

Numerical prediction of the effects of rocking motion and soil nonlinearity on the performance of full-scale experimental prototypes on shallow foundations

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ABSTRACT: Over the past thirty years, research on soil–foundation–structure interaction (SFSI) has highlighted that conventional fixed-base structural models are inadequate to reproduce the seismic performance of structures when the subsoil is deformable. SFSI leads to a modification of the structural response, consisting in a lengthening of the natural period of vibration, an increase in the energy dissipation capacity, and a growth in structural displacement due to the foundation rocking and swaying. Experimental investigations on scaled or full-scale prototypes help to reduce the uncertainty in quantifying these effects, particularly concerning the nonlinear response of the soil–foundation system. This paper first compares the experimental data and the numerical simulations of forced vibration tests executed on the full-scale prototype EUROPROTEAS, located in the Euroseistest TST site in Greece, within SISIFO (Seismic Impedance for Soil–structure Interaction from On-site tests). The observed nonlinear response of the soil–foundation system, even under relatively low levels of dynamic excitation, motivated the design of a smaller prototype, named PERSEFONE, with different geometrical properties to be tested on the same site under higher levels of forces. Given the good agreement obtained between numerical simulations and experimental data in the SISIFO project, analogous numerical models have also been developed for the PERSEFONE prototype, enabling a preliminary prediction of its response. Finally, a comparison of the different mechanisms characterizing SFSI effects in the two prototypes is performed on the basis of the numerical simulations.

KEYWORDS: Soil–foundation–structure interaction, shallow foundation, nonlinear analysis, energy dissipation, full-scale tests.

1 INTRODUCTION

Over the past three decades, research on Soil–Foundation–Structure Interaction (SFSI) has highlighted the inadequacy of conventional fixed-base structural models in accurately reproducing the seismic performance of structures founded on deformable soils. The effects associated with SFSI are nonlinear even under low excitation, due to the hysteretic behavior of the foundation soil, which is affected by several uncertainties. One of these effects is the increase in damping which has recently been recognized as beneficial, as it can lead to a reduction in seismic demand on both the superstructure and the foundation (Gazetas, 2019). Experimental investigations on reduced-scale or full-scale prototypes (e.g., Gavras et al., 2020) play a key role in reducing the uncertainty associated with these effects. Moreover, field testing of full-scale structures provides realistic insights into interaction mechanisms, overcoming limitations of laboratory-scale physical models by inherently capturing radiation damping related to the actual subsoil conditions.

The two full-scale prototypes presented in this study were developed within the framework of the ERIES project (*Engineering Research Infrastructures for European Synergies*). The first prototype, named EUROPROTEAS and shown in Figure 1a, was tested in the SISIFO project (*Seismic Impedance for Soil–Structure Interaction From On-site Tests*), which consisted of an extensive experimental campaign including ambient noise measurements, free- and forced-vibration tests (Amendola et al., 2021). More recently, a new prototype, shown in Figure 1b, was designed for the PERSEFONE project (*Permanent Effects in Rocking Shallow or Embedded Foundations Occurring after Earthquakes*), with testing scheduled for September 2025 at the same experimental site. This new structure is slender and smaller, designed to enhance the rocking motion of the foundation and study the consequent permanent effects. To assist the design process and quantify the induced nonlinear effects, a 2D numerical model was developed using the finite difference code FLAC (Itasca, 2023). To validate the subsoil model, the experimental results obtained from the forced-vibration tests of the SISIFO prototype were firstly reproduced numerically. Once validated, the numerical model was used to simulate the expected

response of the PERSEFONE prototype, allowing for the prediction of the nonlinear effects mobilized under forced-vibration tests.

2 EXPERIMENTAL SETUPS

The original EUROPROTEAS prototype (Figure 1a) is a symmetric and reconfigurable frame structure, consisting of two portable reinforced concrete slabs supported by four square hollow steel (SHS), connected by L-shaped steel cross-braces. The structure rests on a foundation slab identical to each of the upper slabs, without embedment. The foundation has a square plan with a width and length of 3.0 m each, and a thickness of 0.4 m; the total height of the structure is 5.0 m. The fixed-base natural frequency, estimated through the finite element software SAP2000, is 9.13 Hz.

The design of the PERSEFONE prototype resulted in a similar symmetric and reconfigurable frame structure (Figure 1b), consisting of four portable reinforced concrete slabs, each one 0.2 m thick, supported by four circular hollow steel (CHS) columns. The foundation has a rectangular plan with a width of 0.8 m, a length of 1.3 m, and a thickness of 0.4 m; the total height is equal to 4.0 m. The fixed-base natural frequency, again estimated through SAP2000, is 2.56 Hz.

