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ADVANCES IN 3D NUMERICAL SIMULATION OF SEISMIC RESPONSE OF SAFETY-RELATED NUCLEAR STRUCTURES

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Abstract

In the event of an earthquake at a nuclear power plant, it is vitally important to ensure the survival of personnel and the integrity of high-value equipment. The objective of this study is to analyze the dynamic response of a safety-related nuclear structure subjected to near-margin seismic signals and to calculate the corresponding in-structure response spectra. A set of seed signals conveniently extracted from databases are selected to generate seismic signals compatible with the target spectrum. These signals are translated to the bedrock level using an iterative deconvolution process. The numerical modeling considers the non-linearities coming from the structural behavior and from the sliding and rocking at the soil-structure interface. The FEM model includes the soil layers up to the bedrock and cyclic symmetry conditions at the lateral boundaries. Reinforced concrete is modeled using the mixture theory that allows combining non-linear constitutive models of damage and plasticity. The floor spectra are calculated in the positions corresponding to the equipment. Finally, the results show how the non-linearity of the material hardly affects the global response of the analyzed structure. However, the sliding nonlinearity significantly affects the response spectrum of the structure, reducing maximum acceleration peaks.

Keywords: Seismic Hazard Analysis, Equipment Safety, Safety-related Nuclear Structure, Material Non-linear Analysis, Sliding Contact Interface.

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1 INTRODUCTION

After the "Fukushima I NPP" event on March-2011, a reassessment of the current nuclear facilities, in the face of demanding earthquakes, appeared to be necessary, incorporating non-linearities in the structural analysis. Modifications have been also proposed in the calculation standards to include non-linearities [1].

A safety-related structure of a NPP is studied herein. Since its analysis involves non-linearities, a complete time integration is carried out, what allows capturing the entire seismic response of the structure. For this reason, a FEM code with explicit integration is chosen.

As input data, the geometry of the structure discretized in finite elements is required, including the properties of its materials and the profile of the underlying soil layers. A set of seed signals are adjusted to target spectrum. These seismic signals are deconvolutioned to the bedrock and applied to the computational model.

The main result is the seismic response of the structure in terms of displacements, velocities, accelerations, and also the levels of damage or plastic strains in each material point of the structure. These first results are post-processed to obtain in-structure response spectra (ISRS), also called floor spectra.

2 GENERATION OF SEISMIC ACCELEROGRAMS

The reference seismic signal is defined as the Seismic Margin Earthquake (SME). The target spectrum corresponds to the UHS spectrum calculated, using the probabilistic seismic hazard method [2], at the foundation base level with the seismic parameters at the location of the building. A seismic source identified as near-field signal with a magnitude of 6.3 and an epicentral distance of 2km. Existing acceleration time histories (seed signals) are selected from the database of the *Pacific Ocean Earthquake Research Center (PEER)*[1]. These signals are fitted to the target spectrum for the nuclear construction site, using a spectral matching technique that modifies the frequency content of the seed signal. For this calculation, the wavelets algorithm proposed by Abrahamson [3, 4] or the algorithm proposed by Al-Atik [5] may be employed.

A family of seven acceleration time histories defined for the three spatial directions {X, Y, Z} is defined to account for the variability and uncertainty of ground motion. Satisfactory adjustments to the target spectrum is obtained in all of them. In near-field motions, the directionality of the signal is determined by the short distance to the seismic source. To consider this variability, the following directionality factors have been considered: {0.81, 0.89, 0.94, 1.0, 1.06, 1.13, 1.23} and applied to the family of seven signals employed in this analysis. It should be noted that these signals are defined at the surface level, at the level of the foundation of the structure. Subsequently, they must be deconvolutioned to the bedrock of the soil model, also considering the uncertainty of the soil properties.

The soil profile below the structure's foundation is obtained from the existing geological data. To address the uncertainty of the mechanical characteristics of the soil layers, a statistical dispersion study is carried out on the geological soil profile. In this way, a sample of 1000 soil profiles has been generated following a correlation according to the Toro model [6] and considering the following criteria:

- Geophysical properties, such as shear wave velocity (Vs), damping, and layers density, follow a log-normal distribution with a median value equal to the base profile and a dispersion coefficient of 0.35.

[1]PEER Database Website: https://ngawest2.berkeley.edu/ (last accessed 04/07/2021)

- The thicknesses of the soil layers are considered to be invariant, and are not subject to statistical dispersion. This simplifies the subsequent geometric modeling of soil layers.

Three different soil profiles are extracted, thus: Upper Bound (UB), Best Estimate (BE) and Lower Bound (LB), corresponding to percentiles 16%, 50% and 84% of the sample population, respectively. In this way, the three representative profiles are defined by the shear wave velocity (Vs), the damping ratio (ξ) and the density (ρ). Figure 1 shows the shear wave velocity and damping ratio for these profiles. The Young's modulus for each soil layer is calculated as a function of the shear wave velocity [7, 8]. Consequently, the Poisson's ratio is obtained from the elastic relationship between Young and shear modulus.

