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ASSESSMENT OF ANALYSIS METHODS FOR LINEAR AND NONLINEAR SOIL-STRUCTURE INTERACTION IN VIEW OF INCREASING SEISMIC INTENSITY AND FOUNDATION UPLIFT

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Abstract

The consideration of soil-structure interaction in linear and nonlinear regime relies on different methods of analysis in seismic design studies. This paper is dedicated to the assessment of various methods for modeling soil-structure interaction in order to define their validity range within seismic design projects in Civil Engineering. Particular emphasis is given on the nonlinearity that is developed along the soil-structure interface (foundation uplifting). The validity range of each method is examined with respect to both the intensity level of seismic action and also some quantities of interest characterizing the response of a typical soil-structure system, such as the uplift ratio of the foundation, the floor spectra developed in representative points within the structure and the demand for reinforcement steel in characteristic sections. Based on the study of a configuration pertaining to a typical industrial building embedded in a homogeneous soil and described by a set of dimensionless parameters, a comparison is performed to identify the relevance and conservatism of each analysis method. Specifically, the study examines pseudo-static and time-history approaches based on the spring method and a FEM-BEM coupled approach.

Keywords: nonlinear soil-structure interaction; spring method; FEM-BEM coupling.

1. INTRODUCTION

Seismic loading has been ubiquitously recognized as a major factor for the design and construction of massive civil engineering structures in high seismicity regions, in particular industrial facilities and energy infrastructure. As requirements for safety and economic viability of such structures are becoming stricter and more demanding, a parallel need for more elaborate and precise analysis methods must be equally satisfied.

Accordingly, performing more accurate simulations for the physical phenomena governing an optimized seismic design constitutes an important step towards this objective. Dynamic soil-structure interaction effects arise as the main component of these simulations and it is essential that numerical models developed for seismic design and dimensioning must be enriched with consideration of such effects as soil plasticity, foundation uplift, sliding along foundation interfaces etc.

Various approaches have been presented so far in scientific literature for the consideration of soil-structure interaction; these can be classified according to the representation of the bounded and unbounded domains (soil: unbounded domain | superstructure with its foundation: bounded domain) in three main categories: a) direct methods, b) hybrid methods, and c) substructure methods.

In direct methods, a modeling based on conventional finite elements is used for both spatial domains exhibiting either linear or non-linear behavior; the dynamic response is obtained through integration of dynamic equilibrium equations in time domain. The method can be implemented in finite element codes of general application as long as an appropriate method for introducing the seismic excitation and boundary conditions has been adopted.

In hybrid methods, the complete soil-structure system is decomposed into two subdomains which are independently modeled. The basic principle of hybrid methods is to consider that all nonlinear phenomena are developed in the bounded domain and that the unbounded domain remains linear; thus, the linear unbounded domain is represented by boundary elements (Boundary Element Method-denoted BEM) and fully formulated in the frequency domain (real frequencies→ Fourier domain or complex frequencies→ Laplace domain) while the bounded nonlinear domain is discretized with the Finite Element Method (FEM), and resolved in the time domain. A FEM-BEM coupling method is therefore constituted which allows adopting the best numerical techniques for each subdomain. The implementation of hybrid methods, especially for the treatment of unbounded domains, requires the use of appropriate BEM software, such as SASSI2010 [1] and MISS3D [2].

Finally, substructure methods are used for evaluating the interaction problem for systems with linear or limited nonlinear behavior. The bounded domain (structure) is modeled with finite elements and the unbounded domain is discretized with boundary elements (different variants may be used: Thin Layer Method ([3]-[7]), Scaled Boundary Finite Element Method [8], etc.) The resolution is usually performed in the frequency domain but combinations of time and frequency domain solution methods can be also employed [8].

The present work focuses on assessing hybrid and substructure SSI methods in a linear and/or nonlinear setting in order to establish a quantitative comparison among the main approaches used in seismic design practice. Diverse methods are thus used to study a simplified soil-structure configuration pertaining to an embedded industrial building with basemat founded on a homogeneous soil profile; for simplicity, the only nonlinearity taken into account is the one developed along the soil-structure interface (foundation uplift). The validity range of each method is examined with respect to both the intensity level of seismic action and some quantities of interest characterizing the overall response, such as the basemat uplift ratio, the floor spectra in representative points, and the demand for reinforcement steel in several characteristic sections within the building.

It is noted that this study is related to a previous article [10], but presents several improvements and additions mainly with respect to the range of employed methods (hybrid methods are taken into consideration in this work) and also in terms of characterization of the overall response through the explicit calculation of reinforcement steel demand within the structural system.

2. PARAMETRIC STUDY

A comprehensive parametric study of a typical industrial building, simple enough to facilitate extrapolation of conclusions for a wider range of civil structures, is herein presented. The main objective of the study is to explore the relevance and conservatism of different calculation methodologies in view of seismic design practice and in reference to the basic analysis method prescribed by the majority of existing seismic design norms, which is a linear substructure method with the earthquake loading typically introduced as a pseudo-static force field after an appropriate spectral combination of characteristic maximal modal responses.