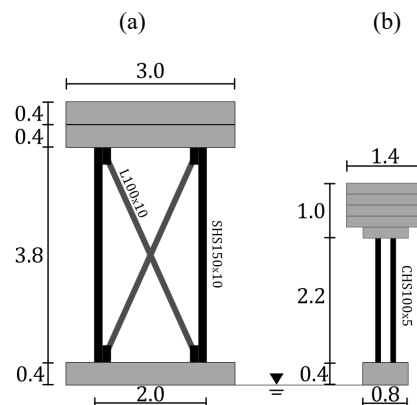


Figure 1. EUROPROTEAS (a) and PERSEFONE (b) prototypes.

As already mentioned, the two prototypes differ in their slenderness ratio (i.e. total height to base width), equal to 1.67 for EUROPROTEAS and 4.5 for PERSEFONE.

The subsoil has been extensively characterized through *in-situ* surface and borehole geophysical tests (Figure 2a), as well as laboratory resonant column (Figure 2b) and shear strength tests. The results are comprehensively reported by Pitilakis et al. (2018).

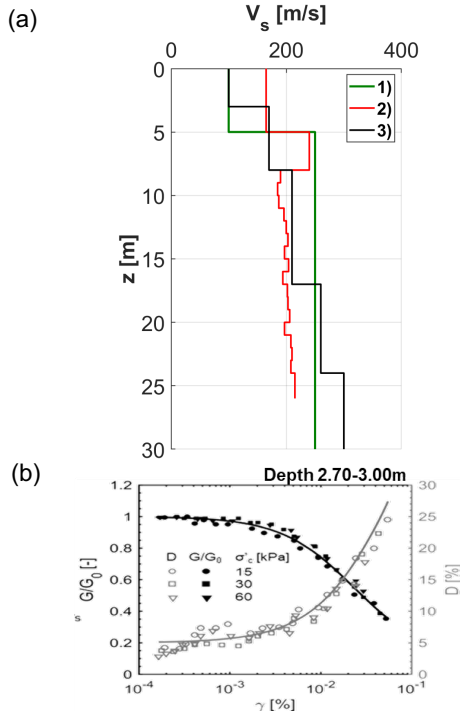


Figure 2. (a) Shear wave velocity profile adopted for the numerical analysis (1) vs. those measured in a Down-Hole test performed beneath the EUROPROTEAS foundation (2) and approximately 50 m away from it (3). (b) Variation of the normalized shear modulus and of the damping ratio with the shear strain measured in resonant column tests.

3 NUMERICAL MODEL

Two numerical models were developed with the FLAC 2D code to reproduce the SFSI mechanisms expected for both prototypes. The subsoil domain has a width of 60 m and a depth of 30 m, with absorbing boundaries at the base to simulate the half-space and free-field conditions on the lateral sides. A two-layers soil profile resting on an elastic half-space (bedrock) was adopted, with shear stiffness and damping properties based on the dynamic tests reported in Figure 2. The profile assumed for the shear wave velocity, V_s (line #1 in Figure 2a) was verified to yield a fundamental frequency consistent with that obtained from the H/V spectral ratio of the environmental noise, approximately 2.0 Hz.

The mechanical parameters reported in Table 1 also include the layer thickness, h , the soil unit weight, γ , and the Poisson's ratio, ν , derived from laboratory tests and soil lithology. The analyses were carried out in terms of total stress, by adopting the Tresca criterion for the upper fine soil layer, characterized by the undrained shear strength, c_u , obtained from triaxial undrained tests; the Mohr-Coulomb criterion was adopted for the underlying silty sandy formation, with shear strength defined by the friction angle, ϕ , resulting from direct shear tests. The water table was assumed to be at ground level.

The nonlinear hysteretic behavior of the upper soil layer was modeled using a sigmoid function, calibrated on the results of resonant column tests (Figure 2b), with an initial damping ratio set to 2%. The pre-failure behavior of the underlying

formation was assumed to be linear visco-elastic, as it was not expected to be affected by significant shear strain increments induced by the dynamic loading, as confirmed by the results of numerical analyses. The structure was modeled as viscoelastic as well, assuming a damping ratio of 5%. The mechanical properties of the concrete in the foundation slab and upper masses were set according to the design specifications for each prototype: C20/25 concrete for EUROPROTEAS and C25/30 concrete for PERSEFONE. For both models, the four columns were represented as a single equivalent wall, whose shear stiffness was calibrated to reproduce the fixed natural frequency of 9.13 Hz for EUROPROTEAS and 2.56 Hz for PERSEFONE, by modeling the soil as stiff as the bedrock.

