



Influence of soil nonlinearity on the seismic behaviour of monopile foundations

F. Ortiz-Wall*

Imperial College London, London, UK

S. Kontoe

University of Patras, Patras, Greece; Imperial College London, London, UK

D.M.G. Taborda

Imperial College London, London, UK

J. K. Möller

Wood Thilsted, London, UK; Imperial College London, London, UK

**f.ortiz-wall23@imperial.ac.uk*

ABSTRACT: The growth of the offshore industry has led to the installation of offshore wind turbines (OWTs) in seismic regions, adding complexity to their design. Seismic loads can impose significant deformation on the soil, resulting in highly nonlinear behaviour. Designing structural systems for these conditions requires complex methodologies for the adequate prediction of their behaviour, such as finite element modelling (FEM) coupled with constitutive models that accurately reproduce the cyclic soil response – an often-challenging task. This study employs dynamic FEM with a cyclic nonlinear elastic model with enhanced hysteresis control to analyse the impact of nonlinear soil behaviour on soil-structure interaction (SSI) under seismic loading. Additionally, equivalent linear (EQL) site response analyses are used to retrieve shear wave velocity and damping profiles of the soil under seismic conditions and inform a new set of numerical analyses. Significant differences were observed between the predictions of the two approaches, influenced by the soil's dynamic properties and seismic intensity. For the pile geometry, soil profile and load cases considered in this study, EQL models appears suitable as a conservative first approximation for design purposes.

Keywords: Nonlinear Soil Behaviour; Equivalent Linear; Monopile Foundation, Finite Element Modelling.

1 INTRODUCTION

Offshore wind energy has grown steadily over recent decades, leading to the development of projects in seismically active regions, where designs must now address the added complexity of seismic loads on both OWTs and their foundations. To date, seismic design standards for OWT foundations have largely remained reliant on the traditional p-y method, which is not directly applicable to large-diameter monopile foundations (Burd et al., 2020). As a result, finite element modelling has become a robust and widely recommended approach to verify the behaviour of the designed system (DNV, 2021). Despite this, FEM comes with high computational costs and, more importantly, capturing accurately complex cyclic soil behaviour remains a challenge, as the selected soil constitutive model needs to be able to reproduce the nonlinear soil behaviour under repeated and irregular loading.

This paper explores two approaches to assess the impact of nonlinear soil behaviour on the seismic

response of monopile foundations: first, by employing a cyclic nonlinear constitutive model with enhanced hysteresis control, and second, by implementing a simplified equivalent linear model to approximate the nonlinear soil behaviour, requiring fewer soil parameters, and allowing for faster calculations.

2 NUMERICAL MODEL

2.1 Model geometry and properties

The 15-MW offshore reference wind turbine (Gaertner et al., 2020) was considered in this study and was modelled in PLAXIS 3D (Bentley Systems, 2024). It consists of a 180 m high tower (above mudline level) supported by a monopile foundation with an embedment depth $L = 45$ m and a diameter $D = 10$ m (i.e. $L/D = 4.5$). The first turbine-monopile mode is reported as 0.17 Hz, while the second mode was estimated computationally, as presented in section 3.2.2.

