16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 4209 (Abstract ID)

Registration Code: S-X1463153704

SOIL-STRUCTURE INTERACTION ANALYSIS FOR ATUCHA II NUCLEAR POWER PLANT USING A DIRECT METHOD

R. Rodari(1), F. Zabala(2), G. Navarta(3)

(1) Assistant Professor, Earthquake Engineering Research Institute, National University of San Juan, Argentina, rodari@unsj.edu.ar
(2) Director, Earthquake Engineering Research Institute, National University of San Juan, Argentina, fzabala@unsj.edu.ar
(3) Researcher, Earthquake Engineering Research Institute, National University of San Juan, Argentina, gnavarta@unsj.edu.ar

Abstract

This paper presents the procedure used to model the dynamic soil – foundation – structure interaction problem, in order to obtain the floor response spectra for the reactor building of the Atucha II nuclear power plant. The procedure is based on a set of time-domain dynamic analyses of finite element models of the building and the foundation soil. The finite element model of the reactor building, including the base mat, is composed by shell, solid and beam elements. A separate finite element model of the unexcavated soil is used to obtain the free-field input motion.

The soil-structure interaction model is a combination of the finite element model of the reactor building and the excavated soil model. The complete model is validated by reproducing the impulsive load test performed on the building in 1993. Then, the dynamic response of the complete model subjected to the free-field input motion is obtained. The general purpose finite element code SAP2000 is used in these analyses.

Keywords: soil-structure interaction, floor response spectra, Atucha II

1. Introduction

Atucha II is a PHWR (pressurized heavy water reactor) located close to the Paraná River in the province of Buenos Aires, Argentina. This is a low seismicity, stable continental region at the east of the country, located more than 800 km away from the seismic areas of Argentina. The Earthquake Research Institute of the National University of San Juan has recently reassessed the seismic hazard at the site [1].

The procedure used to model the dynamic soil – structure interaction in order to obtain the floor response spectra for the reactor building is based on a series of time-domain dynamic analyses of finite element models of the building and the foundation soil. The soil was modeled using a linear equivalent model and the main building structure behavior was considered linear.

A Uniform Hazard Spectrum for the site was developed in [1] for a return period of 10,000 years. This spectrum corresponds to the SL-2 earthquake of the International Atomic Energy Agency (IAEA). The design motions postulated in [1] correspond to two scenarios: an earthquake of magnitude 5 located 20 km from the site, and an earthquake of magnitude 6 at 50 km. Response spectra corresponding to these scenarios are shown in Fig. 1. For each of these scenarios, reference [1] proposed two design ground motion records which were selected from records available at the PEER Strong Ground Motion Database. These ground acceleration time histories, whose spectra are shown in Fig. 2, were selected as input for the calculation of the floor spectra. The control point for these motions is the free field at the ground surface level.

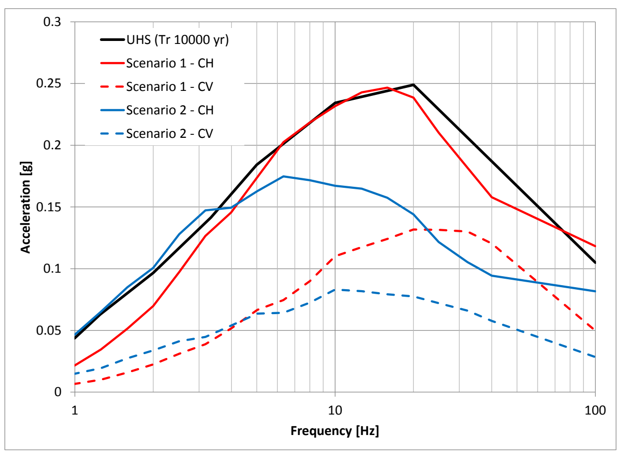

Fig. 1 – Uniform Hazard Spectrum and response spectra for scenarios 1 and 2 [1].

