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Uplift and floor response spectra of a nuclear building using a linear and nonlinear transient soil structure interaction analysis: comparison and modeling limits

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ABSTRACT

In the realm of earthquake-resistant design and assessment of civil engineering structures, it has been widely recognized that in case of strong shaking and significant foundation uplifting, the designer can no longer accept the assumption of linear behaviour and is obliged to adopt a nonlinear analysis approach. This observation motivates the question as to what extent linearity assumption can be retained in a rigorous seismic design methodology for structures allowed to uplifting. The present work aims at establishing a quantitative comparison between a linear (denoted LT) and nonlinear transient analysis (denoted NLT). Both methods belong to direct method for soil-structure interaction (no substructuring) with explicit modelling of the superstructure and the surrounding soil. These two methods are used to study a simplified soil-structure configuration pertaining to a typical industrial building with a thick basemat, which is founded on a stratified soil profile. We consider that the superstructure is designed in the linear range and that the unique source of nonlinearity is located at the soil-foundation interface, along which, we account for foundation uplifting and sliding. The validity range of each method is examined with respect to some quantities of interest characterizing the overall dynamic response, such as the basemat uplift ratio and the floor spectra in representative points within the structure (*i.e.* the external points of the roof which can be used to describe the overall behaviour of the structure).

INTRODUCTION

A key challenge in both seismic design (RCC-CW 2025) and the seismic reassessment of civil engineering structures, particularly for industrial/nuclear facilities and energy infrastructure in regions with high seismic activity or stringent seismic requirements, is the development of simple yet accurate methods to evaluate their response. These methods must also assess the capacity of structures to meet the performance objectives outlined by design standards.

To address this challenge, increasingly sophisticated numerical simulations are employed to capture the studied physical phenomena. Such approaches aim to optimize seismic designs by balancing both safety and cost-effectiveness. Recent studies have underscored the significant influence of nonlinear soil-structure interaction on the seismic response of structures. As a result, it is essential that numerical methods used in seismic design are capable of accurately accounting for effects such as soil plasticity, foundation uplift, and sliding at foundation interfaces.

Several modeling approaches for Soil-Structure Interaction (SSI) have been proposed in the literature (Chatzigogos et al. 2014, Makrypidi et al – 2017, 2019). These methods can be categorized into three main groups based on the way they represent the bounded (superstructure with foundation) and unbounded (soil) domains: a) **direct methods**, b) **hybrid methods**, and c) **substructure methods**.

In **direct methods**, conventional finite element modeling is applied to both the bounded and unbounded domains, which may exhibit either linear or nonlinear behavior. The dynamic response is obtained by solving the dynamic equilibrium equations in the time domain. This method is compatible with general-purpose finite element codes, provided that suitable techniques for incorporating seismic excitation and boundary conditions are implemented.

Hybrid methods involve decomposing the soil-structure system into two subdomains, which are modeled independently. The core concept of hybrid methods is that all nonlinear phenomena are confined to the bounded domain, while the unbounded domain remains linear. The unbounded domain is typically modeled using the Boundary Element Method (BEM), whereas the bounded nonlinear domain is discretized with the Finite Element Method (FEM), with the solution being obtained in the time domain.

Substructure methods are employed to solve the interaction problem for systems with linear or moderately nonlinear behavior. In this approach, the bounded domain (structure) is modeled using finite elements, and the unbounded domain is represented with boundary elements. The artificial boundary conditions applied to the superstructure domain are typically expressed through a force-displacement relationship at the soil-structure interface, known as the impedance operator. These relationships can be formulated in the frequency domain, though hybrid approaches using both time and frequency domain methods can also be utilized.

This study focuses on evaluating direct soil-structure interaction (SSI) methods in both linear and nonlinear contexts to provide a quantitative comparison. These methods are applied to a simplified soil-structure model representing a typical industrial building with a thick basemat, founded on a layered soil profile. For clarity, it is assumed that the superstructure behaves within the linear range, with the only source of nonlinearity in the system arising at the soil-foundation interface, where foundation uplift and sliding are considered. The validity of each method is assessed by examining key response characteristics, such as the basemat uplift ratio and floor spectra at selected locations within the structure. This study specifically seeks to highlight the impact of foundation uplift on the floor response spectra of a building, thereby initiating a discussion on the potential definition of an “uplift spectrum”.

