Earthquake response analysis of reactor vessel internals for next generation reactors

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ABSTRACT

The intent of this paper is the evaluation of the results of an earthquake response analysis method application to a next generation nuclear power plant system with reference, as an example, to an innovative reactor (e.g. IRIS) International Reactor Innovative and Secure conceptual design with a generic implementation procedure.

As it is well known adjacent buildings influence each other through the soil and other interconnecting structures during strong earthquake events, exhibiting different dynamic behaviours from the separate ones characterising the system. A seismic preliminary analysis was performed to evaluate the dynamic load propagation from Ground through the Containment System and Vessel to the Steam Generator's tubes, taking into account also the soil-structure interaction in presence of various soil characteristics.

The input seismic acceleration Time History (TH), used for the analyses was generated, determined and verified to be in agreement with suitable safety margin, with the ground design basis response spectra, as indicated in relevant NRC rules for NPP's (NRC 1.60). To the mentioned purpose numerical models of each of the main system components were set up implemented on the MARC FEM code and analysed using as input the specific time histories previously determined by a simplified model of the whole plant. The results were combined using the modal substructure procedures for studying the detailed dynamic behaviour of nuclear reactor under seismic excitation. This is a procedure widely used in the preliminary design of complex systems and, in general, provides conservative results. The results of the performed analyses with different numerical approaches, the possible effects of Soil-Structures Interactions (SSI) and the response of internal components (e.g. Vessel-SGs and SGs-tubes system) to the ground motions seem to confirm the possibility to achieve data useful for the optimization of geometry and performances of the proposed structural solutions for the considered NPP.

INTRODUCTION

In NPP structures the design of foundations is governed by the sub-soil conditions available at the specific location of the structure. The philosophy of the structure and foundation substructure type choices is influenced to a great extent by the size of the foundation soil and by the mechanical and rheological properties of the ground.

The aim of the present analysis is to demonstrate the site and foundation dependency on the response to a severe seismic accident in the case of a nuclear power plant conceptual design. The seismic analysis coupled with the soil-structure interaction is required as one of the conditions for the design and construction approval of Nuclear Power Plants (NPPs) structures and is also necessary to evaluate their real capacity of dynamic loads bearing. The great variability of the foundation soils lead to the assumption of many mechanical models which idealize the ground as a supporting medium [1]. The considerable influence of the structural rigidity has been qualitatively explained in the literature long back [2]. Subsequently, several studies have been conducted to estimate the effect of this factor [3]. In recent years, attention is focused on the conservatism in the seismic analyses and design methods previously used for the design of the operating nuclear power plants. It is well known that analyses of building structures were based on conservative procedures and assumptions and that therefore the structures might possess sufficient capacity reserves to easily withstand higher seismic loadings [4].

The consideration of soil-structure interaction is of main importance for the evaluation floor response spectra for stiff, heavy buildings (e.g. nuclear power plants), because the structural response can be modified significantly by the interaction effects. A rational analysis of structure and its foundation requires that the interaction between the superstructure, the foundation and the supporting soil medium be accounted for by treating them as part of a single continuous system.

DESCRIPTION OF THE CONSIDERED STRUCTURAL SYSTEM

The first task in structural modelling is to develop fixed-base models of each structure in as much detail as necessary to define adequately the building response at all desired locations while limiting to a reasonable extend the calculation difficulties. In principle the potential sites for construction of the a standard nuclear power plant are selected by choosing appropriate total thickness of soil overlying bedrock and having an acceptable embedment depth of the Nuclear Island.

Moreover the Standard design of a LWR takes into account to an envelope of site conditions, which would be suitable for construction on any given site without performing the site specific analysis and design.
In the case of IRIS NPP, the integral vessel RPV is larger than a traditional PWR pressure vessel, its RV houses not only the nuclear core, but also all the major reactor coolant system components including pumps, pressurizer, control rod mechanisms, eight modular helical coil steam generators, and a steel reflector around the core internal shielding [5]. In this NPP the buildings that characterize the Nuclear Island (NI) may be subdivided into three meaning structures:

- Auxiliary building including external containment (EB);
- Inner containment structures (CS);
- Containment internals including reactor pressure vessel (RPV).