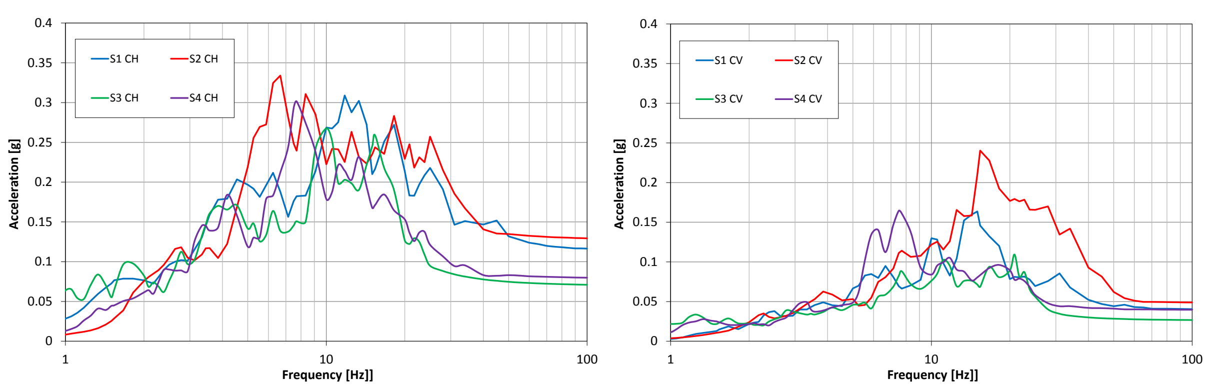

Fig. 2 – Response spectra (5% damping) of the design ground motions

(CH: horizontal component, CV: vertical component).

2. Soil profile and parameters

The soil profile of the upper terrace at the right bank of the Paraná de las Palmas river near Atucha II has been described in reference [2]. The profile consists of a silt and clay deposit about 55 meters thick. This deposit, known as Pampeano formation, is overconsolidated by desiccation. The Puelches dense sand deposit lies below the Pampeano formation. At the lower terrace the Pampeano formation is thinner, and a deposit of normally consolidated soft clay, known as Post-Pampeano formation, overlies it. Fig. 3 shows the overall profile found during the geotechnical investigation of the site.

Small strain shear moduli were estimated using available information of three previous works [3, 4, 5]. The profiles of small strain shear moduli are shown in Fig. 4, together with the profile adopted for this work. The shear moduli were estimated up to a depth equal to 60 meters, selected as the lower limit of the soil finite element model. No information is available to define the parameters of the soil constitutive model at greater depths. The modulus reduction and soil damping curves used in this work are taken from [6].

Site response analyses were performed using program EERA [7] for the horizontal component of each of the four selected design ground motions, which were considered acting in free field at ground surface level. The resulting strain compatible shear modulus and damping profiles are shown in Fig. 4.

Table 1 – Equivalent linear model soil parameters

Depth [m]	r [t/m³]	G [kN/m²]	E [kN/m²]	Vs [m/s]	Vp [m/s]
0 to 15	1.9	87444	260547	215	1500
15 to 30	1.9	186219	550650	313	1500
30 to 45	1.9	325105	951193	414	1500
45 to 60	1.9	456000	1321032	490	1500

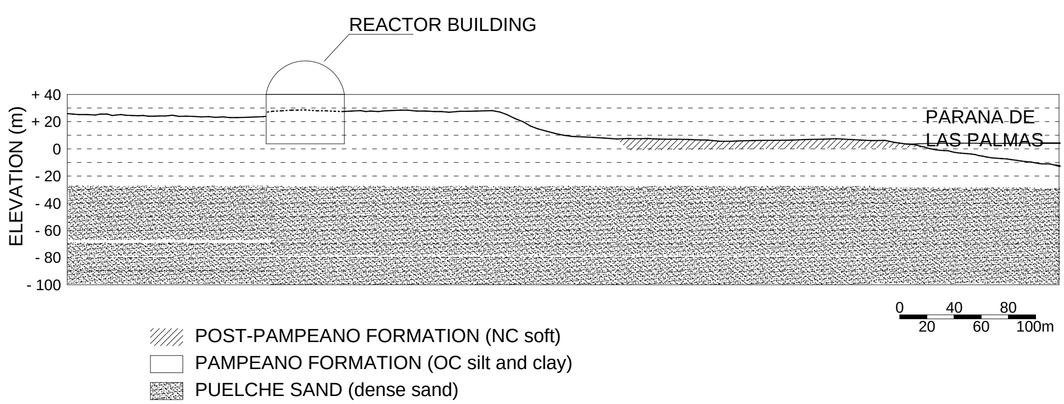

Fig. 3 – Soil profile (adapted from [2]).