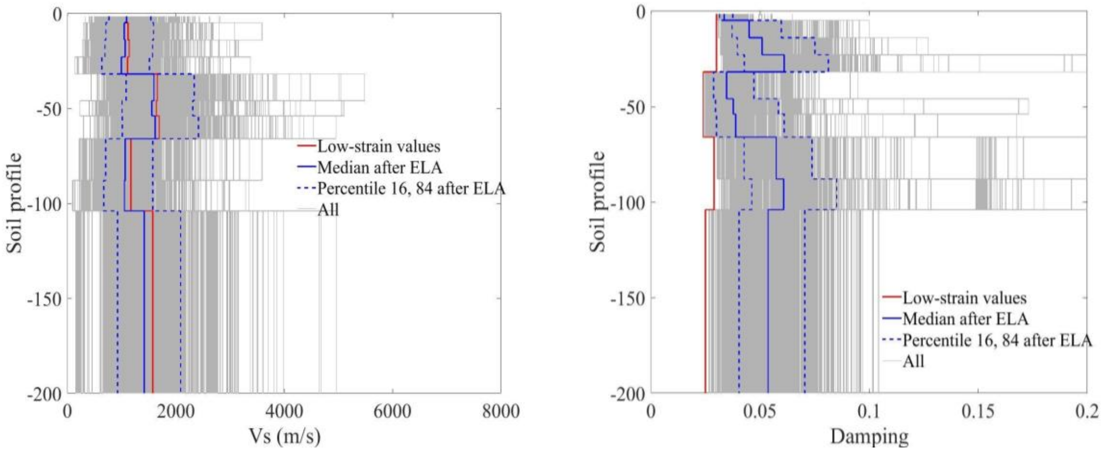

Figure 1. Soil profiles LB, BE and UB. - Shear wave velocity (left) and damping ratio (right).

The numerical analysis requires the seismic signal to be applied at the bedrock, its deconvolution is performed by using an iterative procedure of wave propagation [9]. To take into account the non-linear properties of the soil, the equivalent linear method is employed to obtain the degraded layers' properties for the three soil profiles: LB, BE, UB. [10, 11].

3 MODELLING OF THE SOIL-STRUCTURE INTERACTION PROBLEM

The finite element method (FEM), combined with an explicit strategy of time integration, is used to calculate the effect produced by each seismic signal on the studied building. This methodology allows taking into account the non-linearities, not only in the materials but also in the soil-structure interface. Two sub-models of different nature are defined: one, to model the behavior of the structural part, and the second, to model the free field behavior of the soil. Finally, both models are conveniently combined with a contact algorithm that allows capturing the non-linear effects happening at the interface. The structural sub-model includes all the reinforced concrete components from the foundation to the roof, also including the layer of lean concrete used to level the foundation base. The soil sub-model includes all the layers from the foundation base the bedrock, where the deconvolutioned accelerograms are applied. Each sub-model is described henceforward.

3.1 Modelling of the structure

An existing nuclear structure where auxiliary and control tasks are carried out, is studied herein. It has an almost square shape with a semi-circular cut on the south façade, due to the presence of the containment building. The structure presents a plane of symmetry with respect to the Y direction, while a lack of symmetry is observed in X direction, as it can be seen in Figure 2 (left). This figure shows a schematic view of the plan (64.3 m long and 61.9 m wide, with a semi-circular cutout of 22.5 m radius). Figure 2 (right) shows an elevation view of the structure. The foundation level is at level +91.00. Above, the building has five floors with levels: +96.0 (intermediate floor), +100.0, +108.0 and +114.5, and +120.7 (roof level).

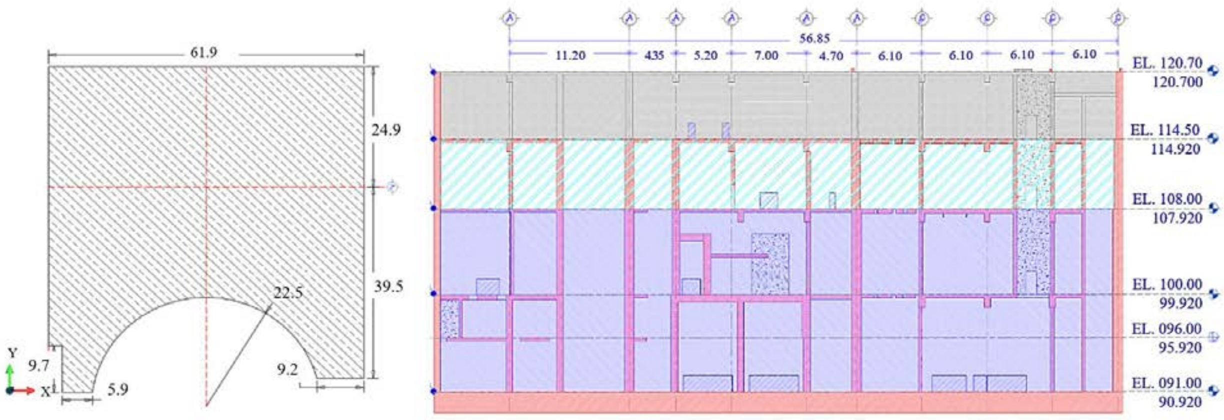

Figure 2. Geometry of the structural sub-model. Plan view (left) and elevation view (right).