The considered structure is a symmetric embedded reinforced concrete building composed of continuous external and internal vertical walls, horizontal slabs and roof, and an interior substructure modeled as a single-degree-of-freedom oscillator, attached to the basemat. The building is founded on a mat foundation, which is allowed to uplift and lies over a purely cohesive soil, being considered as homogeneous and modeled as a linear viscoelastic half space.

Soil is characterized by viscoelastic effective parameters that depend on seismic intensity. The determination of effective soil properties is achieved using a 1D model of the soil stratigraphy in which a deconvolution of the seismic motion is performed from the ground surface down to the deepest layer (D + 30m in the present study; the depth of 30m below basemat surface is considered sufficient for the development 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 characterized by uniform undrained shear strength c_u together with curves for ratio $G/G_{\max}$ and corresponding damping ratio β versus shear distortion γ.

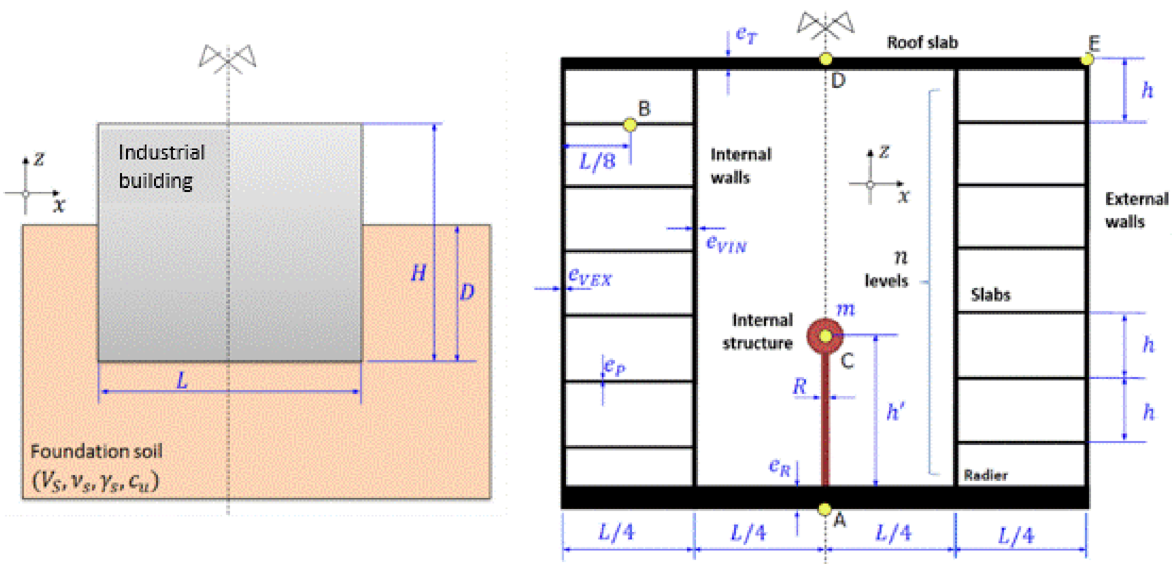

Fig. 1 – Soil-structure configuration examined in the parametric study

The configuration is parameterized on the basis of four dimensionless parameters reflecting aspect and embedment ratio of the building, rigidity contrast between soil and superstructure, and finally foundation bearing capacity with respect to building mass. In the context of the present paper, results from only one soil-structure configuration (the "reference" configuration) will be presented. The definition of dimensionless parameters together with their values for the "reference" configuration are presented in Table 1.

Table 1 – Definition of dimensionless parameters

Aspect ratio	Embedment ratio	Rigidity contrast	Bearing pressure ratio
$P_A = \frac{H}{L} = 2.0$	$P_E = \frac{D}{H} = 0.2$	$P_R = \frac{\omega_1 H}{V_S} = 2.5$	$P_N = \frac{N_{\max}}{mg} = 4.0$
$\boldsymbol{\omega_1}$: fundamental circular frequency of building; fixed-base conditions $\boldsymbol{V_S}$: shear wave velocity of soil medium $\boldsymbol{m}$: total mass of the building $\boldsymbol{N_{\max}}$: bearing capacity for a centered vertical load			

The "reference" configuration is studied for five levels of increasing seismic intensity (PGA ranging from 0.15g to 0.55g, step 0.1g) using both spring and FEM-BEM methods. In particular, the selected methods, which are all summarized in Table 2, range from a conventional linear pseudo-static analysis method (denoted LP) to a linear transient analysis method with FEM-BEM coupling.

