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# Numerical and experimental evidence of the impact of soil nonlinearity on the response of an SSI system

## Preuve numérique et expérimentale de l'impact de la non-linéarité du sol sur la réponse d'un système ISS

M. Koronides*
*Cyprus university of technology, Limassol, Cyprus, Formerly: Imperial College London, London, UK*

S. Kontoe
*University of Patras, Patras, Greece. Imperial College London, London, UK*

L. Zdravković, D.M. Potts
*Imperial College London, London, UK*

A. Vratsikidis, D. Pitilakis, A. Anastasiadis
*Aristotle University of Thessaloniki, Thessaloniki, Greece*

**marios.koronides@cut.ac.cy*

**ABSTRACT**: The influence of dynamic Soil-Structure-Interaction (SSI) phenomena on the response of structures under dynamic loading is widely recognised. These phenomena are more impactful on structures that are founded on soft soil sites, where soil nonlinearity can prevail. The present study examines this aspect by the means of three-dimensional nonlinear finite element analyses of real-scale free-vibration experiments on a steel frame structure. The structure has outer dimensions of 3×3×5m and is rested on a shallow foundation over a soft soil deposit. The numerical analyses focus on modelling experiments of different magnitude excitation forces, to allow the investigation of the SSI system for a wide strain range. Cyclic nonlinear models, which are typically used in numerical analysis to predict the hysteretic behaviour of soils, can simulate well either the stiffness degradation (G-γ) or the damping evolution (D-γ) with cyclic shear strain. Experimental strong motion data are compared against numerical results of analyses that adopt different G-γ-D curves for the shallow foundation soil. This investigation reveals that the choice of G-γ-D curves affects significantly the SSI free-vibration response, highlighting the importance of modelling the nonlinear behaviour of shallow soil appropriately. It also suggests that prioritising the calibration of the D-γ curve over G-γ is more applicable for most of the modelled experiments.

**RÉSUMÉ**: L'influence des phénomènes dynamiques d'interaction sol-structure (ISS) sur la réponse des structures soumises à des charges dynamiques est largement reconnue. Ces phénomènes ont un impact plus important sur les structures fondées sur des sols meubles, où la non-linéarité du sol peut prévaloir. La présente étude examine cet aspect au moyen d'analyses par éléments finis non linéaires tridimensionnels d'expériences de vibration libre à l'échelle réelle sur une structure en colonnes d'acier. La structure a des dimensions extérieures de 3×3×5 m et repose sur une fondation peu profonde au-dessus d'un dépôt de sol meuble. Les analyses numériques se concentrent sur les expériences de modélisation des forces d'excitation de différentes magnitudes, afin de permettre l'étude du système ISS pour une large gamme de déformations. Les modèles non linéaires cycliques, qui sont généralement utilisés dans l'analyse numérique pour prédire le comportement hystérétique des sols, peuvent bien simuler la dégradation de la rigidité (G-γ) ou l'évolution de l'amortissement (D-γ) avec la déformation cyclique de cisaillement. Les données expérimentales sur les mouvements forts sont comparées aux résultats numériques d'analyses qui adoptent différentes courbes G-γ-D pour le sol de fondation peu profond. Cette étude révèle que le choix des courbes G-γ-D affecte de manière significative la réponse en vibration libre du ISS, soulignant l'importance de modéliser de manière appropriée le comportement non linéaire du sol peu profond. Elle suggère également que la priorité donnée à l'étalonnage de la courbe D-γ par rapport à la courbe G-γ est plus applicable pour la plupart des expériences modélisées.

**Keywords:** Dynamic soil-structure interaction; finite element modelling; real-scale experiments; soil nonlinearity; constitutive model calibration.

## 1 INTRODUCTION

The significant role of soil nonlinearity on the dynamic response of soil-structure-interaction (SSI) systems with shallow foundations has been demonstrated in various studies in the past (e.g. Anastasopoulos et al., 2011; 2014; Gazetas et al., 2013). Most of these studies focused on large strain problems, where soil plasticity is significantly engaged, however, soil nonlinearity associated with the small strain response has attracted less attention.