The structural elements of the building are: shear walls, columns, beams and slabs. Various types of reinforced concrete sections are identified for each element depending on the arrangement and quantity of the steel bars, according to the reinforcement project. The structure is discretized with finite elements of shell type, known as 3-node rotation-free shell elements [12]. These elements have an adequate flexural behavior and simplify the assembly with the solid hexahedral elements used to model the foundation and the lean concrete, since both element types only have translational degrees of freedom.

The constitutive law that governs the material behavior of reinforced concrete is approached through the theory of serial-parallel rule of mixtures proposed by Rastellini [13]. This theory allows obtaining the behavior of the composite material by taking into account the appropriate sum of the non-linear constitutive models of its constituent materials. An isotropic damage model is used for the concrete, and an elastoplastic model for the structural steel in each section. The effect of the material non-linearity is one of the aspects analyzed. The reinforcement steel is modeled with an elastoplastic model, while an isotropic damage model with exponential hardening is selected for the structural concrete. The fracture energy parameter for the damage model is taken from previous works [14-15]. Since the original concrete has aged over the years, the characteristic strength of the concrete is considered to have increased by 25% and the stiffness by 11.8%.

Properties	Structural concrete (H-375)	Structural steel	Foundation concrete (H-375)	Lean concrete (H-150)
Constitutive model	isotropic damage	elasto-plasticity	elastic	elastic
Density	2400 kg/m3	7850 kg/m3	2500 kg/m3	2300 kg/m3
Young's modulus	28.7 GPa	210.0 GPa	28.7 GPa	18.14 GPa
Poisson 's Ratio	0.2	0.3	0.2	0.17
Yield strength (fc, ft)	36.8 Mpa, 3.8 MPa	470.9 MPa	-	-

Table 1. Properties of the materials of the elements of the reinforced concrete roof in the building.

The foundation slab is also made of reinforced concrete, with dimensions 61.0x63.6 m and 2 m thick. Its irregular geometry has been discretized with a structured mesh of 8-node linear hexahedrons. Below, it rests on a layer of lean concrete with a similar geometry and dimensions. Although the original thickness was variable, a uniform thickness geometry of 4m has been modeled for simplicity. Likewise, linear hexahedrons with 8 nodes are used. Both parts, the foundation and the lean concrete layer, are assumed to behave within the elastic range.

Material properties are briefly described in Table 1 for the concrete and steel fraction, for all structural elements of the building (aged mechanical properties for concrete are shown). The damping has been set at 10% for all the structural elements following the normative recommendations [16].

The possible interactions with the adjoining buildings have been dismissed since they are separated from the building under study by means of joints. However, there are adjacent soil layers that do interact with the building. The effect of the soil pressure on the walls is considered in those parts where the structure is effectively exposed to contact with the surrounding soil. In order to address the problem of the boundary constraints caused by the surrounding soil, a simplified approach based on springs is developed. Thus, a rigid frame was considered to replicate the shape of the building's silhouette and to apply the elastic constraints imposed by the surrounding soil. This rigid frame copies the movement of the upper layers of the soil by means of multi-point constraints. Then the structure is linked this frame, by means of elastic springs, in order to replicate the lateral soil restraint. The stiffness of the horizontal springs is based on a ballast coefficient of 40 MN/m3, which is a conservative value for the type of existing soil. Figure 3 shows a scheme of the horizontal constraint due to boundary conditions.