Table 2 – Synthetic presentation of analysis methods used in the study

Category	Method	Loading	SSI	Nonlinearity	Seismic input	Analysis method
Spring methods	LP	Response spectrum	Sub-structuring Dynamic impedance Effective soil properties	None	H:Free field V:Free field	Modal analysis Spectral combination Linear pseudo-static analysis
	NLP	Response spectrum	Sub-structuring Dynamic impedance Effective soil properties	Uplift	H:Free field V:Free field	Modal analysis Spectral combination Nonlinear pseudo-static analysis
	LT	Three acceleration time history scenarios	Sub-structuring Dynamic impedance Effective soil properties Kinematic interaction	None	H: Kinematic inter. V: Kinematic inter. Θ: Kinematic inter.	Linear transient analysis
	NLTP	Three acceleration time history scenarios	Sub-structuring Dynamic impedance Effective soil properties Kinematic interaction	Uplift	H: Kinematic inter. V: Kinematic inter. Θ: Kinematic inter.	Nonlinear transient analysis
Hybrid methods	LT-FC	Three acceleration time history - 1 scenario	Sub-structuring Dynamic impedance Effective soil properties Kinematic interaction	None	H: Kinematic inter. V: Kinematic inter. Θ: Kinematic inter.	Linear transient analysis
	NLT-HLT	Three acceleration time history - 1 scenario	Sub-structuring Dynamic impedance Effective soil properties Kinematic interaction	Uplift	H: Kinematic inter. V: Kinematic inter. Θ: Kinematic inter.	Nonlinear transient analysis

A detailed description of the modeling and calculation hypothesis adopted for each analysis method can be found in [11].

Seismic loading is represented in the horizontal direction by the EUR design response spectrum for medium soils (*cf.* [15]) in which the vertical component is defined as 2/3 of the horizontal one. As for the transient analysis, input motion samples have been established by a set of three spectrum-compatible acceleration time histories. For simplicity, a planar seismic excitation is considered parallel to symmetry plane xz (*i.e.* horizontal component parallel to axis y is zero).

It is also noted that for all nonlinear seismic analysis, gravity initialization is performed by taking into account permanent and variable loads in the building.

3. SPRING METHOD

The approaches that use the spring method (LP, NLP, LT and NLTP) introduce a decomposition of the soil-structure interaction problem that under the fundamental hypothesis of linear viscoelastic behavior can be divided into three sub-problems: 1) the problem of kinematic interaction, 2) the problem of calculation of dynamic impedance, and 3) the problem of determination of structural response considering as boundary conditions at the base of the building, the solution of the second sub-problem, and imposing as loading, the result of the first sub-problem. The unbounded soil domain is replaced by four groups of translational springs coupled with dashpots (denoted T1 to T4) aiming at representing the dynamic impedance of the foundation. The values attributed to the vertical and horizontal stiffness of the springs in each group are given in the Table 3. Similar expressions apply for the damping terms.

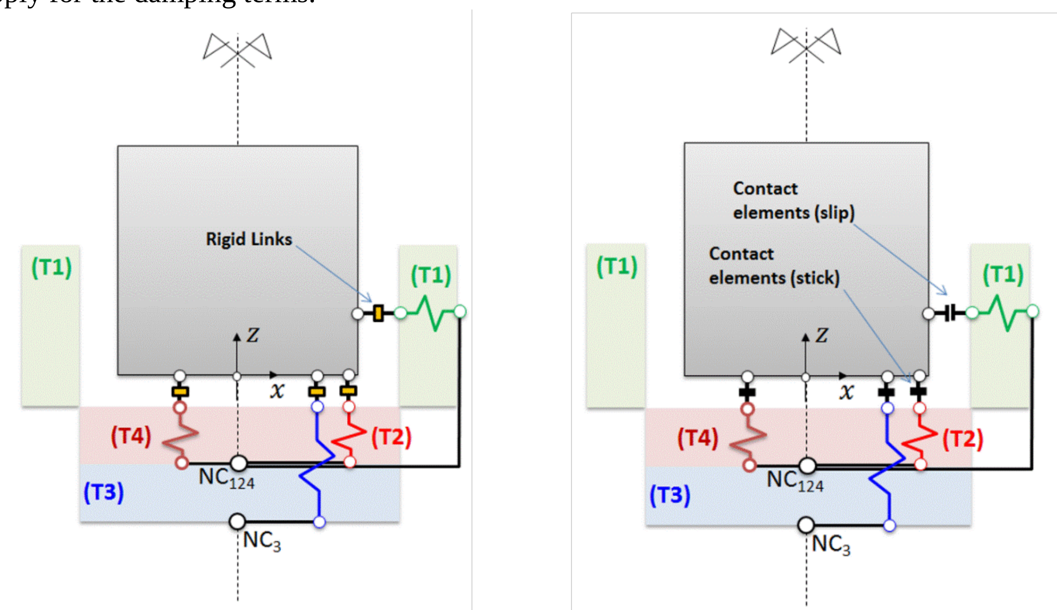

Fig. 2 – Modeling principle – spring method: a) for linear (LP and LT) methods and b) for nonlinear methods (NLP and NLTP)