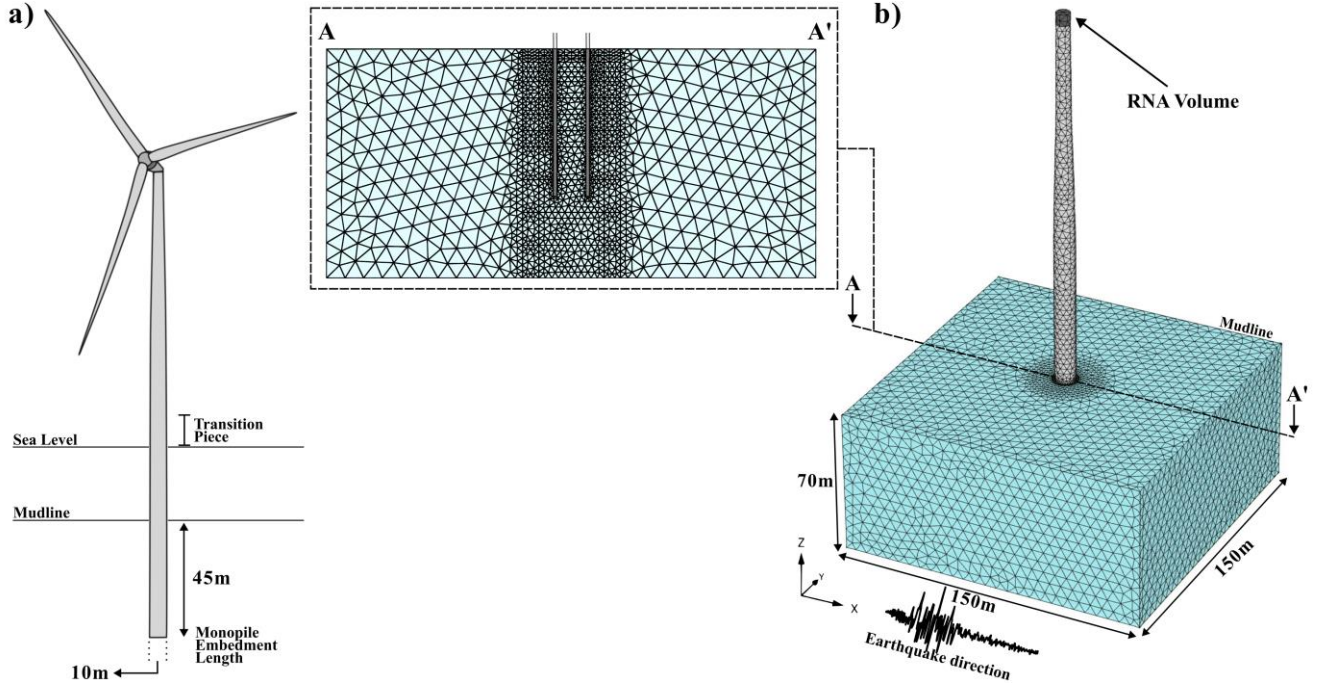


Figure 1. a) Sketch of the IEA 15-MW offshore reference wind turbine, and b) model geometry and mesh in PLAXIS3D

Both the steel tower and monopile were modelled using 6-noded plate elements, as a linear-elastic material with Young's modulus $E = 200 \text{ GPa}$, Poisson's ratio $\nu = 0.28$ and density $\rho = 7850 \text{ kg/m}^3$. The tower diameter (D_t) and wall thickness (t_t) vary with height, with values of $D_t = 10 \text{ m}$ and $t_t = 55.3 \text{ mm}$ at the base, reducing to $D_t = 6.5 \text{ m}$ and $t_t = 24 \text{ mm}$ at the top just before reaching the rotor-nacelle assembly (RNA), while the monopile is simulated with a fixed thickness of $t_m = 55.3 \text{ mm}$. The RNA, blades and other components have been modelled as a linear-elastic steel volume (same E and ν as plates) of equivalent mass at the top of the tower. An additional 7% of outfitting mass was added to all plate elements and another 100t were added specifically to the transition piece (see Gaertner et al. (2020) for more details).

The soil domain extends to 75 m on each side of the monopile, with a total depth of 70 m, and was modelled with approximately 93000 10-noded tetrahedral elements. A maximum average element size of one-tenth of the wavelength of the body waves is enforced by using horizontal surfaces in the vicinity of and below the monopile, satisfying the recommendations for element size given in Kuhlemeyer & Lysmer (1973). Preliminary tests showed that the system response was largely unaffected by using a finer mesh discretisation in areas away from the monopile, enabling substantial savings in the required computational cost. The final mesh used can be seen in Figure 1b.

To model the soil-structure interaction, 12-noded interface elements were placed around the monopile.

For the nonlinear models, the interface stiffness is based on the estimated one-dimensional stiffness of the surrounding soil $E_{oed}(\sigma'_n)$ (Bentley Systems, 2024), which is stress-dependent and given by a power law as:

$$E_{oed}(\sigma'_n) = 2.48 \times 10^5 [\text{kPa}] \cdot \left(\frac{\sigma'_n}{100 \text{ kPa}} \right) \quad (1)$$

where σ'_n is the effective normal stress in the interface stress point. For the equivalent linear models, the interface stiffness is automatically determined from E and ν (Bentley Systems, 2024). In both cases, the adoption of a limiting tensile stress of 0 kPa for the interface elements enable the modelling of the loss of contact between the soil and the structure.

Dynamic free-field boundary conditions were applied on all four lateral borders of the model to prevent wave trapping into the discretised domain. This is done by placing free-field elements, matching the adjacent soil properties, on the lateral sides of the model. These elements transfer the free-field motion to the main domain by applying equivalent normal and shear forces, and absorption of the reflected waves coming from inside the model is achieved by adding dashpots in both shear and normal directions to each node (Bentley Systems, 2024). These boundary conditions allow for consistent results with those expected from wave propagation theory (Azeiteiro et al., 2023). Conversely, at the base of the model, perfectly rigid bedrock conditions have been assumed.