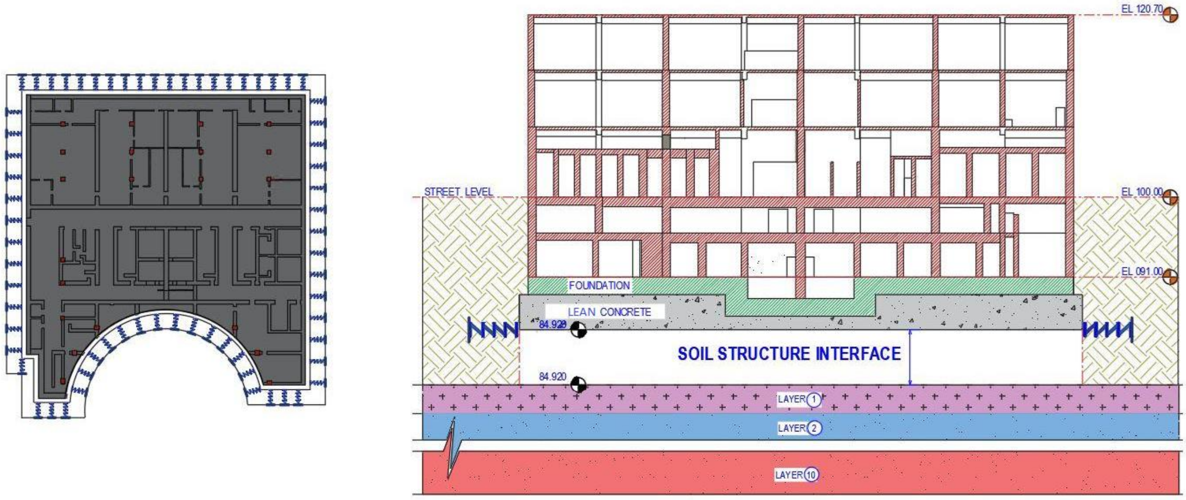

Figure 3. Horizontal constraint due to lateral boundary conditions.

3.2 Modelling of the soil

The thicknesses of the soil layers correspond to the profile identified by existing geotechnical surveys. Ten layers of soil are taken with their best estimation of thickness and mechanical properties. The bedrock was identified as the depth at which shear wave velocities exceeded 2,500 m/s, resulting in a model depth of 198.30 m.

The horizontal dimension of the soil model is defined by the distance between the limits of the foundation and the boundary of the soil. This length must be 3 times the foundation size, which leads us to a minimum dimension of 600 m x 600 m, to prevent that waves could be reflected from the model boundaries to the structure generating disturbances in the records, and, subsequently, altering the response spectra.

Cyclic symmetry boundary conditions are applied to the lateral faces in order to assume that the soil domain radiates the ground motion to infinity [17]. In this way, all pairs of points on opposing faces are rigidly linked, as master and slave, in terms of displacements, velocities and accelerations.

3.3 Full soil-structure model

The full numerical model is created by joining the structure and soil models. An interface has to be defined between both models. Figure 4 shows the location of the soil-structure interface. This interface is located just below the lean concrete layer in contact with the first layer of soil. Between these two surfaces, a contact algorithm is defined that uses penalty parameters to counteract the possible penetrations at the interface level. The present study considers three types of models, with different hypotheses for this interface:

- fixed base (FB), which assumes that the structure does not interact with the soil in any way, so no contact exists and the structure base is considered fixed to bedrock (surface signals are applied in this case).
- fixed contact (FC), which assumes that both sides of the interface move together without relative motion.
- sliding contact (SC), which allows relative displacements of gapping and horizontal sliding with a friction coefficient of 0.5 (and null cohesion).

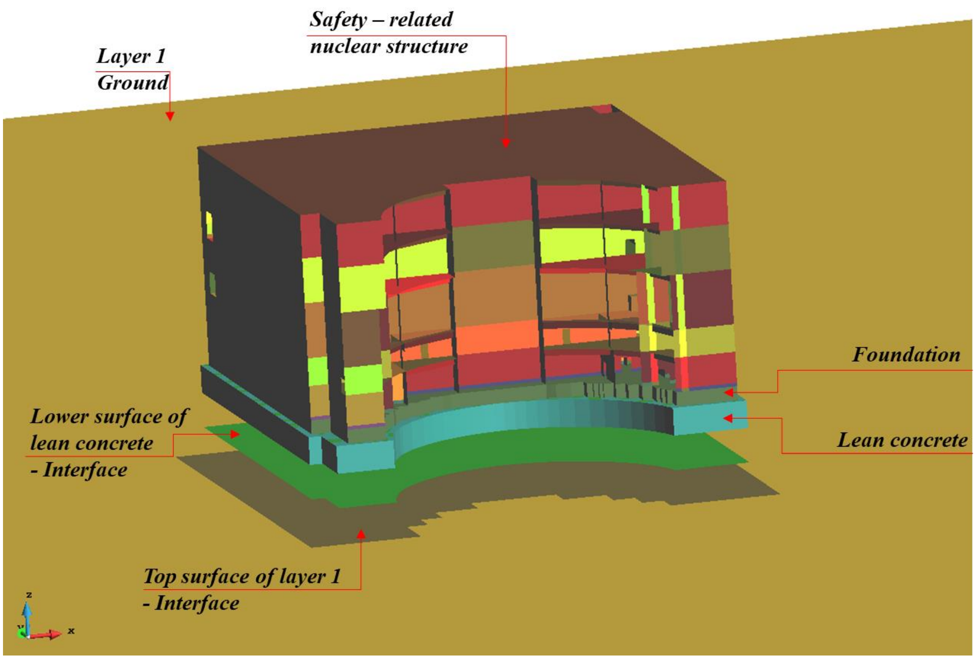

Figure 4. Interface location scheme for soil-structure contact.

The structure model is discretized with 324,150 3-node shell elements. In addition, 59,796 8-node linear hexahedral elements are employed to discretize the foundation and the lean concrete layer. The minimum side of the shell elements is 34 cm in order to reduce the computational cost by maximizing the time step of the explicit integration procedure, without losing resolution in the response. A critical time step of $0.23 \cdot 10^{-3}$ seconds has been set.