Table 3 – Stiffness terms for the definition of foundation impedance springs

	Group (T1)	Group (T2)	Group (T3)	Group (T4)
$\boldsymbol{k}_{\mathbf{XX}}$	$k_{\mathrm{XX1}} = \dfrac{\mathrm{K}_{\mathrm{XRY}}}{n_B D}$	$k_{\mathrm{XX2}} = \dfrac{1}{n_R}(\mathrm{K}_{\mathrm{XX}} - 2n_B k_{\mathrm{XX1}})$	$k_{\mathrm{XX3}} = 0$	$k_{\mathrm{XX4}} = 0$
$\boldsymbol{k}_{\mathbf{ZZ}}$	$k_{\mathrm{ZZ1}} = 0$	$k_{\mathrm{ZZ2}} = 0$	$k_{\mathrm{ZZ3}} = \dfrac{\mathrm{K}_{\mathrm{ZZ}}}{n_R}$	$k_{\mathrm{ZZ4}} = \dfrac{A_R}{n_R I_R}(\mathrm{K}_{\mathrm{RY}} - 2n_B k_{\mathrm{XX1}} \dfrac{I_B}{A_B})$

$\boldsymbol{n_R}$: number of nodes in the mat foundation
$\boldsymbol{n_B}$: number of nodes of vertical wall in contact with the soil
$\boldsymbol{D}$: embedment depth
$\boldsymbol{A_R}$: area of the mat foundation
$\boldsymbol{A_B}$: contact area between vertical wall and soil
$\boldsymbol{I_R}$: moment of area of mat foundation
$\boldsymbol{I_B}$: moment of area of contact area between vertical wall and soil
$\mathbf{K_{XX}}$: horizontal impedance of the mat foundation (real part)
$\mathbf{K_{ZZ}}$: vertical impedance of the mat foundation (real part)
$\mathbf{K_{RY}}$: rotational impedance of the mat foundation around axis y (real part)
$\mathbf{K_{XRY}}$: coupling term (horizontal translation-rotation, real part)

For nonlinear methods that employ these groups of springs (NLP and NLTP), soil-structure interfaces (along the mat foundation and the embedded vertical walls) are modeled with tensionless node-to-node contact elements allowing for an eventual detachment between the structure and the soil. It has been considered that the basemat-soil interface exhibits an infinite friction resistance (rough interface) and the vertical walls-soil interfaces exhibit zero friction resistance (smooth interfaces).

4. FEM-BEM COUPLING

Alternatively to the spring method, where the frequency dependence of the impedances is not taken into account, hybrid methods with frequency-time domain coupling (LT-FC and NLT-HLT) can allow for a dynamic resolution with preservation of full frequency-dependence of foundation impedance matrix. For the numerical implementation, the FEM model for the bounded substructure (herein, the FEM code *Code_Aster* [12] is used) is coupled to a BEM formulation of the soil impedance matrix that implicitly accounts for the inertial and kinematic soil-structure interaction. This soil impedance matrix as well as the equivalent seismic loading (imposed at the geometric center under the assumption of rigid foundation) are computed with MISS3D [2].

The so-called LT-FC method relies on a resolution performed entirely in the frequency domain where the soil-structure system is considered to be linear and the frequency-dependence of foundation impedance is fully preserved. In this framework, the FEM-BEM coupling is formulated via the introduction of a modal basis where force and displacement fields are projected on. For the computation of soil-structure interaction, MISS3D needs a modal basis comprising a set of trivial (zero) eigen-modes on the soil-structure interface combined with non-zero modes on the same interface. For the first set of eigen-modes, one generally takes the structural eigen-modes obtained by blocking displacements on the interface; as for the second, one usually considers the set of static eigen-modes (also known as “constrained” eigen-modes), recursively obtained by imposing a unit displacement along every degree of freedom of the nodes located on the interface (foundation). It should be noted that the required computing time and resources (currently the possible number of interface nodes is restricted to 10,000 because of the direct MISS3D’s solver) almost make it impossible to model the effects of soil-structure interaction with the complete set of constrained static modes; it is concretely a question of reducing the size of the discretized system (the total number of degrees-of-freedom) by replacing the complete set of constrained static modes by a small number of foundation eigen-modes calculated through the use of spectral properties of the dynamic operator condensed to the interface and chosen according to an appropriately established criterion (Balmes criterion [16]).

If nonlinear behavior has to be accounted for, the hybrid method (NLT-HLT) can be considered. In this approach, all nonlinearities are confined in the bounded FEM domain that includes the superstructure but also a possibly nonlinear domain of surrounding soil (near-field soil domain). On the contrary, the far-field soil domain is assumed linear and hence, it can be resolved by means of a BEM formulation. The nonlinear problem must be formulated in the time domain, and for stability issues, the soil impedance matrix has been shown to provide better results if initially computed in the Laplace domain [13]-[14] and then converted into the time domain. Particularly interesting is the study of foundation uplift, a geometric nonlinearity that can be modeled in

Code_Aster [12] by following different strategies. According to the most common one, the nodes between the superstructure and the soil have to be duplicated resulting in two interfaces linked by contact finite elements. The (time-dependent) soil impedance matrix is then assembled to the external interface and the coupled soil-structure interaction problem is finally solved in the time domain.