The non-dissipative version of the Newmark time integration method (Newmark, 1959) has been adopted with parameters $\alpha = 0.25$ and $\beta = 0.50$.

2.2 Soil modelling

A normally consolidated clayey soil profile with linear variation with depth of the small-strain shear modulus G_{max} was considered in this study (see Figure 3a). Unit soil weight, Poisson's ratio, plasticity index and coefficient of earth pressure at rest were taken as $\gamma = 17 \text{ kN/m}^3$, $\nu = 0.3$, $PI = 15\%$ and $K_o = 1$, respectively. Effective stress analyses were carried out assuming the soil behaved in an undrained manner.

2.2.1 Cyclic nonlinear model

A cyclic nonlinear isotropic model which adopts a modified hyperbolic backbone curve, combined with a Mohr-Coulomb failure criterion and a nonlinear scaling factor, allowing for an enhanced hysteresis control of the soil response (Taborda et al., 2016; Taborda & Zdravkovic, 2012), designated IC MAGE M04, has been implemented as a user-defined model in PLAXIS (Taborda et al., 2023) and is used herein. One key feature of this model is that it includes an implementation of the algorithm proposed in Taborda et al. (2016), which allows the continuous estimation of the damping ratio at each integration point in the mesh.

To allow for meaningful comparisons with the second approach (see Section 2.2.2) plasticity is deactivated by setting a large cohesion value c . The full set of model parameters adopted for the clay layer are presented in Table 1. These were selected to match the reference damping curve proposed by Darendeli (2001), as seen in Figure 2.

Table 1. IC MAGE M04 Parameters for clayey soil profile.

Soil Parameters			
G_{ref}	50127 kPa	c	10 MPa
p'_{ref}	100 kPa	a_0	1.921×10^{-4}
m_G	1	$d_{G,11}$	135.42
b	1.07	$d_{G,2}$	0.203
RG_{min}	0	$d_{G,3}$	7907.73
G_{min}	1 MPa	$d_{G,4}$	0.661

2.2.2 Equivalent Linear Model

Equivalent linear analyses are frequently adopted to approximate the nonlinear behaviour of soils under seismic loading by assuming a constant stiffness and damping established based on a reference level of strain. This is done by estimating the level of deformation in each layer for a given motion, using initial values for the stiffness and damping ratio. Once

calculated, the strain is used to update the stiffness and the damping accordingly, in this case, by using the curves proposed in Darendeli (2001). These values are then used in subsequent analyses to re-calculate the strain level, leading to an iterative process until convergence.

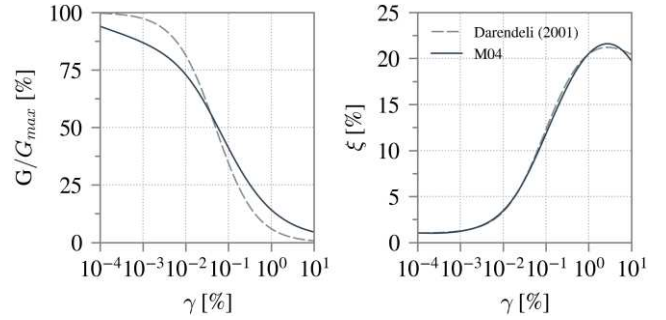


Figure 2: Shear modulus and damping ratio response of model IC MAGE M04 using $PI=15\%$, $OCR=1$, $Freq=1\text{Hz}$ and $N=10$ cycles

A series of 1D site response analyses were carried out for two seismic motions (see Section 2.3) with the software Strata (Kottke & Rathje, 2008) for the stepped V_s profile shown in Figure 3a, considering the reference curves shown in Figure 2, and an effective strain of 65% of the maximum strain. As a result, V_s and damping profiles are obtained (see Figure 3b). The latter serve as input for linear visco-elastic analyses in PLAXIS 3D, with nine soil layers. Rayleigh damping is imposed between 0.17 Hz (turbine's natural frequency) and 3.0 Hz (limit of the motion's frequency content in the range of interest) targeting the damping profile of Figure 3b.

2.3 Input motion

The models were subjected to a seismic motion (EW direction) recorded during the Kobe earthquake (1995) at Kobe University (PEER, 2014), with peak ground acceleration, $PGA = 0.31 g$ (hereinafter referred to as Standard "St" motion). A second signal is obtained by downscaling the motion to $PGA = 0.10 g$ (hereinafter referred to as Downscaled "Dwn" motion) to investigate the influence of motion intensity on the response of the system. In both cases, a quadratic baseline correction is applied using the software SeismoSignal (Seismosoft, 2016), so that the final displacement is close to zero. No further changes were made to the ground motions.