The complete first simplified mathematical model and the degree of discretization were chosen such that the natural behaviour of the structure in the relevant frequency range could be computed with good reliability. Modelling of EB, CS and RPV internal structures required the setting up of appropriate meshes assembled with suitable elements (as e.g. 3-D solid brick and/or shell type elements, available in the used finite element modelling code (FEM)) to represent the behaviour of each structure [6].
To reasonably limit the analyses calculation time the masses of the RPV and the CS internal structures have been considered as lumped ones distributed at appropriate chosen locations within the EB. The criterion for choosing the mass point locations was to provide for accurate representation of the dynamically significant modes of vibration for each of the internal components. The analyses carried out so far were performed with some simplified assumptions. In the considered reactor type External Reactor Building surrounds the containment system. The structure is assumed to be based on rigid foundation at the base, which joints the inner structures to the top soil deposit, and is characterised by a free top roof plate. The available preliminary structural drawing, summarized in Fig. 1, shows that the foundations are located on the ground surface and are assumed closely bed to the soil.

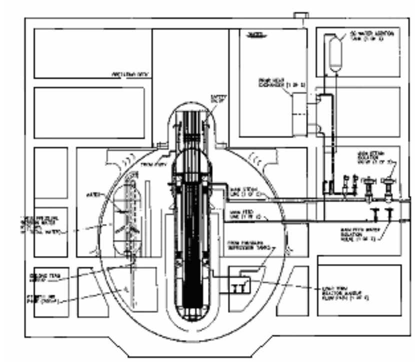


Figure 1 – Scheme of the Reactor Building

METHOD OF ANALYSIS

In general NPP are characterised by structural elements with different stiffness properties, resulting in complex mixed structures. For this reason reasonably accurate and detailed models are necessary to ensure adequate treatment of interaction effects such as Structure-Structure Interaction as well as Soil-Structure Interaction, and so on.
The term Soil-Structure Interaction has been widely used to indicate the mechanics of interaction between the soil and the structure or its part buried in it. In this paper the embedment effect on the SSI characteristics was considered, because it is known that the seismic response of embedded structures is greatly affected by the SSI phenomena. The backfill and surrounding soil resulting from the building embedment makes the phenomena more complicated. Most of the foundations of various buildings fall in the category of rigid foundations. The resistance offered by the side rock is passive in nature. Under the action of an overturning moment when the rock starts loosing contact with the substrata below, it will lift up and slide past the side rock surface. Under this situation, the passive resistance of the side rock will play its role. The concrete structures in a nuclear plant have many thick concrete walls as earthquake resisting walls and as radiation shielding walls. In this paper the problem of determining the transfer function of seismic load is treated with an analytical as well as numerical approach analyses with reference to a simplified configuration.

ANALYTICAL APPROACH

To the purpose of this study a simplified system constituted with three mutually interacting components was considered:
(a) The superstructure including the external containment building (EB);
(b) The foundation;

(c) The substructure referring to the soil matrix.

The complete system subjected to an earthquake loading condition may modify both the axial forces and moments in the structural members among various parts of the structure.

The amount of loads redistribution depends upon the stiffness of the structure as well as of the load-settlement soil characteristics. The substructure receives and transfers the loads of the superstructure to the ground; so its stress as well as the strain was function of the characteristics of whole system.

The difference between the input and transferred motions will be governed by the energy absorbing capacity of structure including the foundation element and also the reflection pattern from the buried portion of the structure.

In general the soil structural interaction is simplified to the analysis of the two main components: the foundation and the substructure, so the superstructure was analysed apart from the mentioned analysis. A simple way to treat the SSI analysis for a seismic excitation in the time domain (in the sub structuring formulation) is to model the flexibility of the soil around the foundation by using impedance function. The considered ideal soil-structure system is characterized by a circular rigid foundation resting on the surface. The footing soil is a half-space medium which is elastic, homogeneous and isotropic, as shown in Fig. 2.