In problems dominated by the nonlinearity of soil behaviour at small strains, it is important to consider an appropriate set of stiffness degradation and damping ratio (G-γ-D) curves, due to the very limited linear elastic strains in such soils (e.g. Darendeli, 2001; Vucetic, 1993). Cyclic nonlinear models that are typically used in such analyses can simulate well either the stiffness or the damping curve (Taborda and Zdravkovic, 2012). It has been shown that the accurate prediction of the damping curve should be prioritised in free-field site response problems (e.g. Han et al., 2016; Papaspiliou et al., 2012), however there are no similar studies for SSI problems.

The present study discusses 3D nonlinear finite element (FE) simulations of real-scale free vibration tests on the prototype structure of EUROPROTEAS. Particular emphasis is placed on modelling the soil behaviour, investigating the impact of adopting different calibrations for the G-γ-D curves.

## 2 EXPERIMENTAL CAMPAIGN

### 2.1 Structure and experimental set-up

The examined structure is the prototype EUROPROTEAS structure that is 5m tall and 3 x 3m in plan. This frame structure comprises four steel columns (SHS 150 x 150 x 10mm), adjustable steel cross bracing system (L-shape 100 x 100 x 10mm), a reinforced concrete mat foundation (9Mg) and adjustable superstructural mass. Although different structural configurations were examined, the present study refers to one configuration only: a structure braced at all four sides and a superstructural mass of 18Mg (Figure 1).

The wider experimental campaign consisted of free and forced vibration tests (Koronides et al., 2023a), with the present study focusing on the former tests. The free vibration tests involved the application of a pull-out force at 9˚ angle to the horizontal plane on the top slab (Figure 1), via a wire rope tensioned by a pulling hoist. The force was measured in-situ by a load cell and upon reaching the desired magnitude, the wire was instantly cut, enabling the structure to oscillate until it came to rest. In both test types, the force was applied along the NS plane of symmetry of the structure, with reference to the coordinate system shown in Figure 1. The response of the SSI system was monitored by a comprehensive instrumentation that was placed on both the structure and the soil.

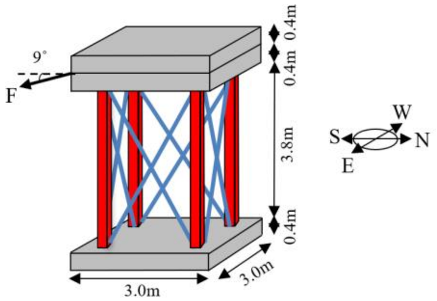


*Figure 1. EUROPROTEAS' structural configuration.*

### 2.2 Soil conditions

EUROPROTEAS is located at the TST site, the centre of Euroseistest site, within a valley in Northern Greece. The geotechnical and geological properties of the site are well documented by previous surveys (e.g. D. Pitilakis et al., 2018; K. Pitilakis et al., 1999), with Figure 2(a) and (b) presenting the resulting soil stratigraphy and shear velocity profiles, respectively. The shallow soil layers mainly consist of silty sand and low plasticity clay of very low stiffness, a condition that is anticipated to amplify SSI effects. The geotechnical site characterisation encompassed also resonant column and cyclic triaxial tests for soil samples retrieved from various depths, some of which are indicated in Figure 2(a). These tests produced stiffness degradation and damping ratio curves (G-γ-D) for the corresponding soil layers. The process of projecting the G-γ-D curves from lab conditions to site conditions is described in Koronides et al. (2023a).