The soil model is discretized with 1.43M linear hexahedral elements, considering that the vertical size of soil elements must be conveniently chosen in order to properly reproduce the dynamic response, especially the high frequencies, guaranteeing that the propagation of waves is capable of reproducing frequencies up to 100 Hz [16]. Consequently, the maximum vertical sizes of each layer of soil are established according to the shear wave velocity, requiring elements of 2.20 m for some layers and of 3.40 m for others.

The FEM kernel employed is PLCD[2], an in-house CIMNE[3] code with explicit scheme. This code can solve structural dynamic problems with several constitutive material formulations and finite element typologies. The capabilities and accuracy of the PLCD code for the structural analysis of nuclear facilities has been thoroughly validated using experimental data [15] and has been widely used in other similar works on nuclear buildings [18] and by analyzing composite structures [19]. In addition, the GID[4] software, also developed by CIMNE, is employed as pre- and post-processor.

4 STRUCTURAL ANALYSIS

The main objective of the present work is to calculate the in-structure response spectra (ISRS) of safety-related nuclear structures with special emphasis on considering the non-linearities coming from the material and from the soil-structure contact interface.

The numerical simulation is performed in two calculation phases. In the first phase, the gravitational loads are applied, such as the self-weight of the structure, permanent loads and 25% of use loads. In the second phase, the seismic action is applied to the bedrock of the model. The acceleration is calculated at each of the predefined control points, enabling the calculation of ISRS. These results are carried out in order to examine two different aspects:

- The first aspect is the effect of material non-linearity. With this aim, the fixed base case (FB) is compared with the sliding-contact case (SC).
- The second aspect is the effect of non-linearity coming from the sliding on the interface. For this, three contact scenarios are compared, namely: only structure with fixed base (FB), soil-structure with fixed contact (FC), and soil-structure with sliding contact (SC). ISRS are calculated for 5% damping.

The effect of the mechanical uncertainty of the soil column is also analyzed by comparing ISRS obtained for three soil columns: LB, BE, UB. In the comparisons, the response spectra obtained is the average of seven seismic actions. Each record-keeping point is located in the position of singular security-related equipment.

Summing up, the calculations are performed for 7 accelerograms generated for near field earthquakes, considering 3 scenarios for the soil-structure interface (FB, FC, SC) and 3 soil columns (LB, BE, UB). Thus, 49 numerical calculations are to be performed.

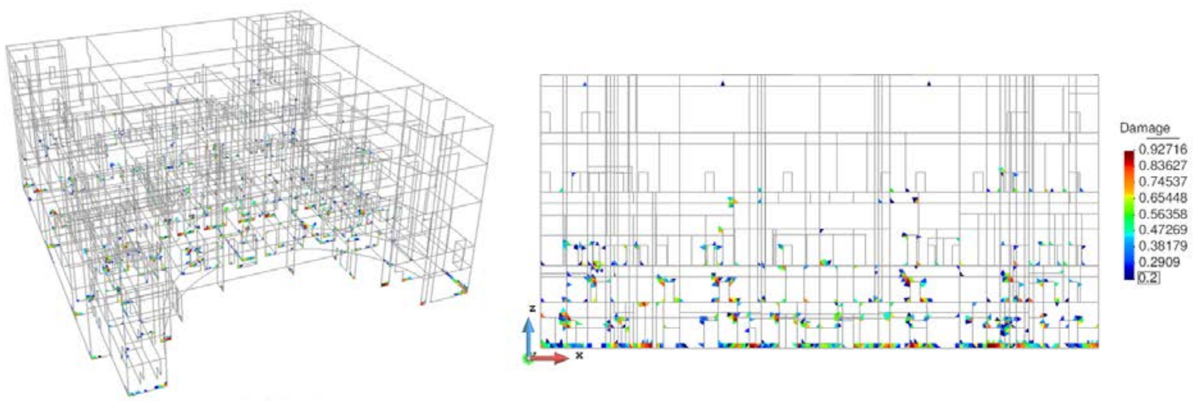

Figure 5. Structural damage for FB case. General 3D view (left), and cross-sectional view (right).

[2]PLCD home page: www.cimne.com/PLCd
[3]CIMNE - International center for numerical methods in engineering: www.cimne.com
[4]GID home page: www.gidsimulation.com

5 RESULTS

When a material point of the structure exceeds the elastic limit, the concrete begins to crack, what produces a decrease in stiffness and the possible plastification of the structural steel. The evolution of microcracks in concrete is described with a mechanical damage model where its main internal variable varies between 0 (no damage) and 1 (completely damaged). Figure 5 shows that the level of damage recorded is remarkably low in all cases, and appears in much localized areas due to local stress concentration effects.

