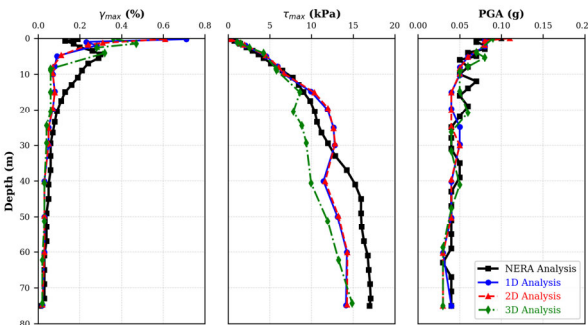


Figure 8. Comparison of maximum shear strain (γ), maximum shear stress (τ_{max}), and peak ground acceleration (PGA) profiles over depth for 1D, 2D, and 3D analyses.

5.2 Influence of Constitutive Model

Figure 9 contrasts the mudline response spectra computed with the HS, HSsmall and UBC3D models. For the same soil profile, UBC3D produces markedly lower spectral accelerations ($\approx 0.1g$ compared with $\approx 0.4g$ for HS/HSsmall), demonstrating it is not suitable for modelling the fine-grained silty material studied here. This limitation highlights a broader industry gap: the absence of an advanced cyclic constitutive model for

carbonate silts and clays implemented within a true 3D formulation. Such a model is essential for reliable seismic response prediction in these soil types.

The contrast between HS and HSsmall highlights the role of small-strain stiffness. This is because HSsmall includes the G_0 formulation, giving a stiffer initial response and hence shorter low natural periods. The standard HS model lacks G_0 and instead relies on constant reference moduli supplemented by Rayleigh damping, which influences peak spectral values only (for example, $\xi_{1,2} = 0.5\%$ at $f_1 = 1$ Hz and $f_2 = 3$ Hz, as shown in Figure 9); besides, the system becomes softer, and its spectral peak shifts to longer periods. Because the Rayleigh coefficients were calibrated at a single reference frequency, they over-damp the high-frequency range and under-damp the low-frequency range, further suppressing the HS spectral peak. Rayleigh damping adds viscous energy loss but does not capture the strain-dependent hysteresis intrinsic to HSsmall.

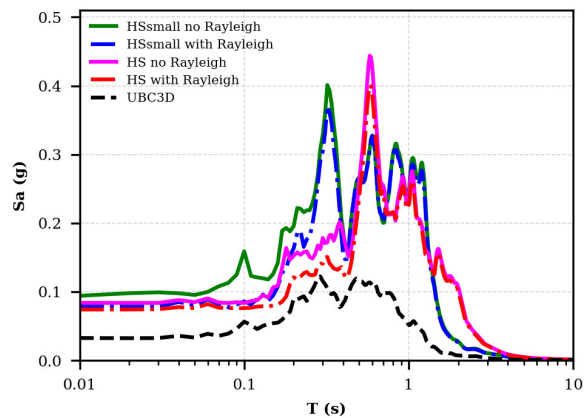


Figure 9. Effect of constitutive model on mudline response spectra.

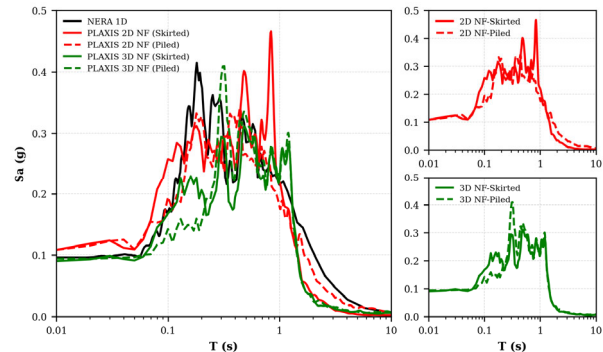


Figure 10. Impact of SSI on spectral acceleration, showing shifts in dominant periods and amplitudes across 1D, 2D, and 3D approaches.