Table 1. Mechanical properties of the subsoil layered model

Lithology	h	γ	V_s	ν	c_u	ϕ
[-]	[m]	[kN/m ³]	[m/s]	[-]	[kPa]	[°]
silty clay with sand	5.0	20	100	0.3	30	0
silty sand with clay	25.0	20	250	0.3	0	37
bedrock	-	20	1200	0.3	-	-

Dynamic identification of the numerical models was performed by applying an ambient white noise signal at the bedrock and subsequently analyzing the free-vibration phase. The fixed-base frequency, f_0 , was obtained from the peak of the Fourier spectrum of the horizontal displacement at the top of the structure, considering only the free-vibration phase. When the bedrock was replaced with the actual deformable subsoil, the dynamic identification led to values of the compliant-base frequencies, f^* , equal to 3.2 Hz for SISIFO and 2.2 Hz for PERSEFONE. The former results are in good agreement with the experimental free-vibration values measured in snap-back and random noise tests (Amendola et al., 2021).

4 FORCED VIBRATION TESTS

The simulations of forced-vibration tests were performed by applying a dynamic force, F_s , at the top of the structure equal to that exerted by a rotating-mass shaker. F_s is given by:

$$F_s = E(2\pi f)^2 \sin(2\pi f) \quad (1)$$

where E is the total eccentricity generated by the shaker (ranging from 1.85 kg·m in test Forced A to 11.31 kg·m in test Forced D) and f is the excitation frequency, varied within the typical range of interest for seismic engineering (1-10 Hz). In the numerical model, the harmonic excitation was applied to the top nodes of the upper slab. To account for the 2D geometry of the model and its discretization, the total force was divided by the out-of-plane dimension of the foundation, i.e. its length.

Given the kinematic variables and the masses involved, the moment-rotation (M:θ) hysteresis loops were derived from the dynamic equilibrium of the soil-foundation-structure system (Amendola et al., 2021). The reference maximum rotation, θ_{max} , was defined as the peak value in each cycle.

Figure 3 compares the experimental results in terms of θ_{max} (empty histograms) of the Forced A and Forced D tests from the SISIFO project on the EUROPROTEAS prototype with the numerical predictions (filled histograms). The comparison demonstrates a good ability of the model to reproduce the experimental response, notwithstanding the simplified assumptions on the 2D geometry and the mechanical properties of the structure and the soil. It can be observed that, for the Forced A test, the maximum rotation is predicted at f equal to 3

Hz, in agreement with the experimental results. In contrast, for the Forced D test, the maximum rotation is reached experimentally at f equal to 2.5 Hz, due to the degradation of stiffness and the consequent shift of the resonance frequency to lower values induced by soil non-linearity. This frequency shift is not captured by the numerical model, which still predicts resonance at $f=3$ Hz.

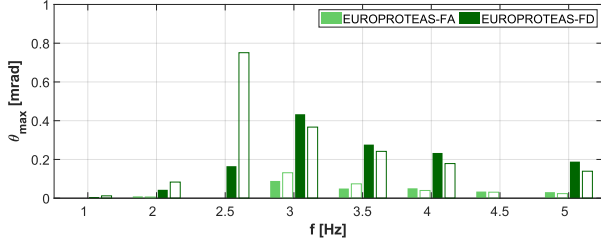


Figure 3. Comparison between the rotations derived from experimental data and numerical simulations of Forced tests A and D.

5 NUMERICAL RESULTS

The forced vibrations tests described in Section 4 were also numerically performed for PERSEFONE prototype. Key results are hereafter presented as function of the dimensionless frequency, a_0 , defined by Gazetas (1983) as follows:

$$a_0 = \frac{2\pi f B}{V_s} \quad (2)$$

where f is the loading frequency from Equation (1), B is the foundation width and V_s is the shear wave velocity of the first layer. In the following figures, the solid orange and red lines represent the results obtained for PERSEFONE prototype by simulating Forced A and Forced D tests, respectively. Similarly, the light green and dark green lines represent the results for the EUROPROTEAS prototype under the Forced A and Forced D tests, respectively. For both prototypes, the compliant-base fundamental frequency, f^* , is indicated by a dashed black line.