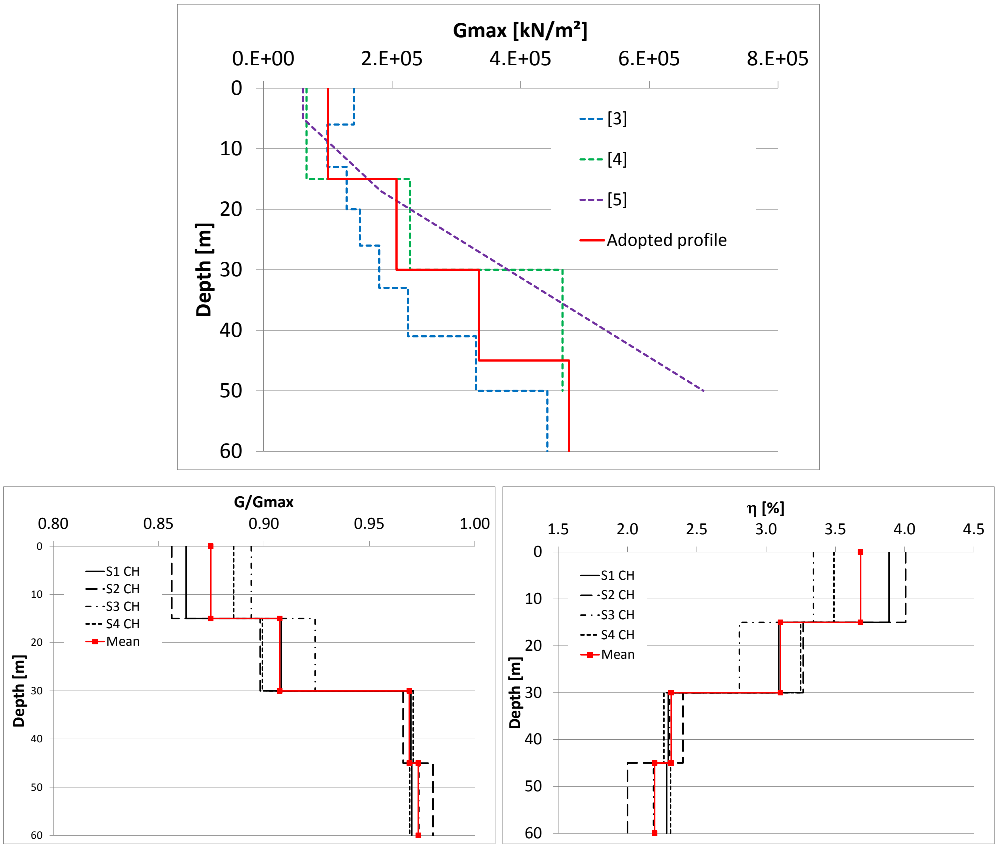

Fig. 4 – Profiles of small strain shear moduli (Gmax), strain compatible shear moduli and soil material damping.

3. Structural model

The finite element model of the reactor building is shown in Fig. 6. The structural model, including the base slab, is composed of 21744 shell, solid and beam elements. The total mass of the structural model is 131880 metric tons. The fundamental frequency for the fixed base condition is found to be equal to 5.8 Hz, which is close to a "fixed base" frequency identified during the impulsive test performed in 1993 [8]. The corresponding modal shape is shown in Fig. 5.

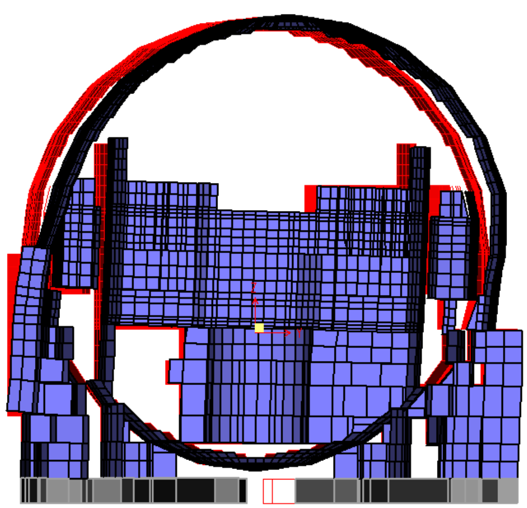

Fig. 5 – Modal shape corresponding to 5.8 Hz frequency, fixed base condition.

4. Foundation soil model

A tridimensional finite element elastic model of the unexcavated soil foundation, free of additional structures, was developed in order to obtain the free field motion. The model shown in Fig. 6 comprises an extension of 560 meters wide and 60 meters deep and was built using 19,680 solid elements of 6 and 8 nodes. All the analyses presented in this paper have been performed by direct integration of the equations of motion using the commercial software SAP2000 [9].

The assumed wave propagation mechanism of the free-field motion is vertically propagating SH-waves (producing horizontal motions) and vertically propagating P-waves (producing vertical motions). With these assumptions it is possible to generate an artificial horizontal boundary at the base of the model, replacing the ground below this boundary with a set of viscous dampers and time varying nodal forces. The forces time history is chosen so as the accelerations at the surface of the model match the free-field control motion. The parameters of the dashpots arranged on each of the two horizontal directions are proportional to the transmission speed of the S wave (Vs in Table 1), while the parameters of the vertical dampers are proportional to the transmission speed of the P wave (Vp in Table 1). A detailed description of this procedure can be found in [10], [11] and [12]. A Kuhlemeyer-Lysmer viscous boundary condition was imposed at the vertical boundaries of the model [13], in order to minimize spurious reflections of mechanical waves at the artificial boundaries of the finite element model. Owing to software limitations, the soil material damping was modeled by a Rayleigh type viscous damping, with mass-proportional and stiffness-proportional coefficients chosen so that the damping factor does not exceed 4% within the frequency range from 0.3 Hz to 21 Hz, and reaches a minimum of 1% at about 1Hz. Although this is a conservative approach, the overall effect is not important, since the equivalent damping of the whole system is controlled by the energy radiation rather than by the material damping.