5. RESULTS

The results of this work focus on three specific aspects of dynamic response, namely: a) maximum uplift ratio developed during loading, b) floor response spectra in positions of interest within the building, and c) demand for reinforcement steel in characteristic sections of the structural system. Uplift ratio is calculated as the ratio of area of uplifted zone over total basemat area. In particular, for linear methods, uplift ratio is calculated based on the number of rigid links which are in tension, and for nonlinear methods, based on the contact elements that exhibit a zero normal force.

Fig. 3 and Fig. 4 present the considered positions for the calculation of these quantities. In particular, floor response spectra are calculated in five positions either in the horizontal direction, the vertical direction or in both. Moreover, obtained results for transferred floor spectra are presented in dimensionless form as spectral ratios in order to facilitate the comparison among different methods; all calculated response spectra are normalized by division with the corresponding spectrum acquired by method LP. Reinforcement steel demand is calculated at the base of the external and internal walls and also at the foundation basemat.

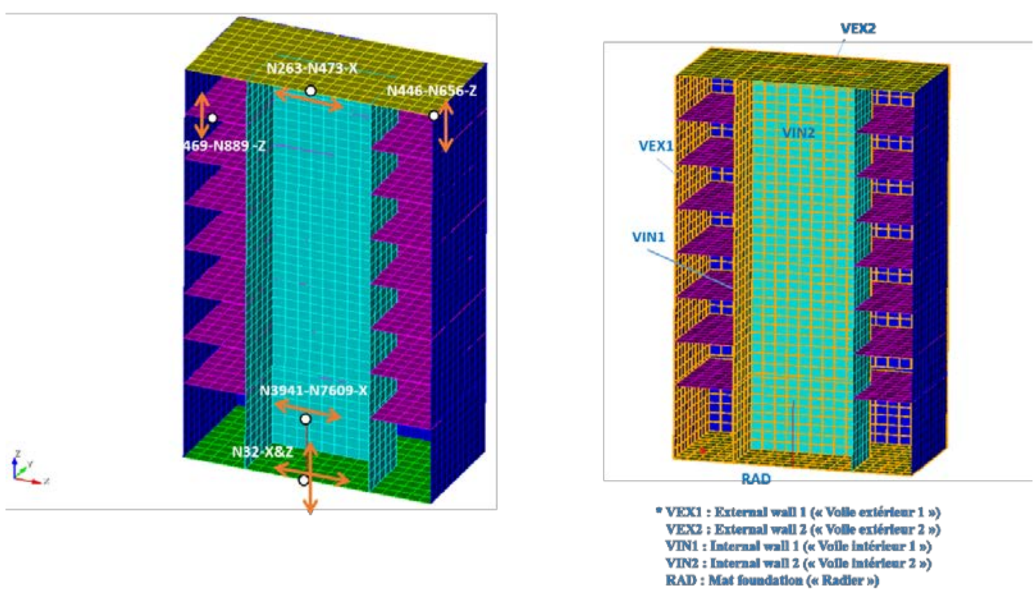

Fig. 3 – Positions for calculation of transferred floor spectra and reinforcement steel demand

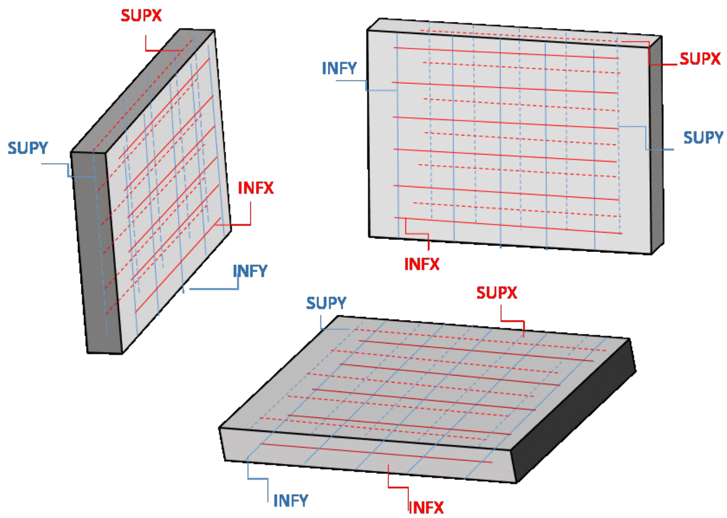

Fig. 4 – Definition of wall's axis and faces

The obtained results are presented in a synthetic form so as to allow for a quantitative comparison of each method with respect to "reference" method LP. This comparison highlights various aspects of the overall response, such as the effect of the kinematic interaction, the gain in passing from pseudo-static to transient analysis, the relocation of seismic surcharge within the structure when the uplift is initiated, etc. Constructing multiaxial radar diagrams, where selected performance criteria form vector $\mathbf{a} = \{a_i\}$, facilitates the synthetic presentation of results. This vector is represented in each radar diagram as function of analysis method and seismic intensity.