BASIC ASSUMPTIONS FOR THE STUDY

This study considers a simplified soil-structure model representing a typical industrial building. The structure is a symmetric reinforced concrete building, featuring continuous external vertical walls, a roof, and an interior steel substructure connected to the basemat. It is a square-based reinforced concrete structure with approximate sides of 14m. The building extends 18m above the platform and is composed of four 80cm-thick walls and a roof slab of the same thickness. The interior structure, with an inner diameter of 9.50m and a height of 11m, is anchored to a circular thickening of the slab, which has a diameter of 10m and a thickness of 0.20m, secured with 59 anchor rods. The mechanical characteristics of the structure are summarized in the following table.

Table 1: Mechanical characteristics of the studied structure.

Component	Symbol	Value
Static Young’s modulus-concrete structure	E_{cd}	11.2GPa
Dynamic Young’s modulus-external walls and mat foundation	$E_{cm,u}$	33.7GPa
Dynamic Young’s modulus-roof	$E_{cm,c}$	16.85GPa
Young’s modulus-steel structure	E_s	203GPa
Static Poisson’s ratio-concrete structure	ν_{cd}	0.20
Dynamic Poisson’s ratio-concrete structure	ν_{cm}	0
Poisson’s ratio-steel structure	ν_s	0.3
Concrete mass density	ρ_c	2500kg/m^3
Steel mass density	ρ_s	7800kg/m^3

Damping critical ratio – concrete structure	β_c	7%
Damping critical ratio – steel structure	β_s	4%

The building is founded via a thick, rigid basemat, which is allowed to uplift, and is placed on a stratified soil profile, modeled as a linear viscoelastic half-space. Soil is characterized by viscoelastic effective parameters calculated so as to be compatible with the anticipated soil distortions due to seismic loading. The determination of effective soil properties is achieved by using a 1D model of the soil stratigraphy whereby a deconvolution of the seismic motion is performed from the ground surface down to the deepest layer. A depth of approximately 30m below the basemat surface is considered to be sufficient for the evolution of phenomena related to soil-structure interaction. The 1D model used in this type of calculation corresponds to the idealized horizontal stratigraphy established at the location of the building and is completed with stiffness reduction curves G/G_{max} (γ) for an equivalent linear deconvolution analysis.

The mechanical characteristics of the geological layers used to model the surrounding soil are the Young's modulus E, the Poisson ratio v, the mass density ρ and the material damping β obtained for the soil by the site response analysis. Damping in each soil layer is obtained using the Rayleigh method by adjusting the Rayleigh parameters for mass and stiffness based on the hysteretic damping ratios of each soil layer, in order to achieve a soil column response comparable to the preliminary frequency-based calculation. Table 2 summarizes the mechanical characteristics of the geological layers considered for the base configuration.

Table 2: Soil parameters for the studied configuration.

Soil	**Thickness [m]**	$\boldsymbol{E}$ **[MPa]**	$\boldsymbol{v}[-]$	$\boldsymbol{\rho}$ **[kg/m³]**	$\boldsymbol{\beta}$ **[%]**
Backfill	4	660	0.48	2050	3.0
Bedrock	2	4200	0.48	2300	1.0
	2	7200	0.48	2500	1.0
	4	9500	0.43	2500	1.0
	14	17400	0.43	2450	1.0
	2	17900	0.40	2500	0.5