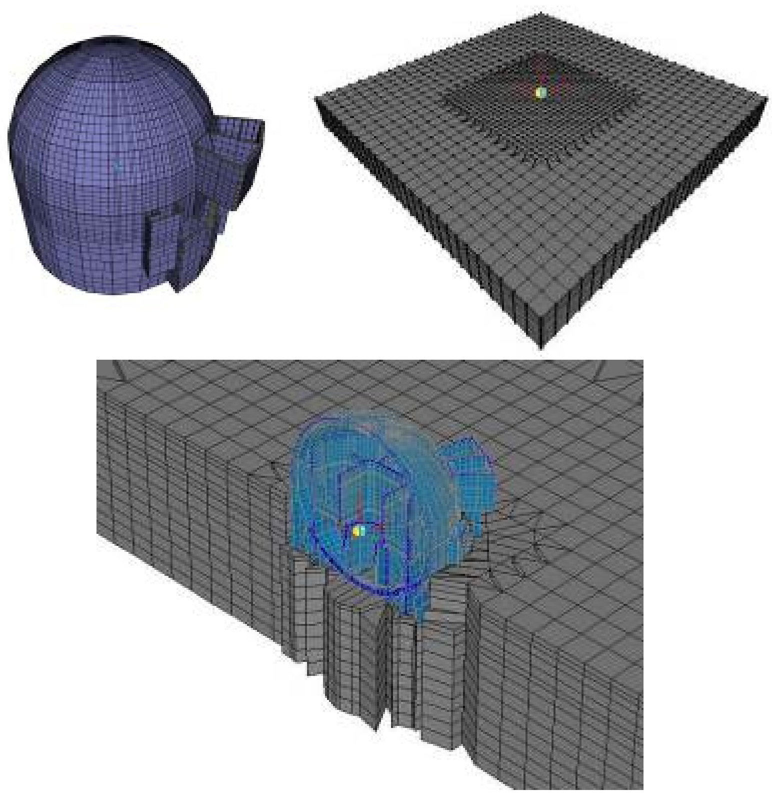

Fig. 6 – Finite element models of the structure, the foundation soil and the SSI system.

The procedure used to obtain the time history of nodal forces, which applied at the base of the foundation soil model produces the control motion accelerations at the surface level, is as follows:

1. Obtain response surface accelerations $\ddot{u}_0(t)$ for an initial arbitrary time history of forces $f_0(t)$ applied at the base model. The initial force time history is chosen so as to have an adequate power spectrum in the problem frequency range of interest.

2. Calculate the Fourier transforms of the surface acceleration $\ddot{U}_0(\omega)=\mathcal{F}(\ddot{u}_0(t))$ and of the forces $F_0(\omega)=\mathcal{F}(f_0(t))$.

3. Calculate the transfer function $H(\omega)=\ddot{U}_0(\omega)/F_0(\omega)$. This is performed for each direction (X, Y and Z). The resulting transfer functions are plotted in Fig. 7.

4. Calculate the Fourier transform $\ddot{U}(\omega)$ of the control motion acceleration time history $\ddot{u}(t)$.

5. Calculate the Fourier transform of the force time history which produces at surface level an acceleration time history equal to $\ddot{u}(t)$, $F(\omega)=\ddot{U}(\omega)/H(\omega)$.

6. Obtain the force time history by inverse Fourier transform: $f(t)=\mathcal{F}^{-1}(F(\omega))$.

7. Obtain the response surface acceleration time history $\ddot{u}(t)$ produced by the force time history $f(t)$ applied at the base of the model, and verify that $\ddot{u}(t)$ matches the control motion acceleration time history.