Maximal Uplift ratio

Fig. 5 presents the obtained five-axis radar diagram, which corresponds to the calculated uplift ratio for each seismic intensity level. Each analysis method is presented in the diagram by a different curve whose color and type correspond to a particular feature of the analysis method; blue curves correspond to pseudo-static methods and red curves to transient methods. Continuous lines correspond to linear methods, whereas dotted lines to nonlinear methods. Green continuous line corresponds to method LT-FC, which is the linear variant of coupled FEM-BEM methods. Axes represent maximal uplift ratio, which may vary from 0 (no uplift) to 1 (total basemat detachment).

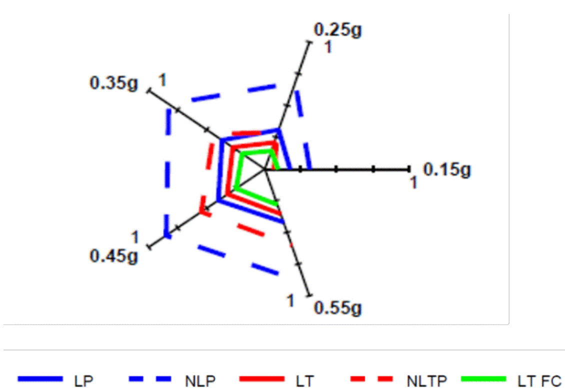

Fig. 5 – Radar diagram of uplift ratio

The results in Fig. 5 reveal that method NLP leads to the highest uplift ratio for all seismic intensities, confirming in this way the excessive conservatism of this approach as regards calculation of uplift ratio and assessment of overall stability. In particular, uplift ratio increases rapidly to more than 70% for intensities of 0.25g or larger, anticipating loss of overall stability from overturning and/or bearing pressure failure.

Another important point is that the results confirm the envelope character of pseudo-static methods (linear LP and nonlinear NLP) with respect to corresponding transient methods (linear LT and partial nonlinear NLTP), highlighting the conservative character of pseudo-static methods. For linear methods in particular, passage from LP to method LT may lead to a reduction in the calculated uplift ratio of the order of 15%. As for the transient methods, passage from LT to NLTP has negligible effect on results for intensities smaller than 0.25g. For larger intensities, however, the two methods diverge because in any case, uplift ratio for linear methods cannot exceed a theoretical limit, which is 50% for the examined symmetric structure. Finally, coupled FEM-BEM linear method LT-FC produces by far the smallest uplift ratios compared to other methods.

Floor spectrum ratios

Fig. 6 and Fig. 7 present radar diagrams related to floor spectra ratios, calculated in two characteristic frequencies: a) at f =35Hz (*cf.* Fig. 6) and b) at the frequency which yields the maximal spectral ratio (*cf.* Fig. 7). The first result highlights the accuracy of different methods in evaluating maximum seismic accelerations at different points of the structure, whereas the second one reveals the level of conservatism of the reference method in relation to each considered method.

Each diagram possesses seven axes; the first one (axis parallel to $+x$) represents the calculated uplift ratio; the following six axes provide calculated spectral ratios in the selected positions within the structure (positions which have been presented in Fig. 3). A separate radar diagram is provided for each seismic intensity level and contains five curves as in Fig. 5. It should be noted that for method LP, spectral ratios are by definition equal to 1 and that no distinction is made between LP and NLP (pseudo-static methods). Transferred response spectrum calculation in these cases is performed using code FSG [17] directly based on modal characteristics of the structure.