The motions were applied in the horizontal direction $\hat{x}$ at the base of the model and the time-histories, as well as Fourier amplitude spectra are included in Figure 5 to Figure 8. It is important to note that the predominant frequency range of the earthquake motion is around 0.6 Hz – 0.95 Hz.

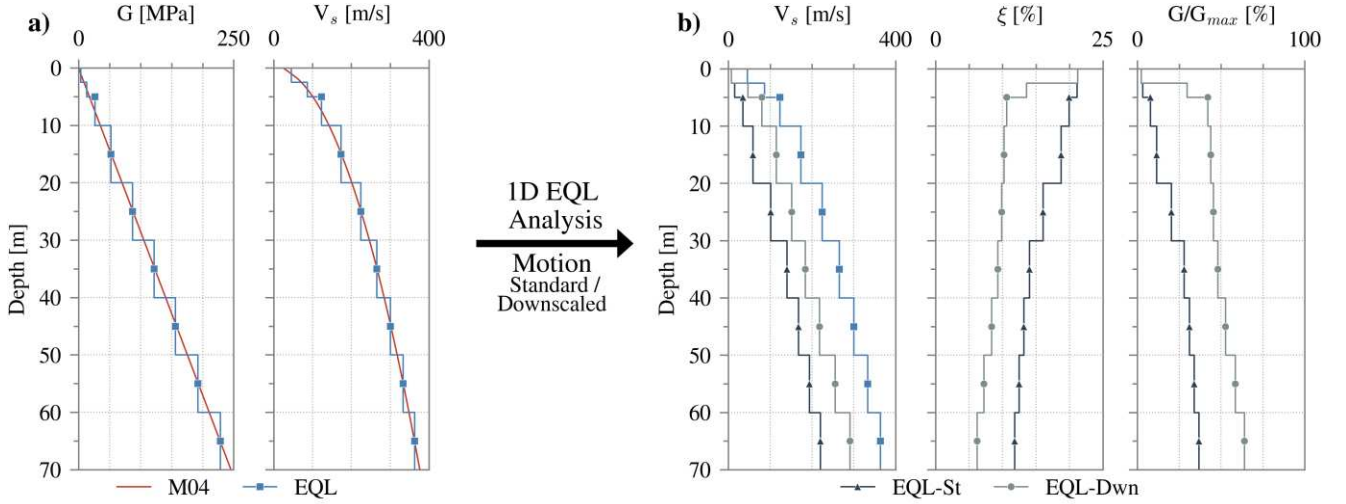


Figure 3. a) Clayey soil profile considered for M04 model and layered approximation for EQL 1D analysis, and b) retrieved shear wave velocity V_s , damping ξ , and normalised shear stiffness profiles after EQL analysis for standard (St) and downscaled (Dwn) motions

3 RESULTS AND DISCUSSION

3.1 Stiffness and damping

The simulated normalised shear modulus G/G_{max} and damping ratio ξ for both standard and downscaled motion simulations for a point in the vicinity of the monopile (at 10 m depth) are illustrated in Figure 4.

As expected, the soil stiffness in the nonlinear models decays as the strain level increases. Strain levels are higher in the case of the standard motion than the downscaled motion, which leads to lower normalised stiffness. The damping ratio is constantly updated over time depending on the induced strain level in the ground at that precise time instant, reaching, as expected, higher overall values for the standard motion case.

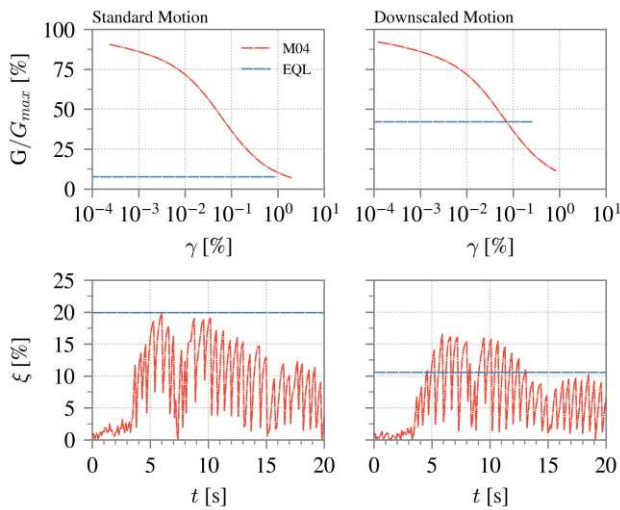


Figure 4. Normalised shear modulus over strains and damping ratio over time for a point 10 cm away from the monopile and 10m depth

On the other hand, EQL models show (inherently) a constant value of the normalised shear stiffness and damping ratio, although the values do change between simulations of different motions, something that was also shown in Figure 3. Strain levels also change between simulations, with higher values for the standard case, as expected.