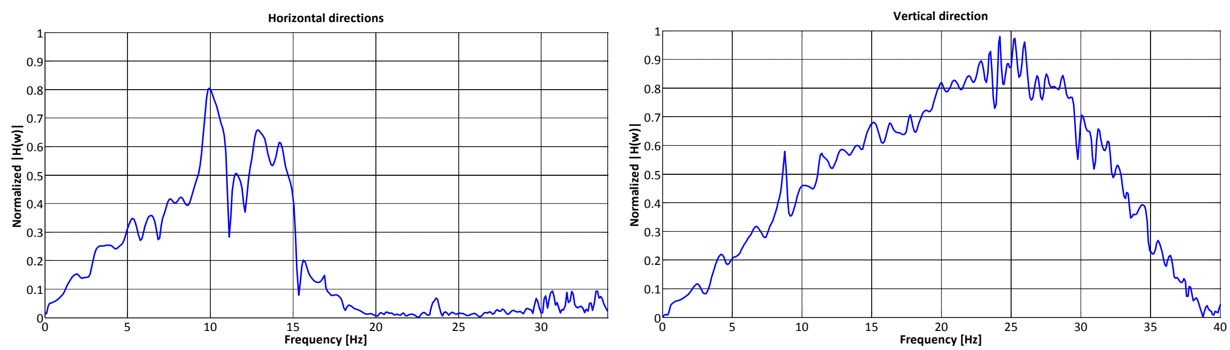

Fig. 7 – Transfer functions between the forces applied at the bottom boundary of the foundation soil model and the response accelerations at surface level.

The response spectra obtained at surface level area compared to the spectra of one of the control motions (S1) in Fig. 8.

The force time history was input to the full SSI model described below, in order to obtain the dynamic response of the model. The procedure described here does not allow to separate the inertial and kinematic effects, but rather considers both interaction effects at the same time.

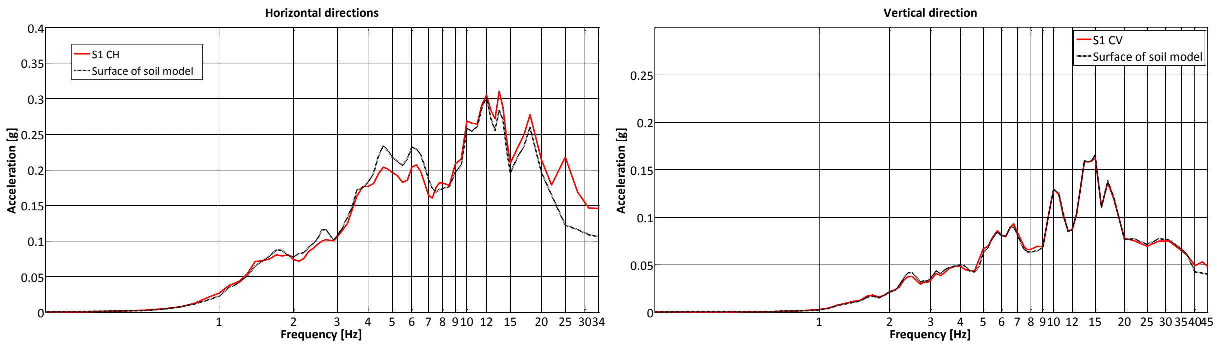

Fig. 8 – Response spectra on the surface level of foundation soil model.

5. Full soil – structure interaction model

The full SSI model shown in Fig. 5 is a combination of the reactor building finite element model and the soil finite element model minus the elements of the foundation excavation volume. The full model was validated by reproducing the tests performed on the building in 1993 and described in [3], [8] and [14]. One of the tests was reproduced with the model by applying a downward impulsive load to a node located on the ground surface level of the model, in a position comparable to that of the actual location where the impulsive loading (a 1.6 ton mass dropped from a crane) was applied. For each floor of the modeled building, the Fourier amplitude spectrum of the horizontal displacement response was obtained, and then an overall mean spectrum was calculated as the mean value of these Fourier spectra. The combined soil-structure natural frequencies were estimated as those corresponding to the peak values of the mean Fourier spectrum as shown in Fig. 9.

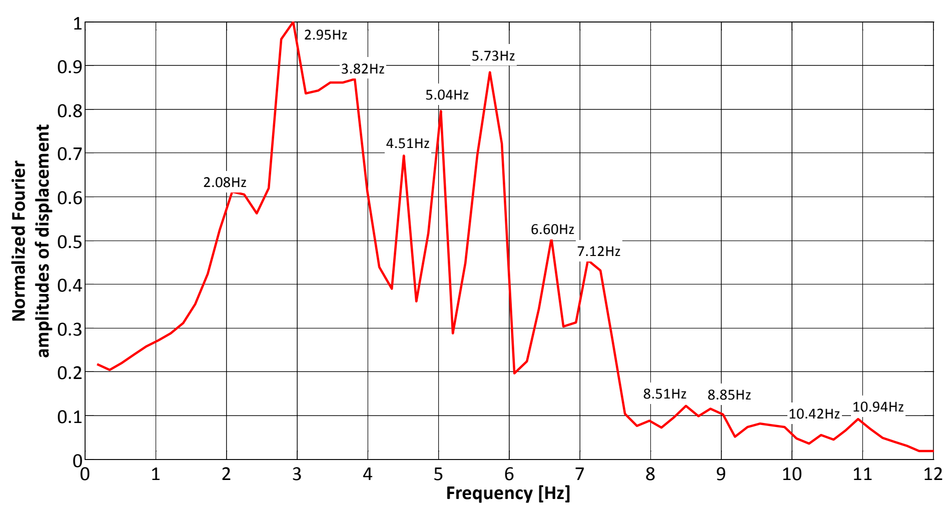

Fig. 9 – Mean spectrum of Fourier amplitudes of floor horizontal displacement.