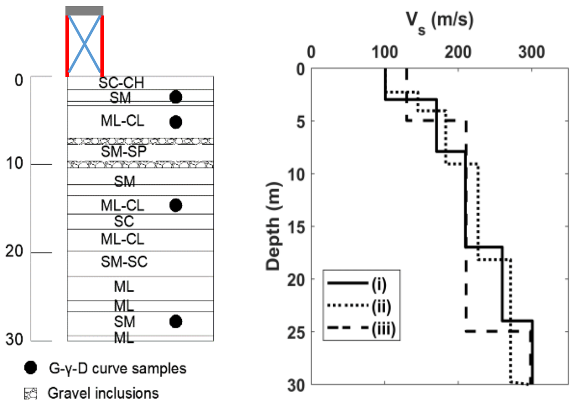


*Figure 2. (a) Soil stratigraphy, (b) shear wave profiles proposed by (i) K. Pitilakis et al. (1999), (ii) Raptakis et al. (2000) and (iii) Raptakis and Makra (2015).*

## 3 NUMERICAL MODEL

Free vibration simulations were carried out using the Imperial College Finite Element Program (ICFEP) (Potts & Zdravković, 1999). Each simulation consists of three phases: a static phase where the deadloads are applied, a static phase where the pull-out force is applied, and a dynamic phase where the free oscillation of the structure is computed.

## 3.1 Problem geometry and boundary conditions

The numerical model simulates only the half of the domain, exploiting the symmetry of the problem. Figure 3 presents the 3D mesh, dimensions and some of the boundary conditions.

The simulated soil domain has dimensions 15 x 7.5 x 6m, which were found to produce neglible boundary effects on the structural response and tolerable effects on the soil response. The 6m deep soil domain is divided into two soil layers (Figure 3a), as identified from the experimental studies, the results of which are shown in Figure 2(a). The domain consists of 20-noded brick elements, the size of which was based on the guidelines of Kuhlemeyer and Lysmer (1973). The corner nodes of the bottom boundary of the soil domain are fully fixed, while all nodes at the plane of symmetry (including the soil, base and top slab and braces) are fixed in the out-of plane (y) direction only. Dashpots and springs (Kontoe, 2006) are applied along the normal and tangential directions on the remaining soil boundary nodes, except those at the ground surface.

The numerical model explicitly simulates the dimensions of the actual structure (Figure 3(b)). The foundation and top slabs are modelled with 20-noded brick elements, while the steel columns and braces with 3-noded beam elements. To establish a moment connection between the columns and slabs, the beam elements of the columns extend into the brick elements of the slabs. The stiffness of the extended beam elements and the damping (ξ) of concrete and steel were calibrated in Koronides et al. (2023a). The remaining structural properties are chosen in accordance with modern regulations (i.e. British Standards Institution, 2004, 2005). All structural elements are assumed to be linear elastic, with their properties, including Young's modulus (E), Poisson's ratio (ν), density (ρ), cross sectional area (A), moment of inertia (I), torsional constant (J) and target Rayleigh damping (ξ), given in Table 1.

The excitation pull-out force (F) is applied at a node of top slab that lies in the plane of symmetry, as shown in Figure 3(a). Two free vibration experiments of 2.9kN and 15.7kN pull-out force are simulated. However, considering that only half of the problem geometry is modelled, half of the above magnitudes was applied in each analysis.

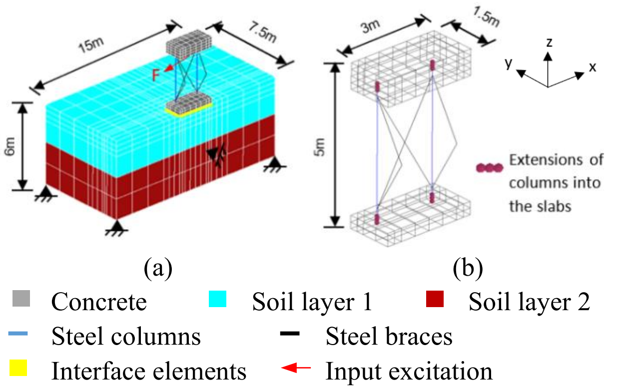


*Figure 3. FE mesh, dimensions and some boundary conditions of the simulated experiments: (a) SSI system, (b) Structure.*