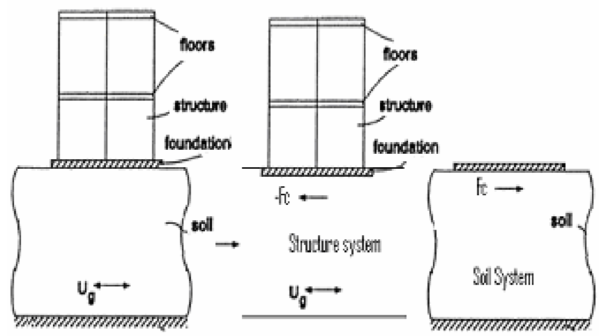


Figure 2- Soil-Structure System and Substructures Model

The procedure for solution of coupled problem SSI is performed in the frequency domain and this one makes it difficult to incorporate SSI in routine dynamic analysis. The method used to take into account SSI is formulated in accordance with the substructure method which is explained in the time domain analysis, through the separation of soil-structure interaction system as indicated in the previously Figure 2. The resulting displacement vector {u} is represented as follow:

$$\{u\} = \{u^*\} + \{\Delta u\} \tag{1}$$

Where {u*} is the displacement of soil, while {Δu} is the displacement due to the excitation force vector. The interaction vector force F_c could be also related to the displacement vector through the relationship:

$$\{F_C\} = [K_C]\ \{\Delta u_C\} \tag{2}$$

where Kc is the impedance matrix, whose properties obviously depend on the frequency as well as the foundation geometry and the characteristics of soil media. The impedance function is the ratio of a harmonic force applied to the foundation to the resulting harmonic displacement at the bottom of foundation.

Moreover it could be considered also as a ratio of two complex polynomials, which transferred functions of discrete-time recursive filters by taking the Fourier transform [7-8]. The general form of foundation impedance functions, K(ω), is:

$$K(\omega) = K_0\,[\,K_1(\omega) + I\,K_2(\omega)\,] \qquad \text{With} \qquad K_0 = \frac{8Gr}{2-\nu} \tag{3}$$

where K_0 was the static stiffness for the horizontal movement and K_1 and K_2 were the real and imaginary portion of impedance function representing the stiffness and the damping or dissipation energy of soil. Assuming harmonic characteristics, by simple manipulation, the above impedance function may be approximate, in low frequency range, as follow:

$$K_C(\omega) \approx a_0 + i\omega a_1 - \omega^2 a_2 \quad (4)$$

$$F(t) = \left(K_s + i\omega C_D - \omega^2 M\right)u(t) \quad (5)$$

where ω is the frequency and i is the imaginary unit. The unknown factor a_0, a_1, a_2 (Eq.6) were determined by equating them (Eq.7) to the constants of an equivalent single degree of freedom (SDOF) system. Moreover K_S, C_D and M constants are obtained by fitting the frequency characteristics of the impedance function at a reference natural frequency value ω_0, as indicated in literature [9].
The effect of settlements on the structure is usually ignored even then when differential settlements are important. A proper mathematical model of the system is extremely difficult to be made because of the large number of uncertainties regarding the behaviour of soil as well as the structure under stress; hence a numerical modelling is necessary to get an adequate SSI system where the structures and their surrounding soil are treated as a total or separate system.

NUMERICAL APPROACH

In order to ensure adequate treatment of interaction effects, all the main structures were therefore idealized and modelled in only one 3D finite element model, and subsequently the seismic analysis was carried out by means of the substructures approach. For this reason reasonably accurate and detailed models are necessary. Modelling of structures was sufficiently detailed and contained an adequate number of degree of freedom to obtain the dynamic response at the locations of the main safety-related components and to evaluate the corresponding response spectra as well as the nodal forces and element stresses in the regions of interest.

Substructures model approach

As before mentioned to simplify the analyses calculation time the masses of the RPV and the CS internal structures have been considered as lumped ones distributed at appropriate chosen locations within the EB; the analysis was based on the substructure models approach with appropriately separated models for the building structures as well as frequency-independent soil that allows to separate the NPP seismic analysis problem into a series of simpler ones that can be solved each independently. This methodological approach has been also validated in considering the influence of the mesh sized [10]. The response spectra for vertical and horizontal motions however are normally indicated in codes for several damping coefficients. In Regulatory Guide 1.60 (US Atomic Energy Commission, 1973), the given spectra are for 0.5, 2, 5, 7 and 10% damping. The Time History approach was used in all analyses to evaluate the effects of a Safety Shutdown Earthquake (SSE). The acceleration data were derived from the original horizontal response spectra where the maximum Peak Ground Acceleration (PGA) was equal to 0.2 g, only the horizontal translation direction and for an excitation duration equal to 30s, calculated for an appropriate damping, in according to the previous mentioned Regulatory Guide. .
The response of the structure was obtained for selected time steps of the input earthquake time history. The seismic analysis was performed assuming a proportional damping for each structure, in according with the Equivalent Rayleigh damping. The implemented values are in agreement with the NRC Regulatory Guide 1.61 (USNRC, 1973) that gives acceptable damping value to be used in the seismic analysis for operating (OBE) and shut down (SSE) earthquakes.