The comparison of the modelling approaches shows that for high-intensity events the difference between the two methodologies is considerable, with the EQL model predicting stiffness values well below those predicted by the nonlinear model, while the opposite is true for the damping ratio. For medium-intensity events (and presumably low-intensity ones too), the EQL approach is closer to the nonlinear one both in terms of shear stiffness and damping ratio. These findings are consistent with previous comparisons of similar approaches for site response analyses (e.g. Papaspiliou et al., 2012).

3.2 Acceleration histories and Frequency content

3.2.1 Monopile Response

The obtained acceleration time histories and the corresponding Fourier amplitude spectra at the monopile head for both motions are shown in Figure 5 and Figure 6.

In terms of accelerations, both methods yield comparable magnitudes and exhibit similar behaviour, showing a significant increase in intensity after 3 s – 4 s, followed by a decay around 13 s – 14 s; however, the EQL models produce higher peak values than the nonlinear model on many occasions.

Furthermore, frequency ranges where the Fourier amplitudes are larger in both simulations tend to be within the interval between 0.55 Hz and 1.0 Hz,

which coincides with some of the predominant frequencies of the motion. This alignment is expected as the natural frequencies of the soil deposit for the EQL models are also in the range of $0.40 \text{ Hz} - 0.72 \text{ Hz}$ and $0.58 \text{ Hz} - 0.88 \text{ Hz}$ for the standard and downscaled simulation, respectively, while the initial natural frequency of the deposit for the nonlinear simulations (before any stiffness reduction occurs) is around 1.02 Hz , which should decrease, approaching the main frequency range of the input signal as the soil softens during the motion. This is very important, as the alignment of these frequency peaks might induce an amplified response in the foundation and tower system, explaining the increase in Fourier amplitude within these frequency bands, compared to ranges where $f > 1 \text{ Hz}$.

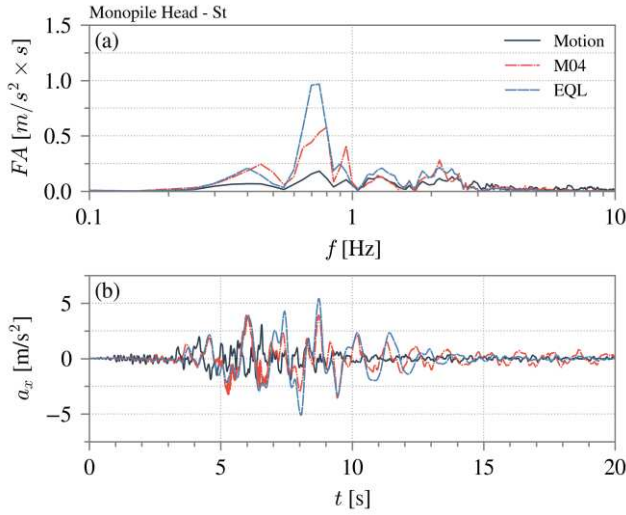


Figure 5. a) FA spectrum and b) acceleration traces at the monopile head for the standard motion simulation

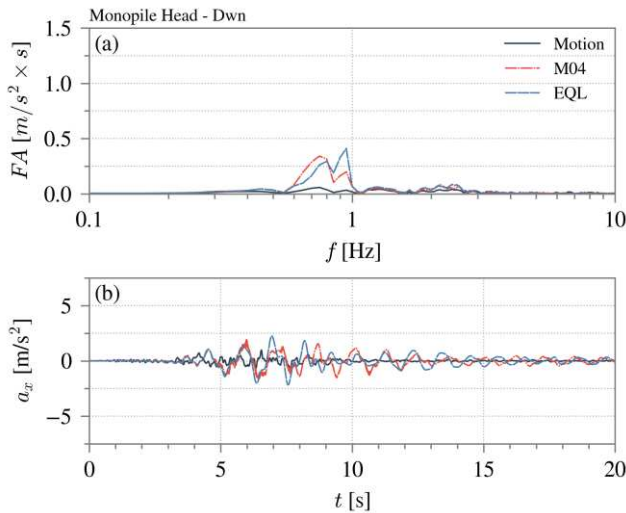


Figure 6. a) FA spectrum and b) acceleration traces at the monopile head for the downscaled motion simulation