Two extreme scenarios are compared: the fixed base case (FB) versus the sliding-contact case (SC). Results are generated for the BE soil profile, considered as representative. Figure 6 shows the effect of non-linearity on the structural response spectrum obtained for fixed base (FB) and sliding-contact (SC), at three elevations of the building {+91.00, +108.00 and +120.70}, for the linear elastic and non-linear material models, in Y direction.

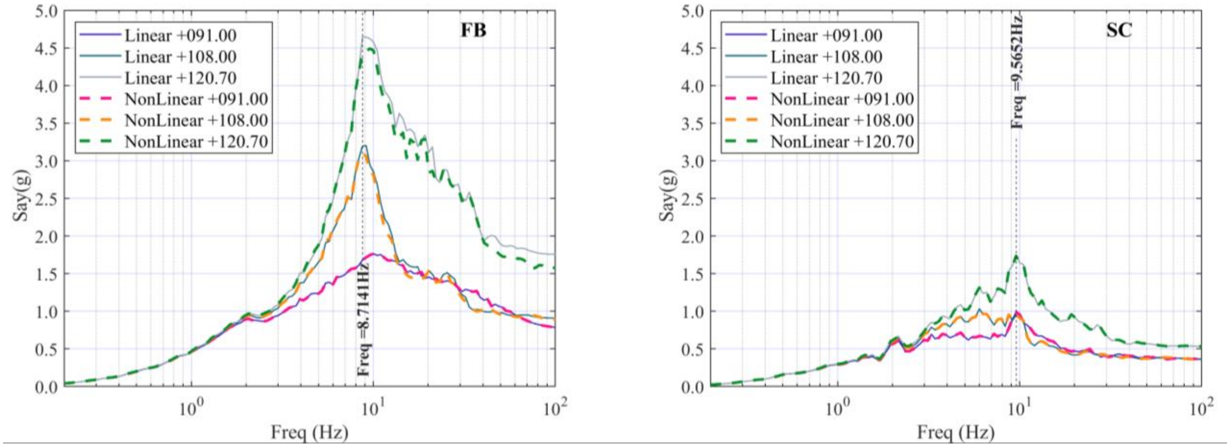

Figure 6. Response spectra in Y direction. FB case (left) and SC case (right).

The effects of the material non-linearity on the ISRS of the studied nuclear structure are insignificant due to its mechanical characteristics. However, the effects of the non-linearity in the soil-structure interface is crucial as it is shown in Figure 6. In the case of the sliding contact (SC), the spectral ordinates reduce drastically if compared with the fixed base case (FB).

One of the sources of uncertainty in the structural analysis of a structure is the underlying soil and its mechanical and dynamic properties. A statistical approach is considered in order to quantify the effect of uncertainty arising from the mechanical properties of the soil column.

Figure 7 shows the ISRS obtained in the structure at the foundation level (+91.00) and at the roof level (+120.7) for three columns {LB, BE, UB} corresponding to the percentiles 18%, 50% and 86% respectively, and two scenarios for the soil-structure interaction {FC, SC}.

The effect of soil uncertainty is noticed when comparing the PSA ratio with respect to the most representative case: the Best Estimate (BE) soil column. The resulting spectra show that the improvement of the mechanical properties of the column (UB) leads to higher ISRS curves and higher values of the PSA ratio with respect to the degraded soil (LB). Movements at interface (SC) significantly reduce the PSA to a remarkably low value. The sliding interface acts as an “isolator unit” for the structure with respect to the mechanical properties of the soil, which highlights the relevance of soil-structure interface nonlinearity in the analysis. The ISRS in the horizontal direction (X, Y) show a very similar behavior while the ISRS in the vertical direction present some peculiarities. The frequency range is extended with respect to the horizontal direction spectra in all cases, and the peak frequency (f^{psa}) reaches higher values when the mechanical properties of the soil improve. This fact becomes more evident in the case with fixed-contact (FC) and it softens in the case of sliding-contact (SC).