System modelling

In order to analyze the dynamic behaviour properly and in view of the geometrical configuration of the reactor building internal structures an appropriate and detailed model was necessary. The adopted simplification is generally necessary for the intrinsic complexity of treating SSI as a whole, as each of the sub-systems, by itself, represents a vast field of possible mechanical idealizations and a wide choice of physical and geometrical parameters. Soil is basically composed of particulate materials, whose non-linear characteristics, when the structure is deformed under seismic excitation, could change the stiffness of the structure and its fundamental vibration period.
An approximate 3D dynamic SSI model based on the substructure method is developed to analyse the seismic response of structures accounting the mass, stiffness and damping matrices and to solve the equations of dynamic equilibrium at each point (Eigenvalue Analysis) and time step (Transient analysis). In general the Containment System, like the external nuclear building, is considered to be fixed at the base and free at the top. The set up model describes, with the mentioned approach of substructures, the influence of rigid foundation of the Nuclear Island in a homogeneous elastic soil over a deformable soil, subjected to a seismic excitation. A rigorous 3-D analysis was required to study the edge effects for different embedment depth. To the purpose the degree of discretization were chosen such that the natural behaviour of the structure in the relevant frequency range could be computed with good reliability.
The Superstructures was represented by the Nuclear Island (NI). Its central part surrounds the inner nuclear power plant structures. It is founded on rigid foundation that joins the bottom of hemispherical concrete CS to the soil. Buildings are

primarily made of important shear wall systems supported by strip footings and both contain structures peripheral to the major portions. These peripheral areas would have a secondary effect on the reactor response and therefore were not considered in this paper. The Substructure is the foundation plan.

The available structural drawing, in the above Figure 1, showed that the foundations of the main NI structures, located on the ground surface, are connected to each other, either directly or through the structures to a varying extent. The Soil was modelled as a homogeneous loose sand zone. The thin layer of soil and its material properties influence in different ways the horizontal and vertical propagation wave and the rocking vibration effects [11-12]. The hypotheses assumed to setup the model were:

- Non linearity of soil mass: this feature is accounted for assuming an elastic perfectly plastic material behaviour based on a yield surface that exhibits hydrostatic stress dependence. This characteristic is observed in a wide class of soil and rock-like materials that are generally classified as Mohr-Coulomb materials (generalized Von Mises materials).

To analyse the transfer function in term of response spectra the soil-structures interaction has been also considered and three different embedment foundations were analysed: 1)Superficial depth; 2) Intermediate depth; 3) Full depth.

The validity of these models is limited in practice as the soil medium is invariably inhomogeneous. Non homogeneity in the soil medium can be properly accounted for in the interactive analysis by using the finite element idealisation as well as the non-linearity that was dealt with appropriate elastic-perfectly plastic option.

NUMERICAL RESULTS

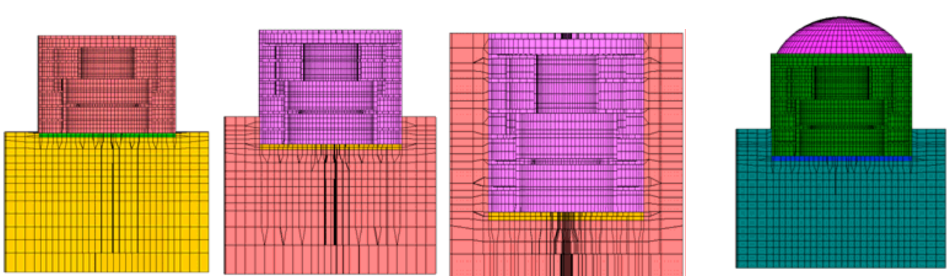
The non-linear characteristics of soil, when the structure is deformed under seismic excitation, could change the stiffness of the structure and could result in a change in its fundamental vibration period. The non-linearity in the model was dealt with an appropriate elastic-perfectly plastic option in MSC.MARC FEM Code. The real structural models of each building (lumped masses and stiffness) were then coupled with the previous mathematical models. The hypothetical Nuclear Island (NI) adopted in this study, is symmetrically located on the concrete foundation with the thickness of the soil layer being h. Also as indicated in the following figures, a wide layer of soil elements was added underneath the basement to account for the softening effect of the structure embedment. Geometrical characteristics of the structures were developed in according to the specifications carried out from the above drawings.