The frequencies presented in Fig. 9 are compared in Table 2 with those calculated in [3] from the records of impulsive tests. The frequencies determined by forced harmonic vibration tests which were conducted on the main reactor building in 1993 and reported in [8] are also included in Table 2. The frequencies of 2.08 Hz, 3.82 Hz and 5.04 Hz found by the SSI model match some frequencies identified during a set of micro-vibration tests performed in 1994 by Kajima [15].

Table 2 – Natural frequencies obtained with the full SSI model, compared to those measured during tests.

Natural frequencies [Hz] Mode	1	2	3	4	5	6	7	8	9	10	12	13
SSI FE Model	2.08	2.95	3.82	4.51	5.04	5.73	6.60	7.12	8.51	8.85	10.42	10.94
Impulsive test	---	2.72	---	4.42	---	5.47	6.49	7.62	8.38		10.31	11.25
Steady state tests												
X 18.80 m	---	2.9	---	4.5	---	5.9	---	7.3	9.2		10.7	11.5
X 0.50 m	---	2.9	---	4.5	---	5.9	---	7.3	9.0		10.5	11.4
Y 18.80m	---	2.9	---	4.5	---	6.2	6.90	7.9	9.1		---	11.2

6. Floor response spectra

For each free field input motion (S1, S2, S3 and S4) and each excitation direction (X, Y, Z), a separate dynamic analysis of the full SSI model was performed by direct integration of the equations of motion in time domain using the commercial software SAP2000. For excitation directions X and Y, the input excitation is the horizontal force history which, when applied at the bottom of the foundation model, produces at the surface level a horizontal acceleration matching that of the horizontal component of the control motion. For the Z direction, the input excitation is the vertical force history which, when applied at the bottom of the foundation model, produces at the surface level a vertical acceleration matching that of the vertical component of the control motion. For each location considered within the building, floor acceleration histories in three directions (X, Y, Z) produced by each excitation were calculated and processed to generate in-structure response spectra. Next, for each location, the collinear spectra resulting of each excitation direction were combined according to the SRSS criterion. This procedure was repeated for the four different free field input motions, and the final response spectra for each location and direction were calculated as the envelope of the spectra resulting from the different input motions. Finally, the usual peak broadening and smoothing procedures recommended in reference [16] were applied to generate the FRS for design. Fig. 10 and Fig. 11 show the envelope and smooth spectra for three different levels within the building. The corresponding floor levels are identified in Fig. 12.

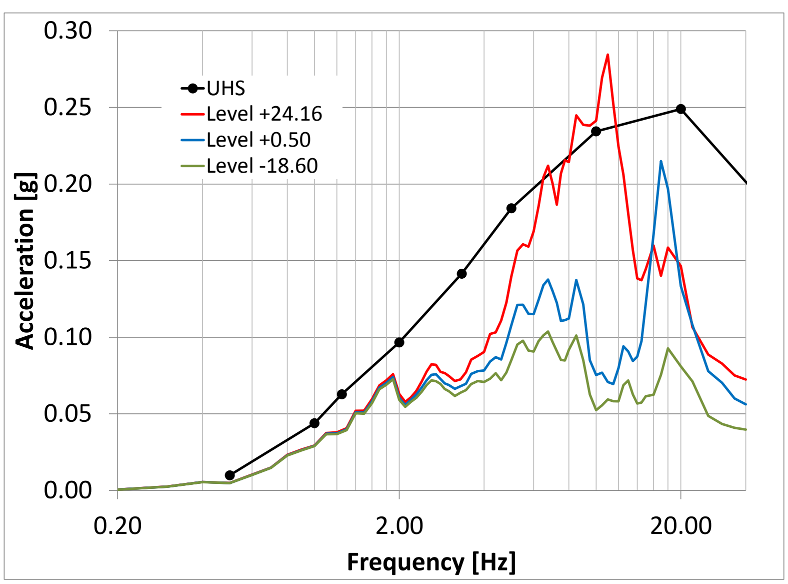

Fig. 10 – Envelope floor response spectra, 5% damping, direction X.