To quantify the degree of soil nonlinearity mobilized in each test, a reference peak shear strain amplitude, γ_{max} , was evaluated. Experimentally, γ_{max} was estimated as the ratio between the peak horizontal velocity recorded at the foundation, $\dot{u}_{f,max}$, and the average shear wave velocity of the soil volume interacting with the foundation, V_s . Numerical values, instead, were directly retrieved from the output of the analyses. According to Gazetas (1983), the characteristic length defining the influenced soil volume is the equivalent radius of the foundation, r_{eq} , i.e. the radius of a circle having the same area as the square footing. For EUROPROTEAS, r_{eq} is approximately 1.7 m, whereas it for PERSEFONE is approximately 0.6 m. Shear stress induced by horizontal loading becomes negligible ($< 10\%$) at a depth greater than $2r_{eq}$. Consequently, the numerical shear strain corresponds to the mean value mobilized down to 3.4 m for EUROPROTEAS and 1.5 m for PERSEFONE below the foundation.

Figure 4a shows γ_{max} versus a_0 . For EUROPROTEAS, numerical results are compared with the experimental data (dashed lines of the same color). In both prototypes, for both tests, a distinct peak in shear strain is observed at the resonance frequency, f^* . Beyond resonance, the shear strain decreases despite the increase in the force amplitude. Overall, PERSEFONE mobilizes higher shear strain than EUROPROTEAS, consistent with its design intent. Similarly, the maximum foundation rotation θ_{max} is greater for PERSEFONE, as shown in Figure 4b. In the Forced D simulation for PERSEFONE, at a_0 equal to 0.13, corresponding

to f equal to 5 Hz, both γ_{max} and θ_{max} exceed the values attained at resonance, whereas in the Forced A test this does not occur. The anomaly is evidence that the force amplitude achieved at the highest frequency applied in the Forced D test induced a significant accumulation of plastic deformation in the soil which increases the final total value of γ_{max} and θ_{max} with respect to that achieved at the resonance frequency.

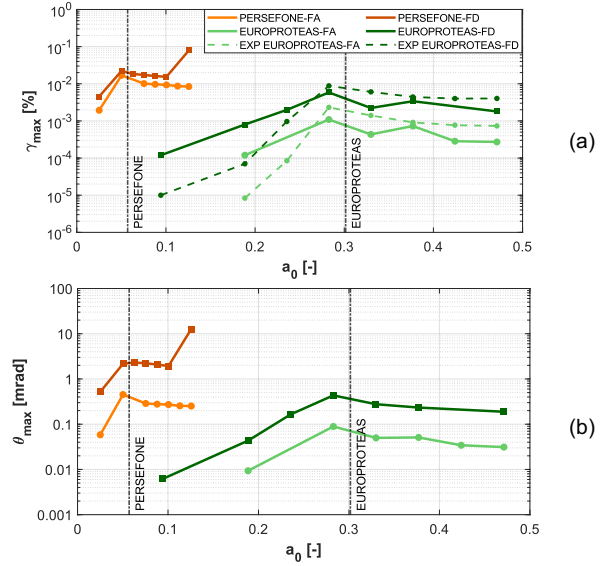


Figure 4. Peak reference shear strain (a) and rotation (b) mobilized in the forced tests on the two prototypes.

The total peak displacement at the top of the prototypes, u_{tot} , was also recorded as a measure of structural kinematic performance. The total drift was then computed as the ratio between the top displacement, u_{tot} , and the total prototype height H . The rigid displacement, u_r , due to foundation rocking, was calculated as the product of the foundation rotation, synchronous with the maximum top displacement, and H . Figure 5 shows the total top drifts (solid lines) and the rocking-induced displacements (dashed lines). The results clearly show that both u_{tot}/H and u_r/H for PERSEFONE are higher than those for EUROPROTEAS, confirming the achievement of the initial objectives to design a prototype more prone to rocking with respect to EUROPROTEAS.

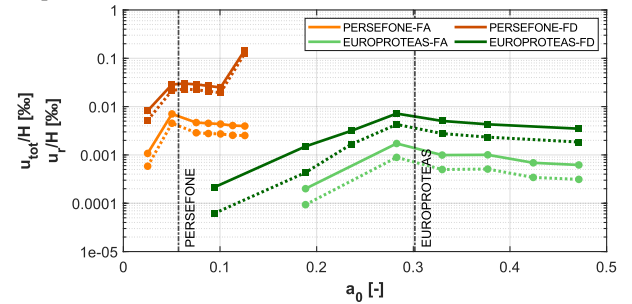


Figure 5. Drift and rotation mobilized during the tests by the two prototypes.