## 3.2 Soil-foundation interface model

As discussed by Koronides (2023), field observations as well as strong motion data interpretation indicated permanent gap areas at the interface between the foundation and the soil. These played a crucial role in the response of the SSI system. The gap distribution and the interface model were calibrated by Koronides et al. (2022a, 2023a) against forced vibration measurements. The resulting distribution of permanent gaps is illustrated in Figure 4. Koronides et al. (2023b) demonstrated that during the various experiments carried out in the experimental campaign, temporary detachment and re-attachment of some foundation areas took place. However, they illustrated that for the present experiments, the above foundation behaviour could be reasonably neglected to reduce the computational cost. Therefore, the present analyses assume permanent interface gaps at the positions shown in Figure 4. only. For the interface model, elastic zero thickness interface elements (Day and Potts, 1994) are employed. These are characterised by shear and normal elastic stiffnesses, $K_s$ and $K_n$, which control the relative elastic displacements between the foundation and the soil. A stiffness value of $K_s$=$K_n$=1E8kN/m$^3$ was adopted for the contact area, whereas $K_s$=$K_n$=1E2kN/m$^3$ was used for the non-contact area. The reduced interface stiffness of the latter area was shown equivalent to simulating permanent gaps, but at lower computational cost (Koronides et al., 2022b; 2023a; 2023b).

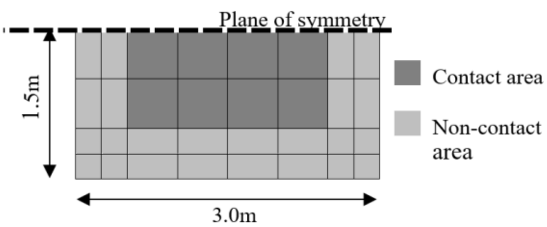


*Figure 4. Permanent contact conditions at the soil-foundation interface.*

*Table 1. Elastic properties of the structural elements.*

| | E (GPa) | ν | ρ (Mg/m$^3$) | A (cm$^2$) | I (cm$^4$) | J ($cm^4$) | ξ (%) |
|---|---|---|---|---|---|---|---|
| Concrete | 31 | 0.2 | 2.5 | - | - | - | 1 |
| Braces | 200 | 0.25 | 7.85 | 19.2 | 177 | 6.97 | 2 |
| Columns | 200 | 0.25 | 7.85 | 54.9 | 1770 | 2830 | 2 |
| Beam elements extended into the foundation | 200E4 | 0.25 | 0.05 | 54.9 | 1770 | 2830 | - |
| Beam elements extended into the top slab | 3 | 0.25 | 0.05 | 54.9 | 1770 | 2830 | - |

## 3.3 Nonlinear soil constitutive model

Koronides (2023) showed that for the examined SSI problem, soil nonlinearity extends within a very shallow soil zone. Herein, the first 0.8m of the soil domain is assumed to behave as nonlinear elastic material, while the remaining soil domain is prescribed as linear elastic. Table 2 presents the soil properties of the two layers at very small strains, that were inferred from previous site characterisation studies (Figure 2(b)). A target Rayleigh damping (ξ) of 5% was selected for the linear soil.

*Table 2. Small strain soil properties.*

| | E (MPa) | ν | ρ (Mg/m$^3$) |
|---|---|---|---|
| Layer 1 | 100 | 0.25 | 2.0 |
| Layer 2 | 186 | 0.20 | 2.1 |

For the nonlinear soil, the Imperial College Generalised Small Strain Stiffness (IC.G3S) model (Taborda et al., 2016) is used. The model prescribes the G-γ-D curves based on a predefined set of parameters. Figure 5 presents the three different G-γ-D curves, whose effect on the SSI response is investigated herein. These are compared with the reference curves, which were interpreted from resonant column and cyclic triaxial tests.