The EB is assumed to be a cylindrical reinforced concrete structure, with different thickness of walls and slabs up to the elevation 47 m. It is founded on a common rigid foundation with an average depth of about 2.0 m. The roof is firstly assumed as a cover plate, as derived from the previous mentioned drawing and successively was adopted to be as hemispherical. Overall in these very preliminary external building analyses the implemented dimension are as following:

1. External diameter De = 58 m
2. Building Height H = 47 m
3. Foundation thickness t = 2 m

The building foundation is assumed to be superficial on the top of the soil mat in the first case (Fig. 3 (a)). The depth was assumed at an intermediate depth in the second one (Fig. 3 (b)) and increased up to the external building full eight in the third one (Fig. 3 (c)). A calculated modal analysis results, not quoted in this paper, pointed out the excessive flexibility of top plate cover in respect for instance to the hemispherical one.

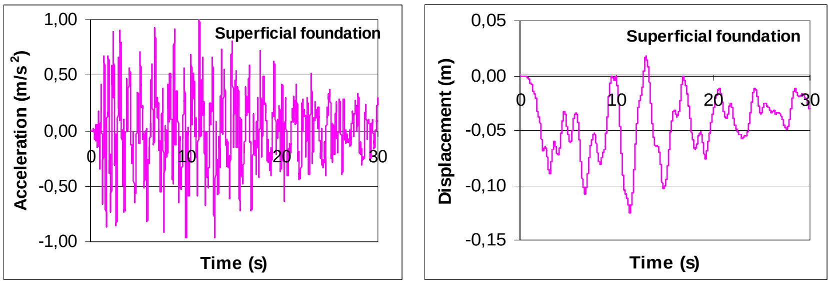
Therefore a new structure characterized by the same mentioned geometrical parameter and dimension and with hemispherical roof have has been set up. In Figure 3(d) is represented only the model with intermediate depth foundation. This last model was set up in the intent to highlight the propagation acceleration behaviour modification due to a more real different geometry if any. In order to obtain the amplification (or decreasing factor) representative of the time response for seismic excitations, the seismic motion (time domain) is applied at the bottom of structure transferred into the frequency domain. In the following graphs are represented further numerical acceleration and displacement results for the considered three different depth foundations.



(a) (b) (c) (d)

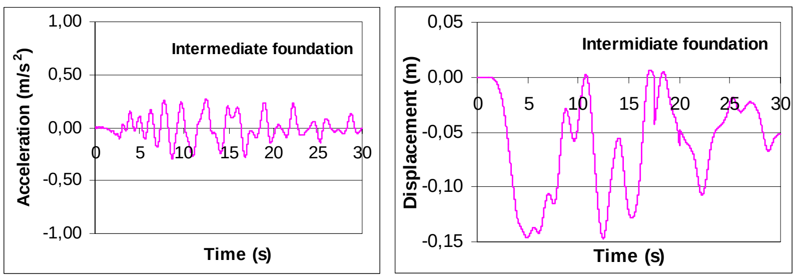
Figs.3 Superficial, Intermediate and Full depth for plate cover roof and (d) Intermediate hemispherical roof foundations

The response function in terms of acceleration and displacement, transferred from the soil trough the EB to the CS for the considered superficial, intermediate depth foundations (at Z = -13 m eight) and full depth foundation are represented respectively in Figures 4, 5 and 6.