Following the Loop-Based (LB) procedure proposed by de Silva et al. (2023) for the interpretation of experimental data, the analysis of the $(M:\theta)$ loops numerically simulated permitted to derive the real and imaginary parts (i.e. dynamic stiffness and damping) of the foundation rocking impedance function (Gazetas, 1983; Kramer and Stewart, 2024), expressed as follows:

$$K_{yy}^* = k_{yy}(a_0)[1 + 2i\beta(a_0)] \quad (3)$$

The frequency-dependent foundation rotational stiffness, k_{yy} , and the damping ratio, β , resulting from the whole series of numerical tests simulations are reported in Figure 6. The rotational stiffness, k_{yy} , was normalized by the initial shear stiffness, G_0 , and the cube of the foundation width B , to allow a more consistent comparison between the results of the two prototypes.

Figure 6a shows that the normalized k_{yy} is higher for PERSEFONE with respect to EUROPROTEAS, which is due to the lower value of the denominator in the former case. The effect of soil nonlinearity is evident in both prototypes: for any given frequency, stiffness decreases and damping increases from Forced A to Forced D tests. The shift between Forced A and Forced D results is more pronounced for the PERSEFONE prototype, being specifically designed to mobilize stronger nonlinear effects.

Regardless of loading amplitude or prototype, the minimum stiffness and maximum damping occur when the excitation frequency approaches the natural frequency of the system. Near resonance, prototype motion is maximized, leading to shear modulus degradation in the soil and an increase in damping ratio, which translate into a reduction of rotational stiffness and an enhancement of dissipative capacity, respectively.

The figure also indicates that the well-established analytical solutions proposed by Gazetas (1991) for elastic half space agrees satisfactorily with the numerical predictions as long as the strain amplitude mobilized in the foundation soil does not trespass the linear threshold level (approximately 0.001%, see Figure 2b), namely for the case of Forced A test on EUROPROTEAS (see Figures 4a-6a). However, for both Forced D tests, the numerical predictions deviate from the Gazetas solution due to significant nonlinearity, with the PERSEFONE results showing the most pronounced deviation, confirming the mobilization of higher nonlinear effects compared to EUROPROTEAS ones.

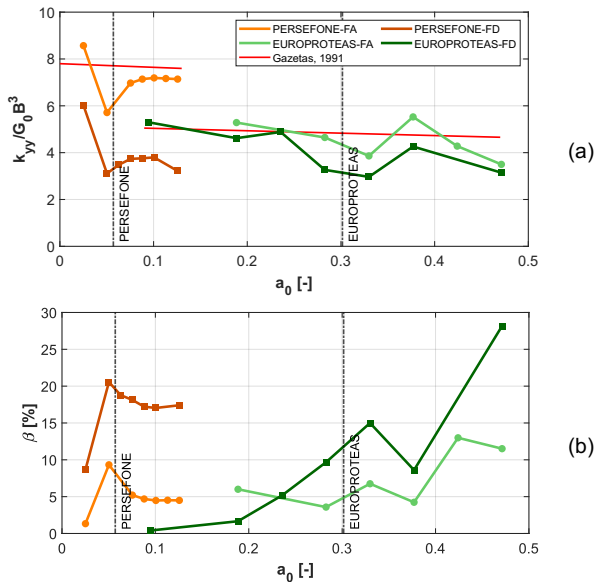


Figure 6. Rotational stiffness (a) and damping ratio (b) predicted for the tests on the two prototypes.

6 CONCLUSIONS

The predictive numerical analyses presented in this paper, combined with the experimental data previously gathered in the SISIFO project, addressed a more conscious planning of the testing program on the PERSEFONE prototype at EUROPROTEAS test site. Indeed, the numerical results

highlighted that soil nonlinearity and dissipative capacity are evident even at small soil strains and therefore should always be taken into account in the seismic design of structures, expected to be influenced by SFSI mechanisms.

In practice, the analytical formulations commonly adopted as simple predictive tools for foundation seismic impedance remain reliable only as long as the soil response stays within the linear range. Conversely, the growing availability of advanced computational tools capable of capturing nonlinear SFSI mechanisms suggests that the time is now ripe to design structures in which inertial loads are advantageously reduced by the dissipative capacity of the soil.

In the predictive analyses summarized herein, this aspect proved to be non-negligible, particularly at resonance, for both structural models. Nonlinear effects induced by foundation rocking motion were especially pronounced in the PERSEFONE prototype, designed on purpose to enhance this type of response. More comprehensive predictions and interpretations of the tests programmed, addressed to the assessment of the permanent effects associated with the mobilization of significant nonlinearities, are currently in progress.

7 ACKNOWLEDGEMENTS

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