5.3 Soil–Structure Interaction and Foundation Performance

Soil–structure interaction (SSI) can modify the propagation and redistribution of seismic energy within the near-field zone, altering both the amplitude and the frequency content of surface motions. Figure 10 compares spectral accelerations computed with and without embedded foundations and show how SSI affects resonant peaks and overall response characteristics.

In the far-field 1D NERA analysis, the response is governed typically by the layered soil column, producing a narrow resonance peak at the period $T_0 \approx 0.2$ s. Incorporating foundations in 2D and 3D PLAXIS models broadens this response: multiple peaks appear because lateral and out-of-plane wave propagation and additional inertial effects are now present. As a result, 2D simulations exhibit pronounced peaks at mid-range frequencies, whereas 3D simulations distribute energy across a wider frequency band and reduce peak

accelerations, consistent with the multidirectional scattering effects described in Section 5.1.

The choice of foundation system further influences the response. For the studied ground profile, piled foundations increased the stiffness of the soil–foundation system, shifting certain resonant frequencies to shorter periods and introducing additional radiation and hysteretic damping through group interaction and soil-pile coupling. Skirted foundations, although generally stiffer than free-field conditions, remain more flexible than piled systems and therefore shift dominant periods slightly longer. The relative effectiveness of each foundation type depends on how the resulting system period aligns with, or avoids, the dominant periods of the soil column.

Figure 11 presents the horizontal mudline displacements for the skirted and piled mudmats. The lateral response of the unpiled mudmat is overall comparable with the piled case, which oscillates around a less drifted mean value compared with the unpiled mat. The difference in the response is attributed to the change in mass and stiffness the pile imparts to the relevant soil column. As expected, the piles clearly do not mitigate the lateral movement but changes the pattern. The difference in the maximum lateral movement shown in Figure 11 is approximately within 10 mm.

From a design standpoint these results reinforce the benefits of advanced SSI modelling. While 1D free-field analyses remain useful for preliminary screening, 2D and 3D near-field models provide a more realistic assessment of energy distribution and resonance phenomena. Selecting between piled and skirted foundations should balance stiffness, damping potential, constructability, and cost to achieve a safe and economical design that meets the full spectrum of seismic demands.

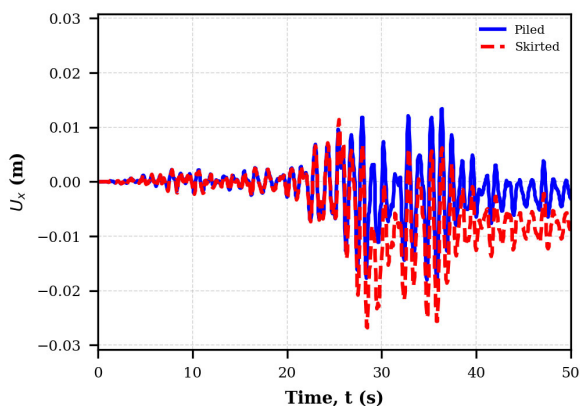


Figure 11. Horizontal mudline displacements for skirted and piled mudmats under seismic loading (3D - HSsmall).

6 CONCLUSIONS

This study explains and demonstrates that advanced 3D finite-element analyses improve the accuracy of offshore seismic site response predictions compared to simpler one- and two-dimensional methods. Constitutive models capable of simulating cyclic strength degradation, pore-pressure buildup, and strain-dependent damping - such as the HSsmall are essential for realistic seismic evaluations, assuming liquefaction is unlikely to occur. However, the HSsmall still suffers from the limitation of its inability to accumulate excess pore pressure during shaking.

Incorporating realistic soil–structure interaction effects may considerably influence the resultant seismic response spectrum, depending on the nature of the structure; this emphasizes the necessity of integrating comprehensive soil behaviour in offshore foundation design, to enable

optimisations and ensure reliability. Further studies are underway to model more complex foundation configurations and additional seismic scenarios to improve on more conventional offshore design methodologies.

7 ACKNOWLEDGEMENTS

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