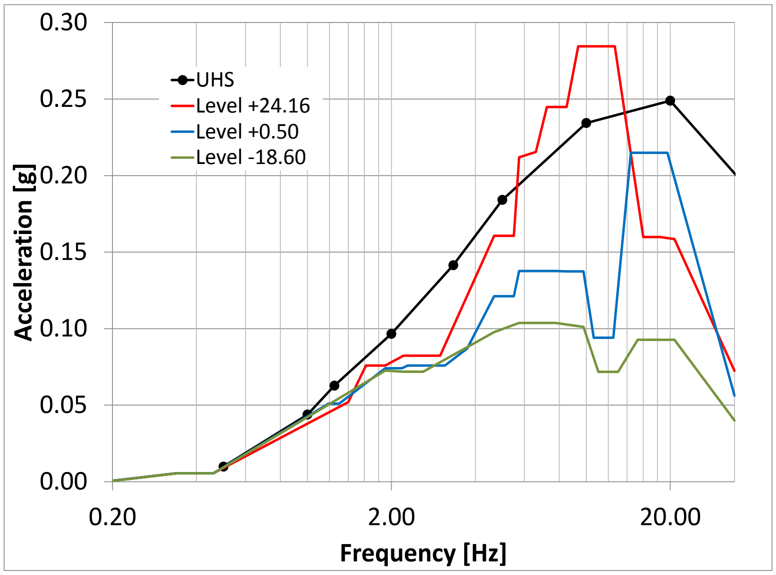

Fig. 11 – Smoothed and broadened floor response spectra, 5% damping, direction X.

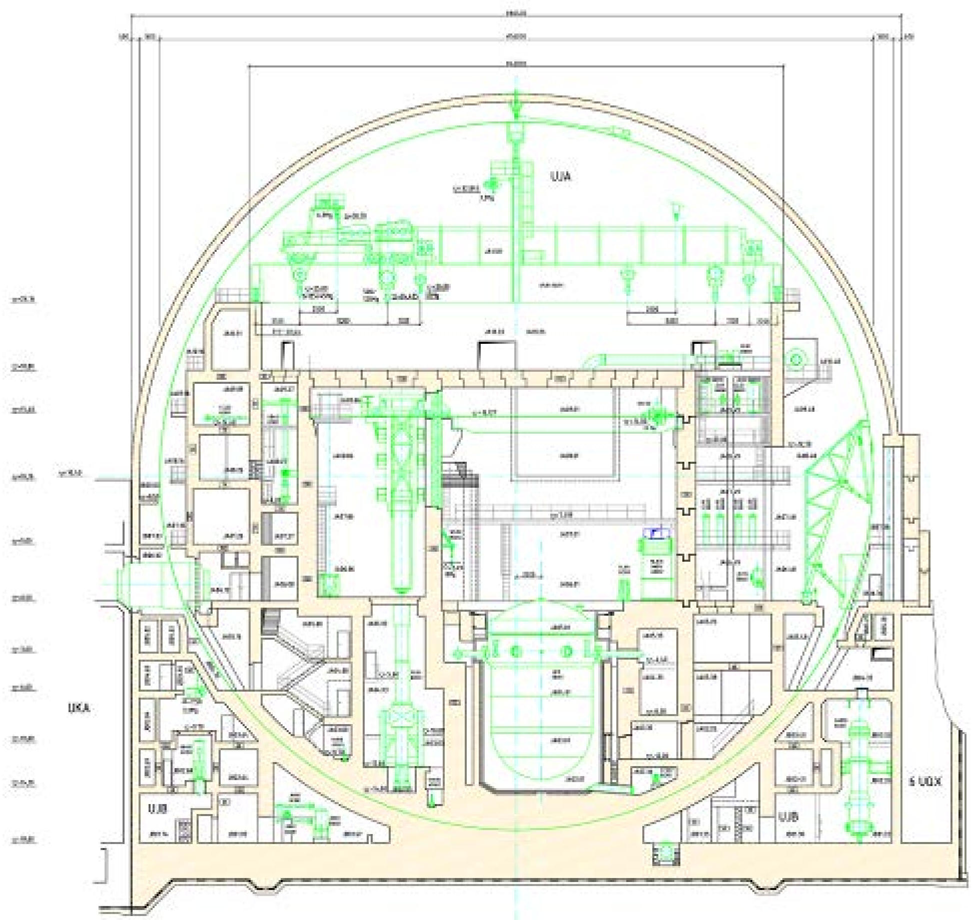

Fig. 12 – Vertical section of the reactor building (parallel to direction X).

7. Conclusions

A three dimensional finite element model of the main reactor building and foundation soil of Atucha II nuclear power plant was used to determine the floor response spectra corresponding to IAEA SL-2 earthquake level. Calculations were performed in time domain, solving the full soil-structure interaction problem. The boundary conditions of the model allowed considering the outgoing wave radiation impinging on both the artificial vertical boundaries and the artificial lower boundary of the model. The input seismic excitation was defined in terms of force time histories applied at the lower boundary of the model. These force time histories were calculated so as to produce on the surface level of the free field model (unexcavated and without any building structure) a response matching that of the postulated control motion. Next, by applying the calculated force time histories to the complete soil-structure model, the change in free field motion caused by the excavation, by the embedded structure stiffness and by the inertial forces arising from the dynamic response of the structure was taken into account, thus considering both the kinematic and the inertial interaction effects.

