Validation of Numerical Model of Embalse Nuclear Power Plant Based on Free Field and In-House Records of a Seismic Event

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ABSTRACT: The Embalse NPP, located in Córdoba, Argentina, is currently being reassessed in order to pursue a renewal of the operating license, due in 2010. New information pertaining to the seismicity of the region has warranted an updated dynamic analysis in order to verify the adequacy of the reactor building to perform as intended in the original design under the Maximum Credible Earthquake. A dynamic finite element model has been setup, from which a synthetic model representing the first 60 fixed-base modes is extracted and incorporated in the SASSI computer program. The model has been verified and calibrated using actual acceleration records obtained during a minor seismic event (PGA ≈ 1.4% g) that took place in 2003. This paper presents the modeling procedure followed in order to obtain the synthetic model and the results of the calibration, where it is seen that the model accurately represents the behavior of the structure recorded during the 2003 seismic event both in the free field and at two locations inside the reactor building.

INTRODUCTION

The Embalse NPP (Córdoba, Argentina) is a thermonuclear facility for electric power generation with a 648 MWe raw capacity. The power generator was the fourth CANDU (Canadian Uranium Deuterium) reactor to be put in commercial operation and its performance to date has been very satisfactory, with an average charge factor of 84%.

The plant was designed in the late 70s by Atomic Energy of Canada, Ltd. (AECL, Canada) and a consortium of local engineering companies led by Techint (Argentina), whereas the main contractor was Italimplanti (Italy). During the licensing process, the National Institute of Seismic Prevention of Argentina (INPRES, in Spanish) increased the design earthquake requirements from a peak ground acceleration, PGA = 0.14 to 0.35 g. This prompted a detailed review of the seismic performance of the design, which was carried out by the National University of Cordoba (Argentina) in the early 80s. As a result of the requirements incorporated to the design criteria and assumptions in the original design, as well as a final reassessment of the seismic the demands from PGA = 0.35 g to 0.26 g after an independent review by external consultant (D’Appolonia), the original design was found to be adequate and no reinforcements were warranted.

The plant is currently being reassessed in order to pursue a renewal of the operating license, due in 2010. As a part of the review process, the current owner of the plant, Nucleoelecátrica Argentina, S.A. (NA-SA), has retained the University of Cordoba to perform updated seismic analyses using state-of-the-practice analysis tools in order to study the response of the reactor building on the basis of the 0.35 g earthquake originally dictated by the INPRES. The present paper describes the details of the numerical model, as well as a validation performed by means of experimental records of the structure response to a minor seismic event (PGA < 2% g) that took place in 2003. Two companion papers describe the seismic environment and the response of the reactor building to the design event.

NUMERICAL MODEL OF REACTOR BUILDING

Superstructure

The reactor building consists of an external pre-stressed concrete cylindrical shell and dome (external structure), and an internal building formed by several concrete slabs and shear walls (internal structure), connected to a 1.7-m thick concrete foundation slab. The thickness of the external shell structure is 1.10 m, whereas the outside diameter of the cylinder is 43.70 m. The masses of the internal and external structures of the reactor building are 38,000 and 26,000 metric tons, respectively.

Figure 1 illustrates sketches of the internal structure, where the complexity of its configuration can be appreciated. One of the main goals of the current analyses is to take advantage of state-of-the-practice analysis tools that would help reduce the uncertainties (and consequently, conservatism) in the analyses carried out in the early 80s. Due to computational limitations at the time, previous analyses did not accurately represent soil-structure interaction and damping due to wave radiation phenomena.

The current model of the soil-structure system is setup by means of the computer program SUPER SASSI PC [1], which consistently represents the aforementioned phenomena, but is not very user-friendly for setting up complex structures. The external structure was thus represented by means of a stick model, whereas the complexity of the internal structure was such that conventional stick models could not straightforwardly be obtained. Hence, a generalized stiffness matrix for the internal structure was defined in terms of generalized fixed-base modal coordinates, considering only the modes that fall within the frequency range of interest, and degrees of freedom defined at the base slab. This technique, herein referred to as dynamic condensation (e.g.,[2]), can only be used in problems where the base can be assumed as rigid. Parametric studies showed this to be a reasonable assumption.
Figure 2 shows the finite element (FE) model of the internal structure developed by means of the computer program SAP2000 [3]. The FE model consists of Timoshenko beam and Mindlin-Reissner shell elements, having a total of 1056 nodes and 6336 DOFs. The criterion followed in order to select the number of modes to be considered is that the natural frequency of the last mode should be twice the highest frequency of interest. For the current analyses, the frequency range of interest has been established as \( f = 0 - 12 \) Hz, hence the last mode to be considered has a natural frequency of 24 Hz, which in this case is the 60th mode. Thus, in order to perform the dynamic condensation, the first 60 fixed-base modes are obtained.