This type of cyclic models can describe well either the damping or shear stiffness curve, which is deemed the most important consideration on the calibration of the model's parameters. As Figure 5 shows and as highlighted by Taborda and Zdravkovic (2012), if the calibration prioritises the shear stiffness degradation curve (CalibrG), the soil damping is highly underestimated and highly overestimated at the very low and medium strain range, respectively. When, though, the calibration prioritises the damping ratio curve (CalibrD), the model underestimates the soil shear stiffness over the very small to small strain range. To further examine the impact of damping on the SSI response, an additional set of G-γ-D curves that predict a stiffness curve similar to CalibrD, but overestimates damping over a wide range of strains (CalibrDO) is also examined.

The adopted nonlinear constitutive model assumes soil shear stiffness as a function of mean effective stress. This is of paramount importance in SSI problems, where the presence of a structure contributes to a stiffer foundation soil in relation to the free-field conditions, affecting the response of the system.

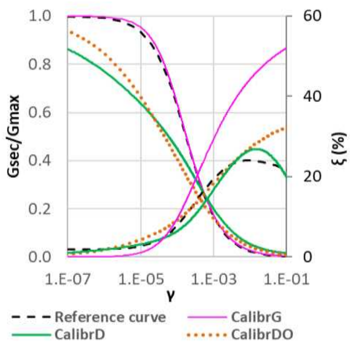


*Figure 5. Examined calibrations of stiffness degradation and damping ratio curves.*

# 4 RESULTS AND DISCUSSION

Figure 6 presents acceleration time histories and the corresponding Fourier spectra recorded at the top slab during the free vibration experiments of pull-out forces 2.9 and 15.7kN, as well as the linear elastic prediction. The accelerations were normalised by the magnitude of the pull-out force to allow comparison between different experiments, with any differences being attributed to nonlinear phenomena. The figure demonstrates that when larger pull-out force was applied, the response was more flexible and lower in amplitude. Possible nonlinearities are soil nonlinearity, interface nonlinearity and structural elements' nonlinearity. The latter is assumed negligible due to the weak bending of the structural elements (Koronides, 2023). Also, as shown by Koronides et al. (2022a; 2023a; 2023b), if interface nonlinearity was predominant, it would have led to a smaller interface contact area and, consequently, weaker radiation damping, resulting in motion amplification. This expectation contradicts the observed de-amplification, suggesting that the observed nonlinear phenomena can be predominantly assigned to soil nonlinearity. Figure 6 also demonstrates that a linear elastic analysis overpredicts both the natural frequency and the magnitude of the

SSI response, highlighting the necessity of incorporating soil nonlinearity.

*Figure 7*Figure 7 presents acceleration time histories and the corresponding Fourier spectra inferred from nonlinear analyses that assume different calibrations for the G-γ-D curves. CalibrG analysis predicts a larger natural frequency for both the small and larger strain response (i.e. F=2.9kN and F=15.7kN, respectively) and, most importantly, it strongly overdamps the motion of the system subjected to a larger force (i.e. F=15.7kN, shown in Figure 7b). Conversely, when the calibration prioritises the damping curve, e.g. CalibrD, the analysis accurately predicts the magnitude and frequency of both experimental responses. This observation demonstrates that, in SSI problems, the calibration of the G-γ-D curves should prioritise capturing the experimental damping curve well, accepting the limited consequences of underestimating the shear stiffness curve.The importance of damping in the examined SSI problem is further corroborated by the comparison of CalibrD and CalibrDO results. The two analyses predict similar response frequency, as a consequence of their similar soil stiffness simulation. However, CalibrDO, which overestimates damping, results in response overdamping for both small strain (i.e. F=2.9kN) and larger strain response (i.e. F=15.7kN). Hence, it can be concluded that a seemingly small damping overestimation can lead to significant errors, emphasizing the critical need for accurately simulating soil damping.

## 5 CONCLUSIONS

Three-dimensional FE simulations of real scale free vibration experiments have been presented. Although the field experiments induced small strains in the soil, soil nonlinearity was important, as indicated by both numerical and experimental data evidence.