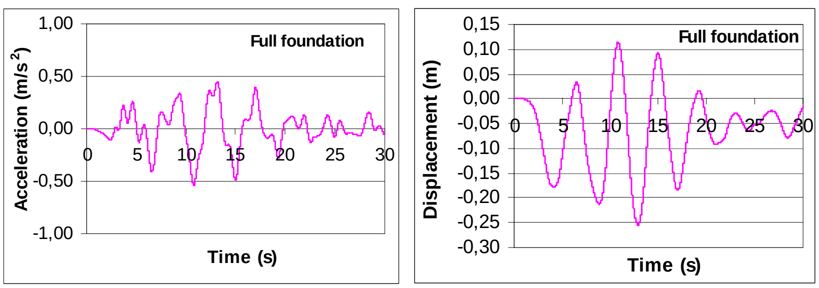


Figs. 4 Acceleration and displacement behaviour for superficial depth foundation

It was observed that intermediate and full depth foundation acceleration behaviours are quite different. This may be due to the assumed hypotheses and NI layout as well as soil geometrical characterization, which highlighted all soil-system interaction damping influence, as well as the structure-structure one, on the acceleration propagation.



Figs. 5 Acceleration and displacement behaviour for intermediate depth foundation



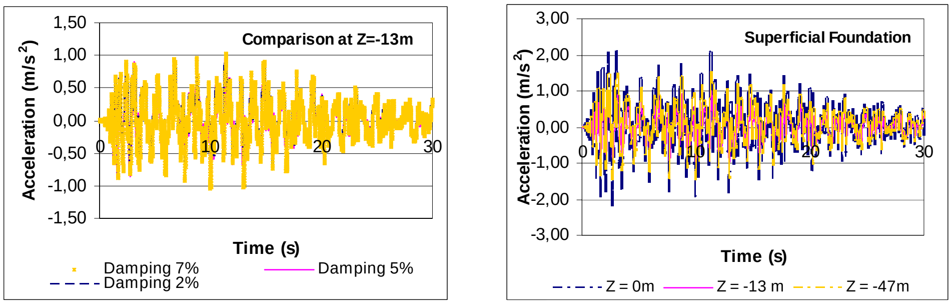
Figs. 6 Acceleration and displacement behaviour for full depth foundation

Moreover the ATH seems to be propagated through the soil to NI in different way depending by the different soil damping and by depth building foundation. The acceleration time history for the considered superficial depth foundations at Z = -13 m share for the mentioned case are represented respectively in Figs. 7 (a) and (b).

All the acceleration results may be used to determine the response spectra of the above mentioned substructure and evaluate the type of the seismic loading propagation.

The transfer functions in term of response spectra have shown that there is a relevant variation in term of acceleration in passing through the soil layer, for the considered case of superficial depth foundation and for a damping coefficient equal to 5%. In the case study, the site with superficial depth foundation was found to have higher peak acceleration than that of intermediate and full ones. Considerable change in spectral acceleration is observed from the response spectrum curve. Therefore the change in natural period may alter the seismic response of any structure considerably. In addition to this, soil medium imparts damping due to its inherent characteristics.

During the ground shaking caused by an earthquake, stress waves propagating through the soil excite the structure, which in turn reflects back part of the energy and thus the particle motion in the soil near the structure is modified.



(a) (b)

Figs. 7 Acceleration TH at Z= -13 m, with different soil damping (a) and for 5% soil damping at different eight (b) for superficial depth foundation

Therefore the ground motion at the base of structure and the “free field” are not the same, as highlighted in the mentioned figure 7. This phenomenon is known as “superficial amplification”. The carried out amplification factor at Z =0 meter is equal to 1.16 while at Z = -13 and -47 meters are respectively 0.51 and 0.81, if compared to the input acceleration. Moreover, the hypothetical Nuclear Building has local soil conditions which amplify oncoming wave acceleration by about 1.2 on the average. Corresponding amplification of the peak acceleration between the structures ground and roof levels was observed due also to the top plate building in-plane flexibility.

As previously mentioned in correspondence of intermediate (Fig. 8) and full depth foundations where the soil energy absorbing and damping capacity is more evident, no high variations in term of acceleration were observed. For intermediate and full depth foundation case a decreasing acceleration factor respectively equal to 3 and 2 was observed in respect to the superficial one.