3.2.2 Tower Response

As mentioned previously, only the first mode of the turbine-monopile is reported in Gaertner et al. (2020). The second mode was estimated by imposing a sine sweep motion with a range of frequencies from 0.1 Hz to 5 Hz . For the fixed base case, the second mode is at 1.26 Hz . Clearly, for any other condition, soil properties will influence this value: for the EQL models, the second modes are at 0.83 Hz and 0.93 Hz for the standard and downscaled motion cases, respectively. The second mode was not determined for the nonlinear models, as the selection of the amplitude of the sine sweep would heavily impact the result due to the changes in soil stiffness and damping ratio over time.

The obtained acceleration time histories and Fourier amplitude spectra at RNA level for both motions are shown in Figure 7 and Figure 8.

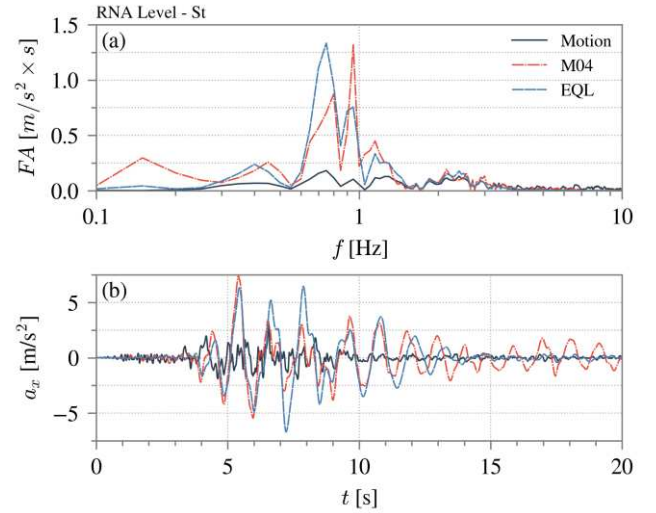


Figure 7. a) FA spectrum and b) acceleration traces at RNA level for the standard motion simulation

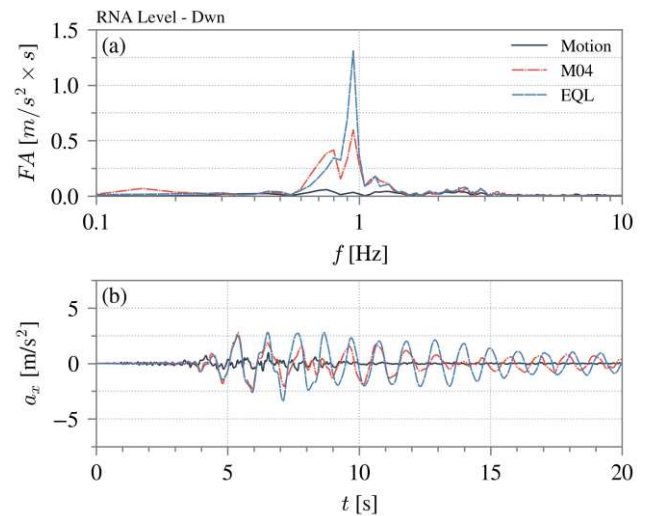


Figure 8. a) FA spectrum and b) acceleration traces at RNA level for the downscaled motion simulation

For the standard motion, similar findings to those mentioned above for the monopile head response can be observed. In contrast, the results for the downscaled motion simulation differ from those observed previously. In this scenario, the acceleration amplitudes of the EQL model remain consistently higher than those of the nonlinear model throughout the entire event, with minimal decay over time. For this method, the pronounced periodicity around 0.95 Hz controls the response, suggesting that the tower is experiencing resonance on its second mode during a sustained time interval, as the properties of the soil deposit remain unchanged.

This resonance effect is also visible, albeit to a lesser extent, in the nonlinear model, where it appears to be more transient. In this case, the system recovers more easily, likely due to the material's varying stiffness under different deformation levels, which can shift the natural frequency of the deposit over time.

3.3 Bending Moments

Bending moments along the monopile were calculated at each time step by using nodal displacements to determine rotation profiles with depth, which enable the calculation of the curvature of each subsection of the monopile. Subsequently, the maximum absolute values were identified to establish an envelope covering all profiles over the full motion duration. The resulting profiles are shown in Figure 9.