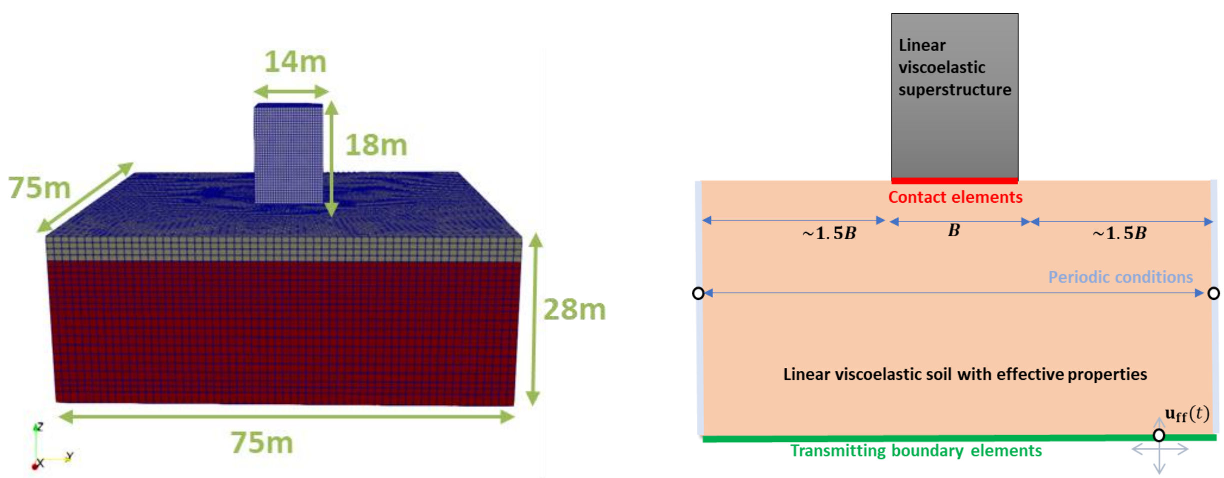

Figure 1. Soil-structure configuration examined in the study.

At the base of the finite element model, we introduce absorbing boundary elements through which the incident seismic motion is applied, calculated at the surface of the substratum using the linear deconvolution calculation performed beforehand. These elements "inherit" the parameters of the elastic materials to which they are attached. The absorbing boundary elements are paraxial zero-order elements and correspond to viscous dampers attached to the base of the soil container for both horizontal and vertical loading.

A periodic motion condition is applied to the lateral boundaries of the model. This method assumes that the motion is identical along the entire perimeter of the model. This condition is implemented with model dimensions large enough to accurately represent realistic behavior at the location of the considered structure. Therefore, in the horizontal direction, the width of the modeled soil domain between the walls of the external structure and the edge of the model is taken to be at least $\sim 1.5B$, where B is the width of the structure. This modeling approach ensures a sufficient distance from the soil-structure interaction zone, thus restoring free-field conditions along the lateral walls of the model.

The mesh size of the finite element model is chosen to ensure at least 8 elements per wavelength for wave propagation. In this regard, a maximum mesh size of 1m can be used within the backfill, allowing for adequate frequency transmission up to 30Hz. The mesh size at bedrock is 1.5m. This level of discretization enables accurate modeling of seismic wave propagation within the modeled soil domain. The dimensions of the surrounding soil domain considered in the calculations are significant (75m × 75m × 28m).

The difference between the model developed for the two methods (linear and nonlinear) lies in the modeling of the foundation-soil interface at the mat foundation. In order to facilitate the inclusion of contact elements for the nonlinear method, the overall modeling is performed by inserting 0.001m thick degenerate hexahedral 8-noded elements between the foundation and the soil. For linear analysis, these elements inherit the viscoelastic properties of the basemat. For the nonlinear methods, these elements are modelled as contact elements following the JOINT_MECA_FROT model from the Code_ASTER software, which allows for the consideration of sliding and foundation uplift.

The seismic loading is represented in the horizontal direction by a design response spectrum with a PGA of around 0.4g in order to introduce a significant uplift, while the vertical component is defined as 2/3 of the horizontal one. As for the transient analyses, a set of five spectrum-compatible acceleration time histories has been defined in order to establish the seismic input motion samples to be used in transient analyses. Each accelerogram contains 8192 time steps with dt = 0.003s. The signals are baseline corrected so that the velocity and displacement time histories are free from drift and exhibit peak values that are compatible with the anticipated intensity measures (IM) for the selected seismic scenario. These 5 signals are used to form 5 three-dimensional excitation scenarios.

METHODOLOGY FOR DIRECT SSI ANALYSIS

The seismic analysis is conducted using the "floating model" technique, which allows the introduction of the horizontal and vertical components of the seismic motion through transmission elements placed at the lower horizontal boundary of the three-dimensional model.

The system of differential equations is solved using a step-by-step integration procedure. The determination of seismic forces for the verification of structural integrity is carried out through transient calculations, which are divided into two distinct phases:

- Initialization under gravitational acceleration and adjustment of the material properties for the structure for both long-term and short-term phases. is based on long-term properties, followed by a transition to the short-term properties of the structure in order to apply the earthquake,
- Application of the horizontal and vertical seismic motions simultaneously at the bedrock level; this is done through a special boundary element (0-order paraxial element) that represents the stress conditions at the rock-soil interface.

For the static initialization phase (first step), a pseudo-time scale is defined. The surrounding soil is activated with static boundary conditions. The structure and foundation-soil interface elements are also activated. In this configuration, the gravitational loads of the structure (permanent static loads) are applied.