Furthermore an overview of the acceleration and displacement time histories are depicted in the following figures 9 (a) and (b) for hemispherical roof as well as the determined response spectra comparison between plate and hemispherical roof. These graphical time histories highlighted the influence of the kinematic interaction and filtering effect of structures as well as soil, mainly when material is going into the nonlinear fields.

Moreover the calculated spectra comparison did not highlight relevant difference in term of the acceleration values corresponding to the two considered geometries, but a shifting towards higher frequency values in the case of hemispherical EB solution. Apart from structural integrity, the foundation should be able to transfer all the forces to the sub-soil in an effective manner. It should have sufficient resistance to sliding, overturning and floatation.

Further, the sub-soil should have sufficient strength to withstand the pressures exerted by the foundation without excessive deformations.

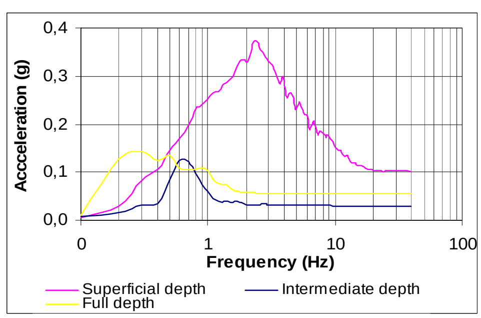
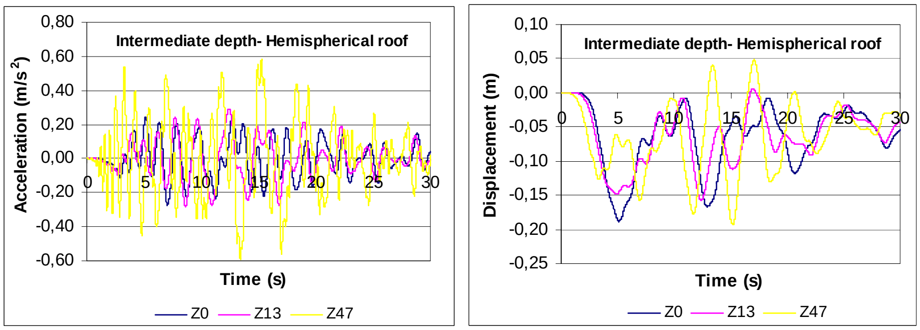


Fig.8 Response spectra comparison at Z= -13 m for different depth foundation

The pressure under a foundation is directly related to the applied loads and the size of foundation. The load transfer takes place through bearing action only at soil- foundation interface. The carried out accelerations could be used as input ones for further CS substructure analyses which are needed to determine the dynamic stress on internal relevant components.



Figs. 9 Acceleration and displacement behaviours for intermediate depth foundation and hemispherical roof

CONCLUSION

As a check of these very preliminary conclusions, it was observed that the outer nuclear building acceleration values were used to determine the response spectra, according to the Regulatory Guide 1.122 (USNRC, 1978).

Analysing the calculated response spectra it can be generally observed that the SSI and adjacent building interaction results in rather slight shift in the fundamental frequency of the building. They also depend on the dynamic properties of each structure such as strength, rigidity, and modal characteristics. Therefore, higher responses may be due, in the first analysed top cover plate model, to the excitation of the masses influenced by different embedment foundation geometry and EB and by the roof flexibility itself. Analysis and design of the NPP structures involve considerations not only on the available geometry but also on the capacity of the most important structural members that transfer the seismic inertial loads from their application points. The showed preliminary results were carried out with the Time History Method applied to the EB 3-D FEM model, in order to evaluate the seismic response transferred to the single element of the internal reactor vessel structures.

In general, the model analysis (considering the frequency dependency of the structural parameters) yielded accelerations values rather smaller than the ground seismic motion.

The differences are due to the influence of real damping characteristics, SSI, kinematic interaction effects by the complex mathematical model, filtering effects of soil and structures, etc.

On the base of all very preliminary analyses, the effects of the described alternative nuclear building in soil embedment have been considered in order to check the possible reference system criticalities and obtain a feed back on the critical design features (if any). On the basis of this study, it can be concluded that the results of the performed analyses make evident the effects of SSI on the response of internal components and highlight the criticality of the external building geometry.

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