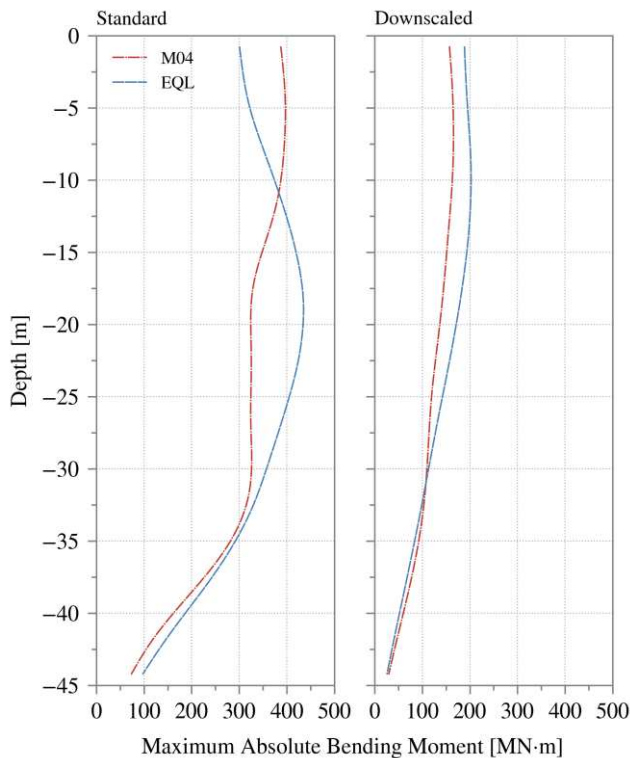


Figure 9. Maximum absolute bending moment along the length of the monopile for both simulations

Predictions of the EQL model generally show higher bending moments than those in the nonlinear model. Maximum values in the standard motion simulation are $399\text{ MN}\cdot\text{m}$ for the nonlinear model and $438\text{ MN}\cdot\text{m}$ for the EQL model (an increase of about 9.8%), while in the downscaled motion simulation the maximum values are $166\text{ MN}\cdot\text{m}$ and $204\text{ MN}\cdot\text{m}$, respectively (an increase of about 23%). When considering the entire monopile depth, the EQL model predicts bending moments that differ, on average, by 17.3% for the standard record and by 17.5% for the downscaled record. In the case of the higher-intensity motion, the equivalent-linear analysis returns larger bending moments for depths ranging from approximately 10 m to 45 m , with lower values predicted in the top 10 m . For the lower-intensity input motion, bending moments predicted by the EQL analysis are consistently larger than those resulting from the nonlinear analysis.

4 CONCLUSIONS

The impact of soil nonlinearity on monopile foundations is studied following two distinct approaches based on the finite element method: (i) a cyclic nonlinear constitutive model, and (ii) a simplified equivalent linear visco-elastic model.

The EQL model predictions are comparable to those of the nonlinear model regarding accelerations, frequency content, and bending moments, though the former consistently produces larger magnitudes of these quantities.

Caution should be taken when the fundamental frequency of the seismic event aligns with that of the deposit or the tower, as this can lead to resonance in the system. EQL models are particularly susceptible to this because their soil's dynamic properties remain constant over time, making it challenging for the system to break out of this pattern of amplified oscillations which in turn result in the over-prediction of the response. In contrast, in the nonlinear analysis, this phenomenon is not as dominant; the model stiffness is constantly adjusted to the induced deformation level preventing the overpowering dominance of a specific frequency, leading to a more controlled response.

Moreover, as the intensity of the seismic motion increases, the difference between the two examined methodologies becomes more pronounced. For the higher intensity motion, the EQL analyses show significantly reduced stiffness and increased damping values, along with a worse match in terms of the bending moment profile.

In conclusion, accounting for soil nonlinearity leads to significant differences compared to the corresponding equivalent linear approach. However, due to the simplicity of linear models, they may serve as an initial estimate, which for the examined case appeared conservative, for assessing the system response. Further study is needed to determine whether this trend holds for other types of soil deposits and varying seismic loads.

AUTHOR CONTRIBUTION STATEMENT

F. Ortiz-Wall: Conceptualisation, Formal Analysis, Methodology, Writing – Original draft. **S. Kontoe:** Supervision, Methodology, Writing – Review & Editing. **D.M.G. Taborda:** Supervision, Methodology, Writing – Review & Editing. **J.K. Möller:** Supervision, Writing – Review & Editing.