The second step involves setting the displacements to zero while preserving the stress field in the soil. The floating model is activated (meaning that the boundary conditions at the vertical extremities of the model are deactivated and replaced by the corresponding reactions). Absorbing boundaries are also activated, along with a set of auxiliary springs used to balance the floating model. The auxiliary springs are placed at the base of the model and have negligible stiffness. This step aims to balance the floating model under the gravitational loads and the reactions applied to the model's boundary. The floating model technique is necessary so that the absorbing boundaries placed along the model's perimeter can function properly (if the degrees of freedom are fixed, there will be perfect reflection of the incident waves rather than "absorption" by the absorbing boundaries).

After balancing the floating model, the displacements are reset to zero, the stress field in the soil is preserved, and the transient seismic loading is applied. The solution is performed in the time domain. This step is resolved by direct integration of the equilibrium equation using an unconditionally stable Newmark scheme (with the default Newmark parameters set to β = 0.25 and γ = 0.5). The damping of the superstructure and the soil is accounted for by using a damping matrix defined by the Rayleigh method, where the damping matrix $[C]$ is defined as a linear combination of the mass matrix $[M]$ and the stiffness matrix $[K]$:

$$[C] = \alpha[K] + \beta[M]$$

The coefficients α and β are adjusted in order to achieve the target damping for each part of the structure over a range of frequencies of interest.

RESULTS

Emphasis has been given on two specific aspects of dynamic response, namely: a) maximum uplift ratio developed during loading, b) floor response spectra in positions of interest within the building. The selected positions for calculation of these quantities are the external points of the roof that are connected with the walls. Floor response spectra are calculated at these positions either in the two horizontal and the vertical direction.

Maximum Uplift ratio

The foundation uplift is determined by calculating the normal force developed along the soil-foundation interface taking into account the regular mesh of the foundation. For linear analysis, the uplift ratio is estimated using the negative vertical reactions (traction), whereas for nonlinear analysis, it is calculated by considering zero vertical reactions. Thus, the uplift ratio at each time step i is calculated as the following ratio:

$$\kappa_i = \frac{n_{\text{neg},i}}{n_{\text{tot}}}$$

where $n_{\text{neg},i}$ is the number of negative normal forces (nodal reaction forces) at a time step i and n_{tot} is the total number of forces obtained at the horizontal interface.

Figure 2 presents, for the Linear Transient analysis (denoted LT), the evolution of the uplift ratio developed at the foundation over time for the 5 scenarios. The implemented methodology for calculating the uplift ratio involves obtaining the internal forces developed at the contact element level and identifying the normal forces that are either zero or negative.