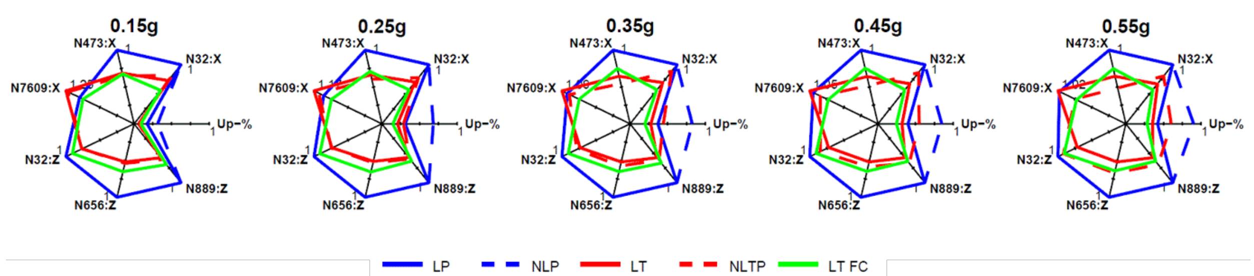

Fig. 6 - Radar diagrams of spectral ratios @f=35Hz

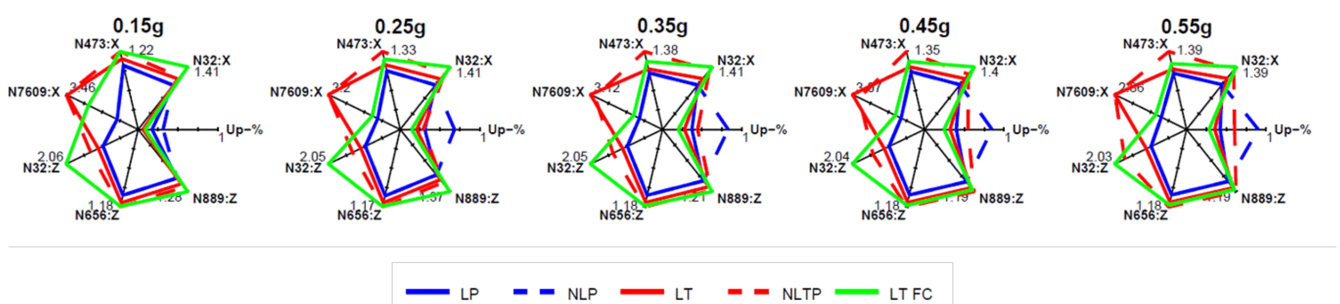

Fig. 7 - Radar diagrams of maximal spectral ratios

Results of radar diagrams related to spectral ratios reveal that, as regards maximum accelerations (*cf*. Fig. 6), differences for increasing intensity levels within the same analysis method remain relatively minor. In terms of comparisons among various methods, FSG methodology yields in most cases envelope results.

On the other hand, regarding maximal spectral ratios (*cf*. Fig. 7), it can be readily observed that variability in terms of intensity level is more significant than in the case of maximum accelerations. These results indicate that there are some frequencies for all transient methods that FSG methodology is not the most conservative one. This suggests that critical appreciation of FSG results is always required, especially for parts of global structural models that exhibit "singularities" in dynamic response (e.g. modes with significant local effects that are not contained in the modal basis used for FGS calculations). It is also interesting to note that transient spring method is very severe for Node 7609 (internal structure) while FEM-BEM method is more severe for Node 32 (basemat).

Demand for reinforcement steel

The third examined aspect of structural response concerns the demand for reinforcement steel at selected sections, presented in Table 4 . Radar diagrams related to the demand in external and internal walls are presented in Fig. 8, while Fig. 9 presents the demand for reinforcement steel in the mat foundation. These diagrams follow the same presentation approach as in Fig. 6 and Fig. 7.

The obtained results reveal that the envelope character of simplified methods is less evident in what regards the evaluation of forces and thus the demand for reinforcement steel. Factors related to this observation are: a) the large variability of nonlinear transient analyses and b) differences in adopted models for structural damping between transient and modal-spectral pseudo-static approaches.

They also reveal that for the pseudo-static methods the effect of the foundation uplifting does not necessarily lead to a discharge of the structural elements. It could be possible that in the same structure exist not only unloaded zones but also zones highly stressed. These diagrams, however, facilitate the comparison with respect to the evolution of the demand in reinforcement steel in conjunction with increasing intensity levels and the effect of nonlinearities.

Table 4 – Definition of the selected sections for the calculation of the demand for reinforcement steel

Element	X-sup	X-inf	Y-sup	Y-inf
RAD	✓	✓	✓	✓
VEX1	-	-	✓	-
VIN1	-	-	✓	-
VEX2	✓	-	✓	-
VIN2	✓	-	✓	-

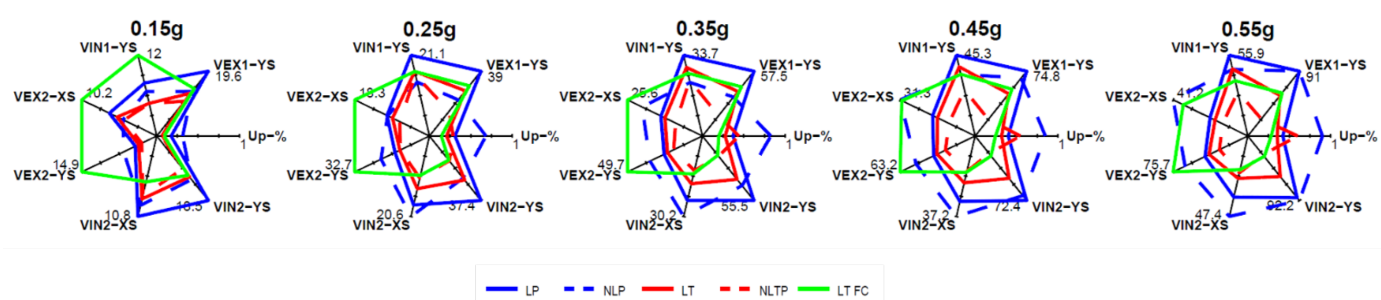

Fig. 8 - Radar diagrams of demand for reinforcement steel in external and internal walls