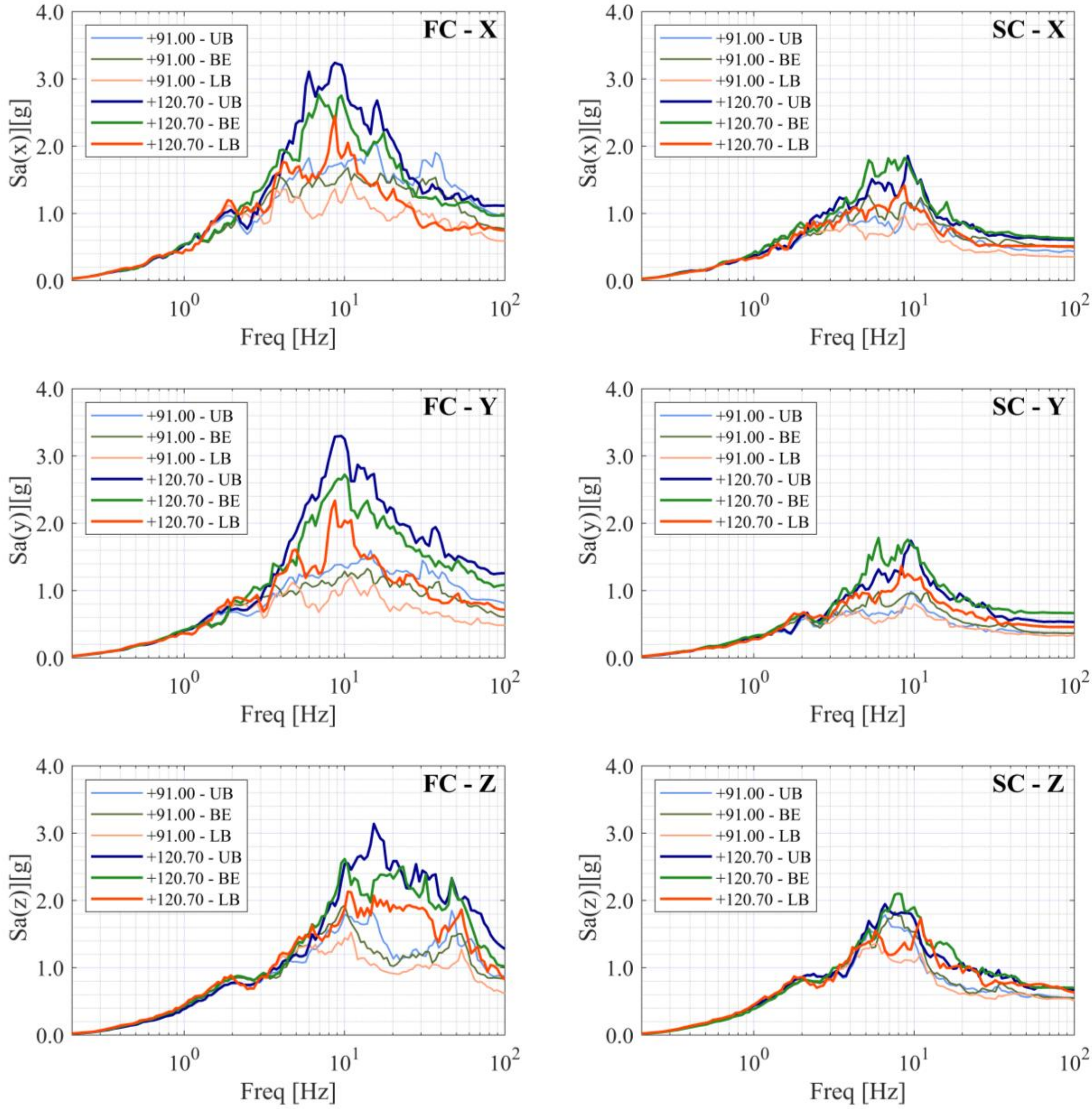

Figure 7. In-structure response spectra in X, Y, Z directions for the three soil columns (UB, BE, LB), and two soil-structure interface scenarios: FC fixed-contact (above). SC sliding-contact (below).

The effects of the non-linearity of the soil-structure interface is analyzed by observing the obtained response spectra. This evaluation is carried out through the ISRS of the movement measured at several floors: at the level of the ground surface (FF), at the foundation level (+91.00), at the level of the third floor in the structure (+108.00) and at the roof level (+120.70), for three soil-structure interface scenarios and three directions {X, Y, Z}.

The analysis of the ISRS allows extracting the maximum spectral acceleration (PSA) and the corresponding maximum frequency (f^{psa}). Figure 8 shows an ISRS matrix for soil-structure interaction (horizontal X, Y) and top-bottom direction (vertical, Z) cases. Table 2 summarizes the main results of the ISRS obtained, to allow the comparison in terms of peak spectral acceleration (PSA) and peak frequency (f^{psa}).