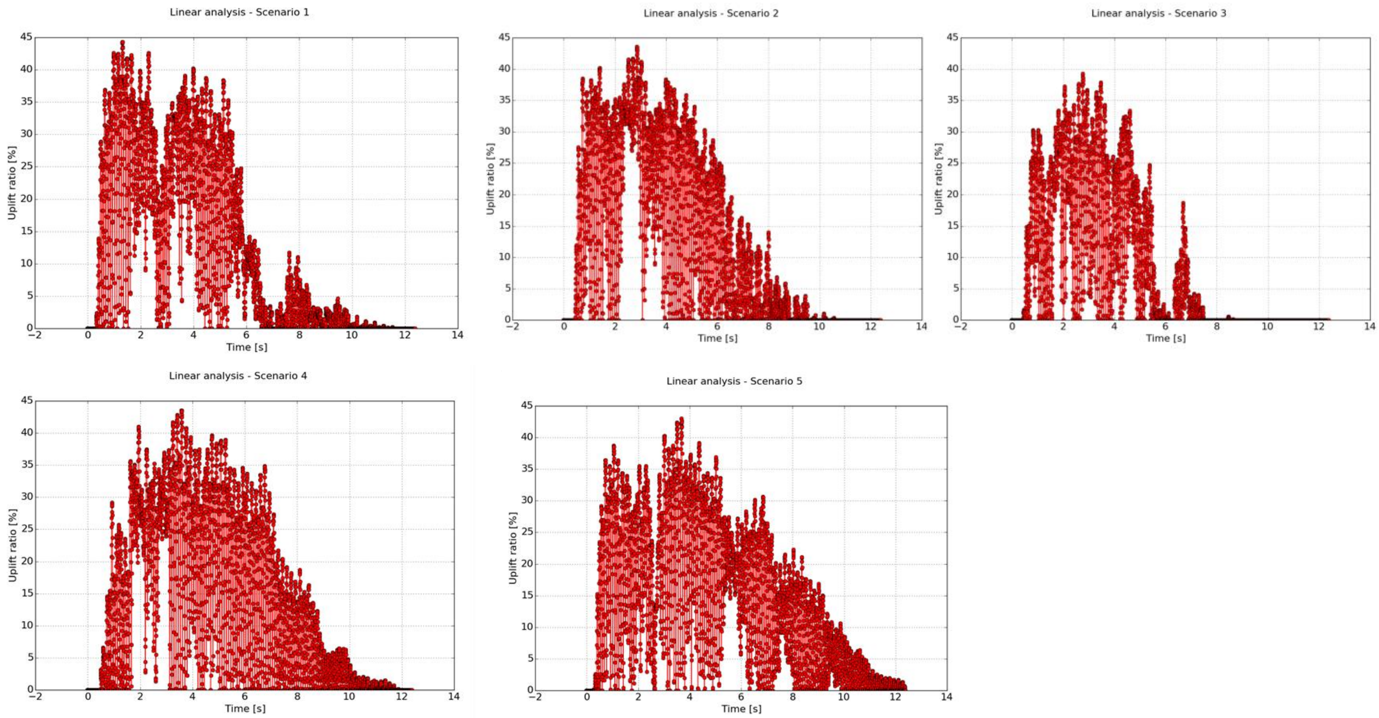

Figure 2. Evolution of uplift ratio over time – LT analysis.

Figure 3 presents the obtained cartography of the anticipated uplifted zone for the most conservative scenario (Scenario 1, Figure 2a) and the moment of the maximal uplift, which corresponds at the time of 1.31s.

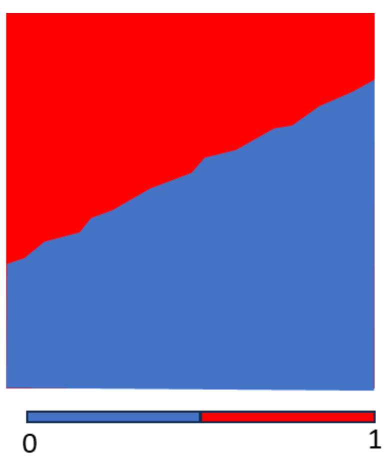

Figure 3. Diagram of maximal uplift ratio 0 no uplift and 1 uplift occurs) – Scenario 1 – LT analysis.

Figure 4 depicts the time evolution of the uplift ratio at the foundation for the 5 scenarios in the Nonlinear Transient analysis (denoted NLT). The methodology employed to calculate the uplift ratio remains unchanged.