The finite element model was validated by reproducing an impulsive test performed on the building in 1993. The natural frequencies calculated with the model match closely to those identified from the test results.

The floor response spectra show distinctive peaks for frequencies of approximately 11 Hz and 18 Hz (Fig. 10 and Fig. 11). The peaks at about 10 Hz are greater at higher levels of the model and the peaks at 18 Hz increase at floor levels near the ground surface level. The frequency around 10 Hz corresponds to one of the frequencies identified in references [3] and [8]. The frequency of 18 Hz corresponds to a natural frequency which can be identified in the forced vibration tests performed by Kajima [15]. Comparisons between the dynamic response of the model and the free-field ground motion (represented by the uniform hazard spectrum in Fig. 10 and Fig. 11) show little amplification for the floors located above the ground level, and reduction of the input motion with depth in the soil. This is consistent with the observed data from actual earthquakes affecting embedded structures.

8. References

[1] Instituto de Investigaciones Antisísmicas Ing. Aldo Bruschi (IDIA) (2010): *Revaloración de la amenaza sísmica en el sitio de emplazamiento de la Central Nuclear Atucha II*. Universidad Nacional de San Juan, Argentina.

[2] Bolognesi-Moretto Ingenieros Consultores S.R.L. (1980): *Central Nuclear Atucha II. Informe Nº 1. Informe Nº 2*. Argentina.

[3] Ceballos M.A., Car E.J, Prato T.A., Prato C.A., Alvarez L.M. (1998): Experimental and numerical determination of the dynamic properties of the reactor building of Atucha II NPP. *Nuclear Engineering and Design*, **182**, 93-106.

[4] Masuda K., Iwamoto K., Kato H., Prato C.A., Iizuka S. (1999): Numerical simulation of soil structure interaction tests of the reactor building of Atucha II NPP: part I. *Transactions of the 15th International Conference on Structural Mechanics in Reactor Technology (SMiRT 15), paper K02/4*.

[5] Pinto F., Godoy A.R., Prato C.A. (1999): Numerical simulation of soil structure interaction tests of the reactor building of Atucha II NPP: part II. *Transactions of the 15th International Conference on Structural Mechanics in Reactor Technology (SMiRT 15), paper K02/5*.

[6] Idriss I.M, Sun J.I. (1992): *User's manual for SHAKE91*. Center for Geotechnical Modeling, Department of Civil & Environmental Engineering, University of California, Davis.

[7] Bardet J.P, Ichii K., Lin C.H. (2000): *EERA. A computer program for Equivalent-linear Earthquake site Response Analysis of layered soil deposits*. Department of Civil Engineering, University of Southern California.

[8] Prato C.A., Konno T., Uchiyama S., Alvarez L.M., Godoy A.R., Ceballos M.A. (1998): Full scale dynamic tests of Atucha II NPP. *Nuclear Engineering and Design*, **179**, 225-243.

[9] Computers & Structures inc. (2009): *CSI analysis reference manual for SAP2000, Etabs and Safe*.

[10] Zhang Y., Yang Z., Bielak J., Conte J.P., Elgamal A. (2003): Treatment of seismic input and boundary conditions in nonlinear seismic analysis of a bridge ground system. *16 ASCE Engineering Mechanics Conference, University of Washington, Seattle*.

[11] Zhang Y., Conte J.P., Yang Z., Elgamal A., Bielak J, Acero G. (2008): Two-dimensional nonlinear earthquake response analysis of a bridge-foundation-ground system. *Earthquake Spectra*, **24** (2), 343-386.

[12] Elgamal A., Yan L., Yang Z., Conte J.P. (2008): Three-dimensional seismic response of Humboldt Bay Bridge-foundation-ground system. *ASCE Journal of Structural Engineering*, **134** (7), 1165-1176.

[13] Lysmer J., Kuhlemeyer R.L. (1969): Finite dynamic model for infinite media. *ASCE Journal of the Engineering Mechanics Division*, **95** (4), 859-878.

[14] Masuda K., Maeda T., Uchiyama S. (1995). "Full-scale vibration test of Atucha II N.P.P – part IV: numerical simulation of steady-state vibration response by axisymmetric FEM". Transactions of the 13th International Conference on Structural Mechanics in Reactor Technology (SMiRT 13), paper H019-1.

[15] Kajima Corporation (1994): *Report of the Atucha II vibration test*.

[16] U.S. Nuclear Regulatory Commission (1978): *Development of floor design response spectra for seismic design or floor-supported equipment or components. Regulatory guide 1.122*.