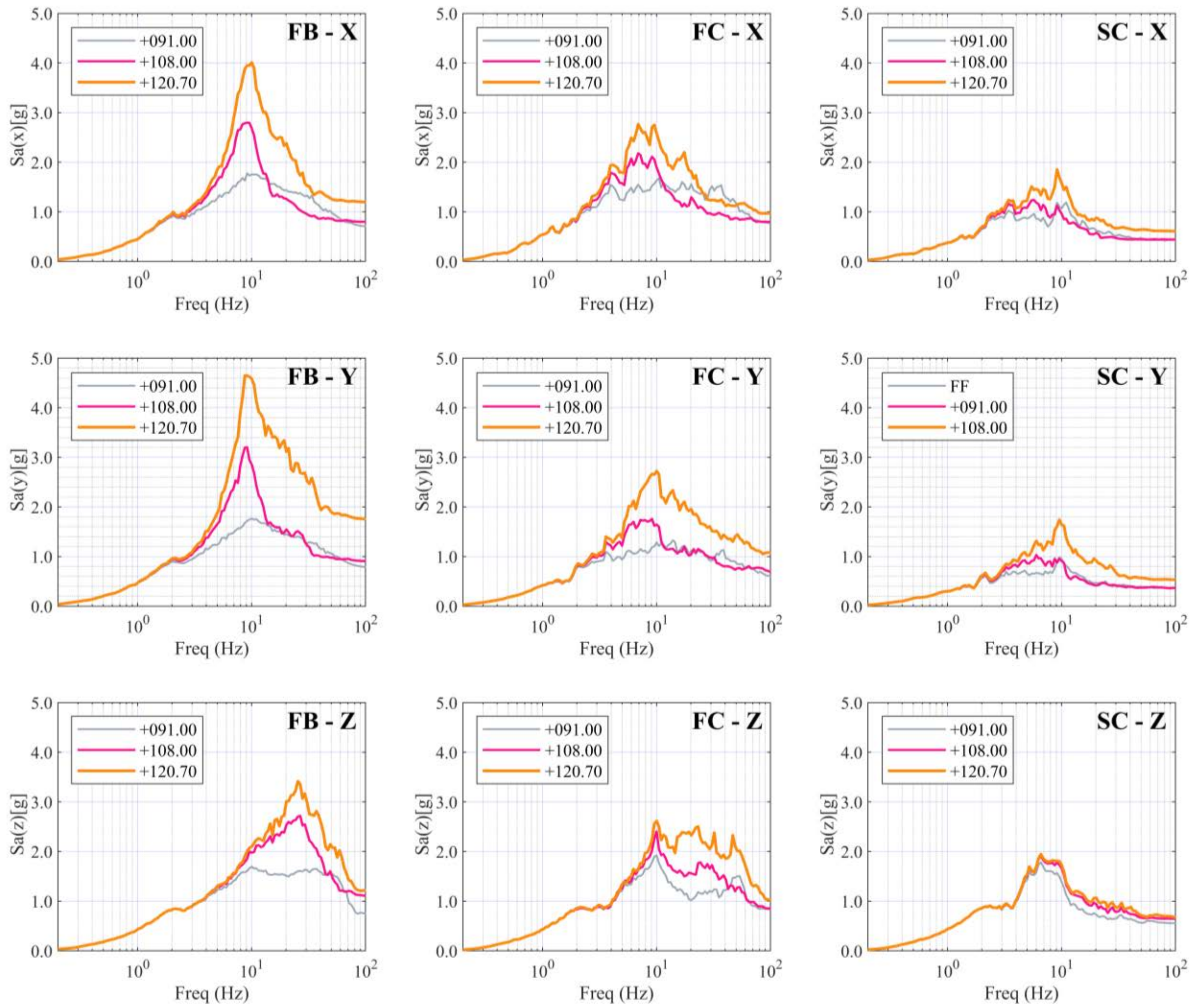

Figure 8. In-structure response spectra for three scenarios: FB (left), FC (center), SC (right).

Dir	FB				FC				SC			
	Base		Roof		Base		Roof		Base		Roof	
	Freq (Hz)	PSA (g)	Freq (Hz)	PSA (g)	Freq (Hz)	PSA (g)	Freq (Hz)	PSA (g)	Freq (Hz)	PSA (g)	Freq (Hz)	PSA (g)
X	9.13	1.78	10.02	x	10.50	1.68	6.90	2.77	11.00	1.19	9.13	1.85
Y	10.02	1.76	8.71	4.65	13.89	1.33	10.02	2.72	9.57	0.99	9.57	1.74
Z	10.02	1.69	25.45	3.41	10.02	1.93	10.02	2.62	6.59	1.78	6.59	1.95

Table 2. Summary of the ISRS features for three contact scenarios.

6 CONCLUSIONS

This work studies the dynamic structural response of an existing safety-related nuclear structure subjected to near-field seismic actions. It evaluates how floor spectra is influenced by including in the analysis the non-linearities coming from the material and the soil-structure interface.

The methodology to determine the seismic action applied to the structure is based on a set of representative seed signals obtained from public databases. Using seismic factors of the area and directionality parameters, a set of seven floors of ground movement in the foundation is defined. These signals are fitted and validated against the target spectra and translated, through a deconvolution process, to the bedrock level where they are applied to the entire model.

The structural scheme is defined with a high degree of realism to capture the structural behavior. The structural typologies involved are specifically modeled: shear walls, slabs, beams and columns. Likewise, the model considers the irregularities existing in the structure. The foundations and the underlying layers of lean concrete are also considered in detail.

In the FEM soil model, specific boundary conditions are carefully considered, such as cyclic symmetry at the edges of the soil domain and the interaction of the structure with dynamic forces due to the surrounding ground and adjacent buildings.

The results show how the non-linearity of the material barely affects the global response of the structure and floor spectra. However, the non-linearity due to sliding at the soil-structure interface significantly affects the response spectra of the structure, reducing the maximum spectral acceleration peaks. Additionally, the ISRS registered at each equipment location would allow a subsequent analysis to assess the integrity of critical equipment in case of a margin near-field earthquake.

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