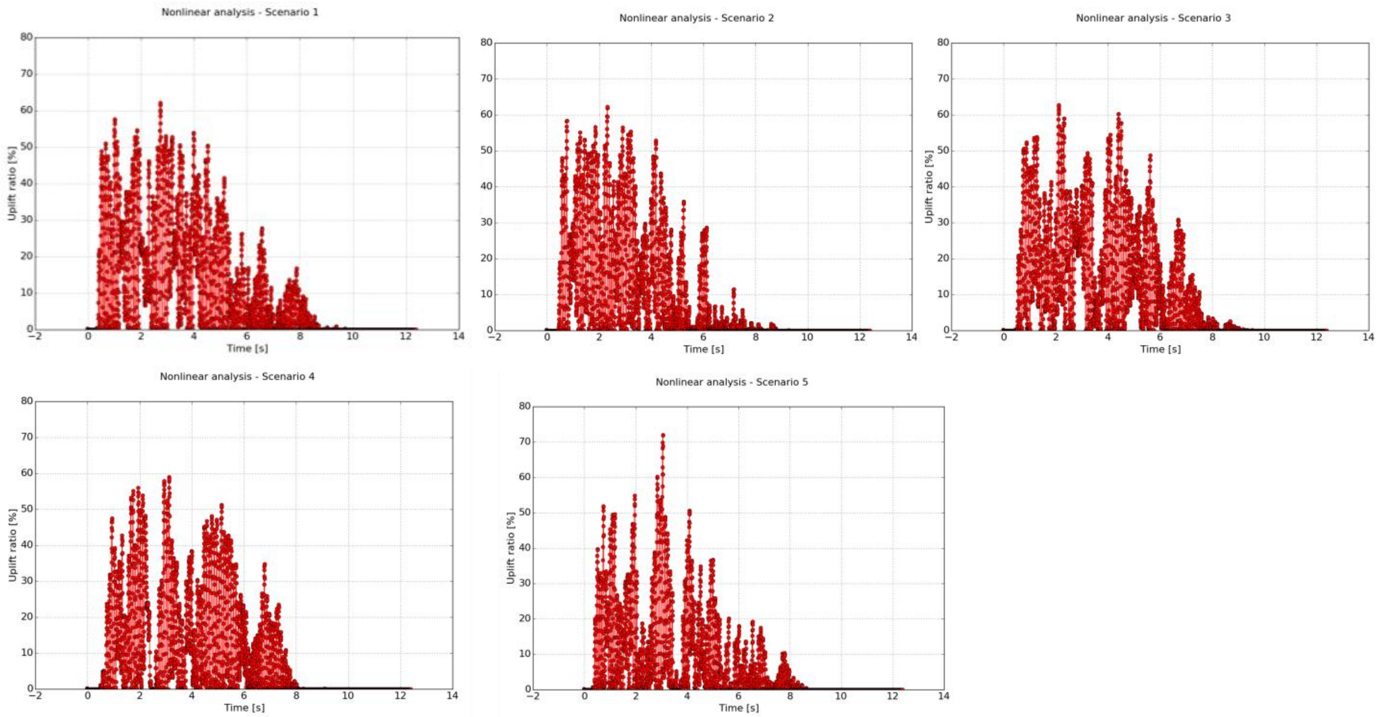

Figure 4. Evolution of uplift ratio over time – NLT analysis.

Figure 5 displays the cartography of the uplifted zone for the same scenario with linear analysis (Scenario 1, Figure 4a) and the moment of the maximal uplift, which occurs at 2.736s.

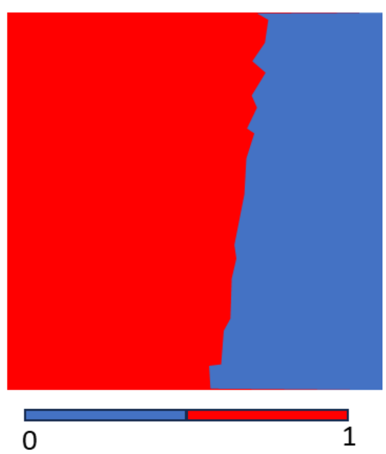

Figure 5. Diagram of maximal uplift ratio (0 no uplift and 1 uplift occurs) – Scenario 1 – NLT analysis.

The difference between the two methods is significant, with the maximum uplift ratio for the LT method at 0.44 and for the NLT method at 0.72. In the LT method, the elements introduced at the interface between the soil and the raft foundation possess both compressive and tensile strength, with the tensile strength preventing part of the potential uplift. In the compressive zone, stresses are higher at the edges of the foundation, as expected for normal stresses beneath a rigid footing on elastic soil. In the uplifted zone, normal stresses are practically zero (notwithstanding a numerical tensile strength that is defined in the contact elements).

Floor response spectra

As part of validating the relevance of the chosen periodic boundary conditions at the lateral boundaries of the model, since all the elements at the boundary follow strictly the same stratigraphy, spectra at different depths in the free field were calculated. In particular, it should be noted that the entire model exhibits a homogeneous response similar to the behavior observed in 1D. Figure 6 shows the comparison between the results of linear deconvolution and the 3D results in terms of free-field spectra (far-field zone). The response spectra obtained at different depths are comparable to the preliminary 3D analyses, with some higher accelerations for the overall model, which do not impact the results of this study (due to different damping used in the two methods – linear deconvolution → viscous damping / 3D analysis → Rayleigh damping).