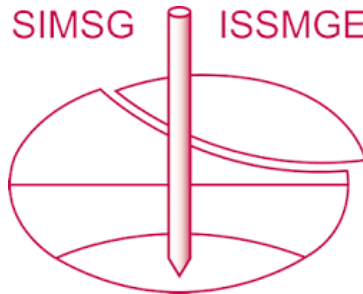
ACKNOWLEDGEMENTS

Support for this research was provided by the National Agency for Research and Development (ANID) and its overseas programme DOCTORADO BECAS CHILE/2023 – 72230056.

REFERENCES

- Azeiteiro, R. J. N., Brasile, S., & Bui, T. A. (2023). Importance of free field boundary conditions to the finite element modelling of surface topography effects on incident waves. 10th European Conference on Numerical Methods in Geotechnical Engineering (NUMGE2023). <https://doi.org/10.53243/NUMGE2023-331>
- Bentley Systems. (2024). PLAXIS 3D - Reference Manual.
- Burd, H. J., Abadie, C. N., Byrne, B. W., Houlsby, G. T., Martin, C. M., McAdam, R. A., Jardine, R. J., Pedro, A. M. G., Potts, D. M., Taborda, D. M. G., Zdravković, L., & Andrade, M. P. (2020). Application of the PISA design model to monopiles embedded in layered soils. *Géotechnique*, 70 (11), 1067–1082. <https://doi.org/10.1680/jgeot.20.PISA.009>
- Darendeli, M. B. (2001). Development of a new family of normalized modulus reduction and material damping curves. PhD Thesis, Austin: University of Texas.
- DNV. (2021). DNV-ST-0126 Support structures for wind turbines.
- Gaertner, E., Rinker, J., Sethuraman, L., Zahle, F., Anderson, B., Barter, G., Abbas, N., Meng, F., Bortolotti, P., Skrzypinski, W., Viselli, A., Scott, G., Feil, R., Bredmose, H., Dykes, K., Shields, M., Allen, C. & Viselli, A. (2020). Definition of the IEA 15-megawatt offshore reference wind turbine. National Renewable Energy Laboratory (NREL). <https://www.nrel.gov/docs/fy20osti/75698.pdf>
- Kottke, A. R., & Rathje, E. M. (2008). Technical manual for Strata. PEER report 2008/10, pacific earthquake engineering research center, college of engineering. University of California, Berkeley.
- Kuhlemeyer, R. L., & Lysmer, J. (1973). Finite Element Method Accuracy for Wave Propagation Problems. *Journal of the Soil Mechanics and Foundations Division*, 99(5), 421–427. <https://doi.org/10.1061/JSFEAQ.0001885>
- Newmark, N. M. (1959). A Method of Computation for Structural Dynamics. *Journal of the Engineering Mechanics Division*, 85(3), 67–94. <https://doi.org/10.1061/JMCEA3.0000098>
- Papaspiliou, M., Kontoe, S., & Bommer, J. J. (2012). An exploration of incorporating site response into PSHA—Part I: Issues related to site response analysis methods. *Soil Dynamics and Earthquake Engineering*, 42, 302–315. <https://doi.org/10.1016/J.SOILDYN.2012.06.011>
- PEER. (2014). NGA-West2 Database. Pacific Earthquake Engineering Research Center, University of California Berkley. <https://ngawest2.berkeley.edu/>
- Seismosoft (2016) “SeismoSignal - A computer program for signal processing of time-histories”. www.seismosoft.com
- Taborda, D. M. G., Kontoe, S., & Tsiampousi, A. (2023). IC MAGE Model 04 – Mohr-Coulomb failure criterion with isotropic small strain stiffness and enhanced hysteresis control. Zenodo. <https://doi.org/10.5281/zenodo.8430797>
- Taborda, D. M. G., Potts, D. M., & Zdravković, L. (2016). On the assessment of energy dissipated through hysteresis in finite element analysis. *Computers and Geotechnics*, 71, 180–194. <https://doi.org/10.1016/J.COMPGE0.2015.09.001>
- Taborda, D. M. G., & Zdravkovic, L. (2012). Application of a Hill-Climbing technique to the formulation of a new cyclic nonlinear elastic constitutive model. *Computers and Geotechnics*, 43, 80–91. <https://doi.org/10.1016/J.COMPGE0.2012.02.001>

INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

<https://www.issmge.org/publications/online-library>

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

The paper was published in the proceedings of the 5th International Symposium on Frontiers in Offshore Geotechnics (ISFOG2025) and was edited by Christelle Abadie, Zheng Li, Matthieu Blanc and Luc Thorel. The conference was held from June 9th to June 13th 2025 in Nantes, France.