Numerical results from analyses that adopt different calibrations for the G-γ-D curves were compared, demonstrating the necessity to simulate the damping behaviour of the soil appropriately. In fact, it is suggested that the calibration of cyclic nonlinear models should prioritise capturing the damping curve, even at the cost of underestimating the stiffness curve.

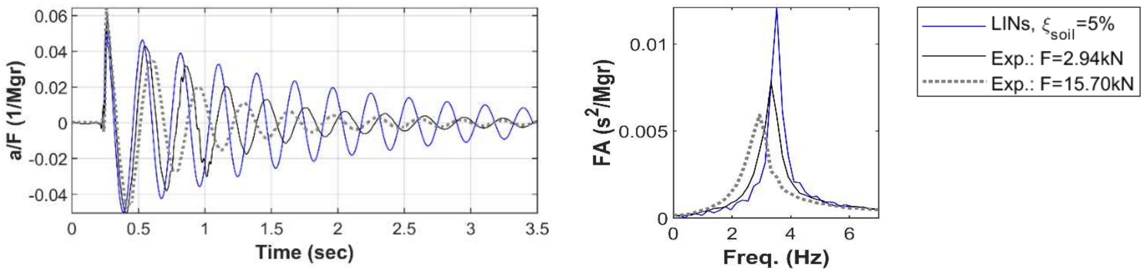


*Figure 6. Normalised top slab acceleration time histories and the corresponding Fourier spectra inferred from linear analysis and experimental data.*

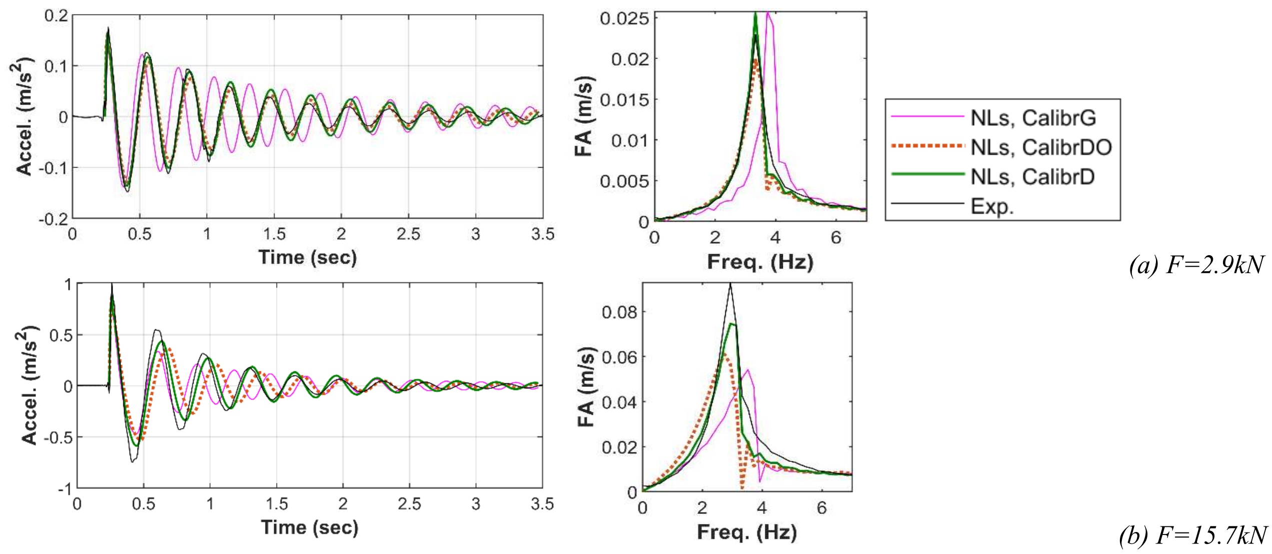


*Figure 7. Top slab acceleration time histories and the corresponding Fourier spectra inferred from experimental data and nonlinear analyses with different calibrations.*

The latter conclusion aligns with similar conclusions drawn from free-field site response analyses (e.g. Han et al., 2016; Papaspiliou et al., 2012).

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