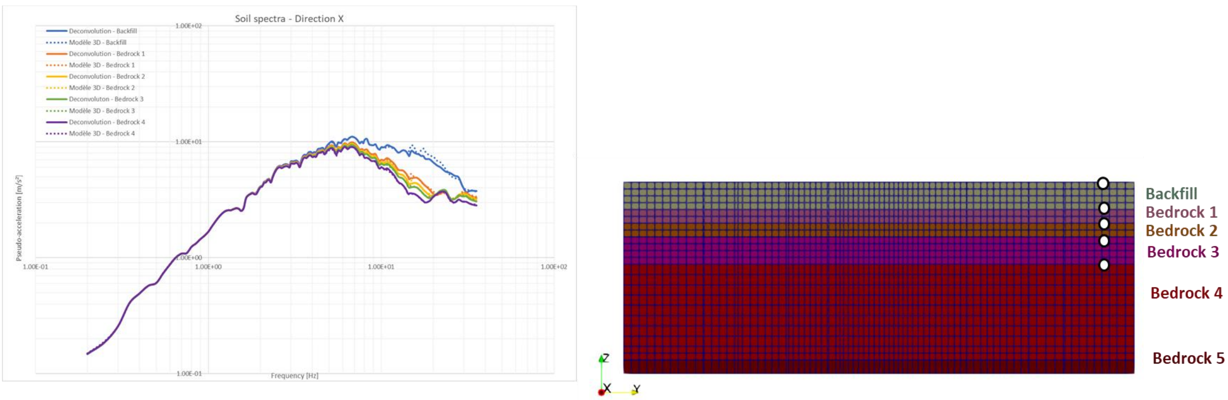

Figure 6. Response spectra – Free field- Response analysis/3D analysis.

By comparing the two analyses (LT and NLT) in Figure 7, we can conclude that, since the soil is linear and we are positioned far from the structure, the spectra do not show any differences.

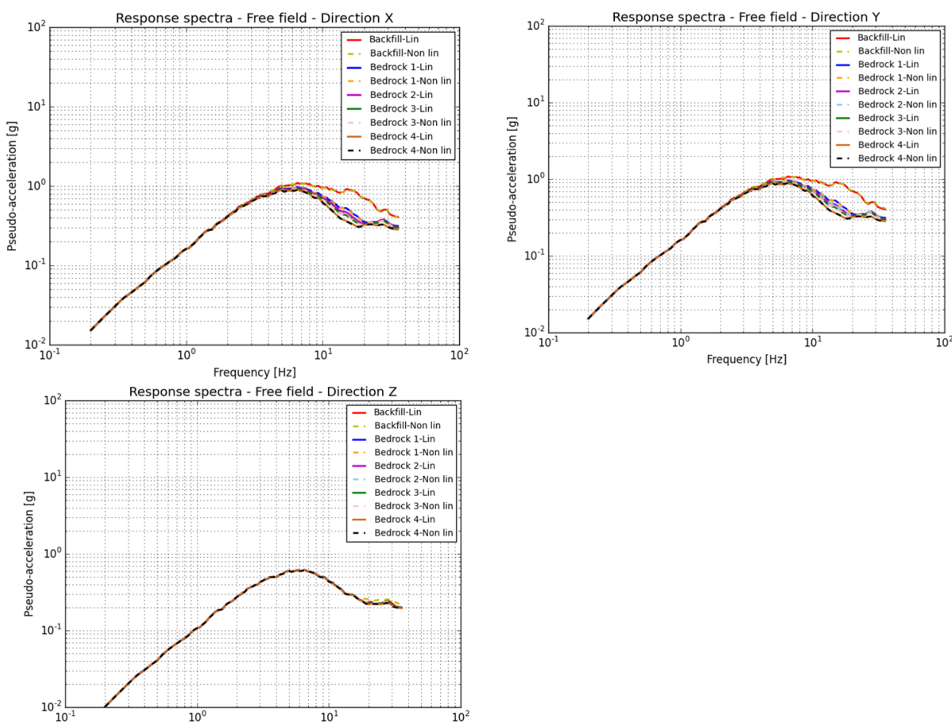

Figure 7. Response spectra – Free field- LT/NLT analysis.

Figure 8 presents the obtained 5% damping floor response spectra for the LT method and NLT method in both horizontal directions. As for the introduction of damping, the matrix of the global damping of the superstructure is calculated using the Rayleigh formulation. The definition of the parameters is based on the criterion that the critical damping between 0.1Hz and 40Hz is approximately equal to 7% or 4% (attributed value of damping in the corresponding part of the superstructure). Soil damping is also introduced by Rayleigh formulation as discussed before. It is important to note that the linear results are represented by a continuous line for the 4 selected points (extremities), while the corresponding results for the nonlinear method are shown with dotted lines.