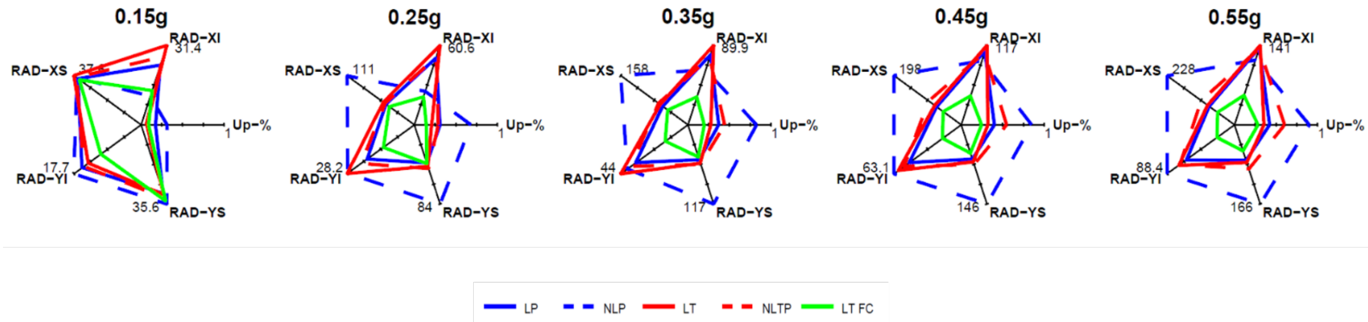

Fig. 9 - Radar diagrams of demand for reinforcement steel in mat foundation

6. CONCLUSIONS

The present work has been concerned with a classification of seismic design approaches for linear and nonlinear soil-structure interaction 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, floor response spectra, and the demand for reinforcement steel at several representative sections within the structural system. The significant volume of obtained results has been synthetically presented using radar diagrams which allow for a direct comparison among diverse analysis methods and with respect to various levels of shaking intensities.

The main outcome of the study has been compiled as a design methodology (see also [10]) of buildings in the presence of uplift which is concretized in the following elements:

1. Method LP is the reference method for seismic design. Soil-structure interaction is modeled by linear springs connected to the basemat which can develop both tension and compression forces. Given the design seismic excitation prescribed for the structure, the designer implements LP to calculate uplift ratio at the basemat level (denoted as κ_{LP}). If this ratio is restricted (below 10%), it is implied that uplift has negligible effect on the overall structural response. Kinematic quantities (transferred spectra) can be calculated by modal synthesis techniques or by means of linear transient analyses. Seismic design of the overall structure (basemat included) can be performed on the basis of internal forces as calculated with LP.

2. If κ_{LP} exceeds 10% but stays inferior to 30% (normative limit of the present state-of-the-art), it is inferred that moderate to significant uplift takes place but an estimation using linear methods remains realistic. In this case, it is suggested to neglect uplift effects for structural design of all elements except for the basemat. As flexural curvature is the critical quantity for basemat design, consideration of tension forces in SSI springs is non-conservative for calculation of steel reinforcement demand in the basemat. In order to generate a more realistic and conservative force state for basemat design, the designer is invited to introduce tensionless SSI springs at the basemat level (method NLP) and to apply the required fraction of the pseudo-static seismic forces in order to recuperate κ_{LP} as basemat uplift ratio.

3. Finally, if κ_{LP} exceeds the limit of 30%, it is inferred that uplift is significant to the extent that linear methods are no longer sufficient for quantifying the effects of uplift on structural response. Therefore, the designer is invited to implement more elaborate methods for an accurate assessment of seismic stability, such as transient methods (LT and LT-FC) with or without consideration of nonlinearities. Upon implementation of an improved uplift ratio calculation, two scenarios can arise :

    - Corrected uplift ratio (denoted in this case as κ_{NL}) is inferior to 30% validating the use of linear methods for calculation of floor spectra and reinforcement steel demand. The designer can implement the procedure described in step 2 and the target uplift ratio is now set equal to the corrected value κ_{NL}.

- Corrected uplift ratio κ_{NL} remains larger than 30%. In this case, seismic response is dominated by uplift. Structural design and floor spectra should be calculated using a nonlinear transient analysis approach, which accounts rigorously for the effects of uplift.

Notwithstanding the elements provided in the present paper, further calculations for the NLT-HLT analysis method are currently in progress and their results are expected to be integrated with those of the rest of the methods. Moreover, the parametric study can be pursued by studying different soil-structure configurations so that the outcome of the conclusions may cover a larger scope of civil structures.

Finally, the ultimate perspective of the work consists in completing the design norms with guidelines that identify the most adapted hierarchy of analysis methods for design cases with increasing levels of shaking and accordingly, more pronounced effects of nonlinear soil-structure interaction.

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