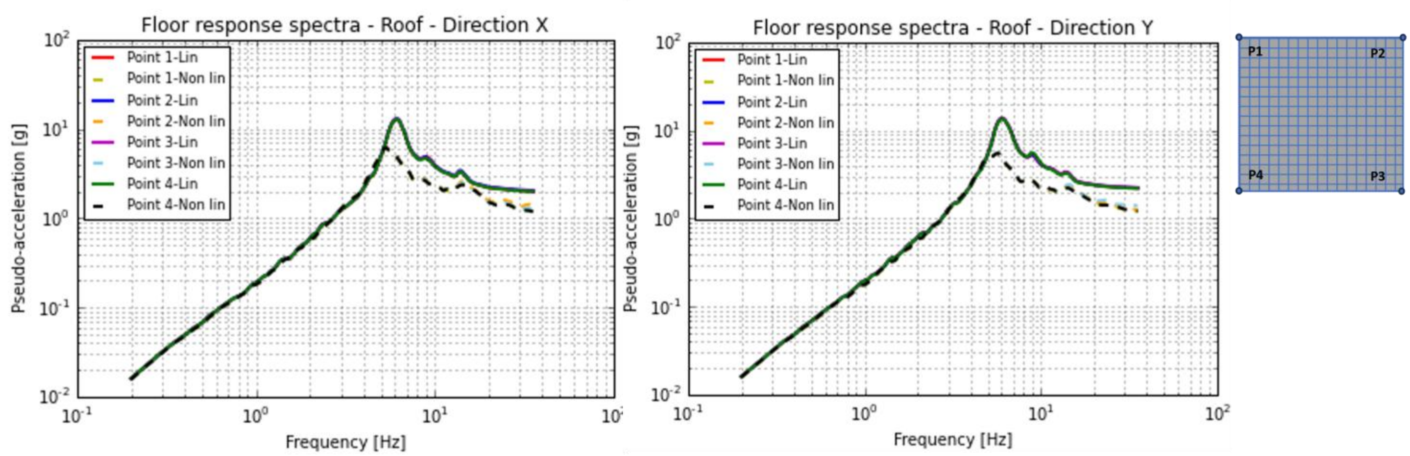

Figure 8. Roof response spectra –LT/NLT analysis.

In both horizontal (X and Y) directions, it is highlighted that foundation uplift can significantly influence the floor response spectra. The first mode for points located at the intersection with the vertical walls shows a slight shift to a lower frequency (LT method $\rightarrow f = 6$Hz whereas NLT method $\rightarrow f \approx 5$Hz), accompanied by a notable attenuation in terms of pseudo-acceleration. This attenuation is also associated with as shift towards richer frequency content in the higher frequencies.

CONCLUSIONS

The paper has presented a comparison between a linear and nonlinear direct method for a soil-structure interaction study. In order to assess the outcome of this comparison, a simplified method is considered and used as a reference. After discussing the basic theoretical principles for the two methods, a typical configuration of an industrial building was subjected to a seismic excitation with a particular emphasis on the nonlinear mechanism of foundation uplift. Several aspects of the overall structural response have been studied, such as calculated uplift ratio and floor response spectra at the roof level. At this stage of the study, only the raw results have been presented so as to identify the subtleties and fully comprehend the impact of foundation uplift on floor response spectra. In fact, foundation uplift can significantly influence the response of floor response spectra of a building in several ways:

- Changes in stiffness: When foundation uplift occurs, the effective stiffness of the foundation is reduced, which can alter the dynamic characteristics of the structure. As the foundation begins to uplift, the structure may experience reduced resistance to horizontal motion, leading to an increase in the displacement of the superstructure. This change in stiffness can shift the natural frequencies of the building, affecting the floor response spectra.
- Nonlinear effects at the soil-foundation interface: Uplift introduces nonlinear behavior at the soil-foundation interface, especially if sliding or uplift occurs. These nonlinearities can cause the structure to behave differently than in a linear model, which impacts the acceleration and

displacement response of the floors. The resulting floor response spectra can show higher peaks or a broader range of frequencies due to the nonlinear interaction.

In summary, high level of foundation uplift (such as the one presented in this paper) can cause significant changes in the floor response spectra by modifying the building's dynamic properties, shifting vibration modes, and introducing nonlinear effects at the soil-structure interface. The result is often a more complex seismic response with different frequencies and amplitudes compared to a purely linear model. The ultimate goal of the present work motivates to the introduction of an “uplift spectrum” which is related to the linear spectrum. Some qualitative characteristics for which potential quantification could be proposed include:

1. The frequency shift of the main peak observed in floor response spectra: uplift reduces the main peak, but establishing an empirical relationship between this phenomenon and the uplift ratio could be of great interest. For example, as observed in the present work, a 72% uplift ratio causes a frequency shift from 6Hz to 5Hz, which is not significant (approximately a 15% reduction).
2. Amplitude shift of the main peak: It seems that the amplitude shift of the main peak could result in either amplification or attenuation depending on the position of frequency shift on the design spectrum.

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