Once the fixed-base modes, \( \Phi_k \), are obtained, the following transformation is performed in order to obtain displacements at the superstructure degrees of freedom (DOFs), \( u_s \), as a function of the displacements at the base, \( u_c \), and generalized fixed-based modal coordinates, \( p_k \), where \( k \) is the number of modes considered in the analyses:

\[
\begin{bmatrix} u_s \\ u_c \end{bmatrix} = \begin{bmatrix} \Phi_s \\ \Phi_c \end{bmatrix} \begin{bmatrix} p_s \\ 0 \end{bmatrix} = T_k \begin{bmatrix} p_s \\ p_c \end{bmatrix}
\]

(1)

where \( \Phi_s \) are the \( k \) modal components at the superstructure DOFs and \( \Phi_c \) are the components at the base. The stiffness and mass matrices in the new coordinate system are given by:

\[
K^* = T_k^T \cdot K \cdot T_k = \begin{bmatrix} \cdots & \omega_k^2 & \cdots \\ \omega_k^2 & 0 & \cdots \\ \cdots & 0 & 0 \end{bmatrix}
\]

(2)
It should be noted in Eq. (1) that the generalized coordinates at the base are actual base displacements. Thus, the stiffness and mass matrices in Eqs. (2) and (3) can readily be coupled with the foundation model developed in SASSI at the base.

Foundation

An in-situ testing program was carried out in order to determine the dynamic stiffness of the soils within the site. The shear wave velocity profile of \( V_S \) of the soils within the site was determined following the Spectral Analysis of Surface Waves (SASW, \([4]\)) method. The test involves generating a mechanical impact at the ground surface by means of a falling mass and monitoring the surface wave generated in terms of vertical accelerations at two locations aligned with the point of impact (see \([4]\) for full details). By performing spectral analyses of the recorded acceleration time histories, an experimental “dispersion curve” is derived. The dispersion curve shows the variation of the phase velocity with wavelength for the range of frequency of interest, where the measurements are accurate and are not significantly affected by ambient noise. By means of an analytical wave propagation model, a theoretical dispersion curve is calculated for a tentative shear wave profile. The tentative soil profile is then adjusted to provide an adequate fit to the experimental dispersion curve.

A total of 3 SASW tests were performed at different locations within near the reactor building, whereas 2 other tests were performed at other locations within the site (Figure 3). The shear wave velocity of bedrock was assumed at \( V_S = 1,500 \text{ m/s} \), and the dispersion curve adjusted by performing iterations on the velocities of the remaining superficial layers. Figure 4 shows a typical match of experimental and theoretical dispersion curves, whereas results obtained for the 3 locations near the reactor building are summarized in Table 1.

Figure 3. Location of SASW test sites

Figure 4. Typical dispersion curve adjustment

Results in Table 1 show that there is some spatial variability (tests were performed about 20 m away from each other), particularly at shallow depths (< 7.5 m). It is considered that shallow layers may have been disturbed during construction of the foundation slab, which bears at a depth of 8 m from natural grade level. Hence, an average profile was obtained and the shear wave velocities of the shallow layers were reduced by about 30% in order to account for increased disturbance of soils adjacent to the reactor building. Figure 5 shows a comparison between the \( V_S \) profile considered in the current analyses and previous estimates. It is seen that, although the current profile provides a more detailed variation of \( V_S \) with depth, the overall trends and values are quite similar.

Since the model discussed in this paper is validated against the recorded response of the structure to a minor earthquake event that took place in 2003, the low strain soil properties (i.e., \( V_S \)) were not adjusted for nonlinear behavior at this stage. In fact, SHAKE91 \([5]\) runs showed that the 2003 event (PGA \( \approx 1.4\% \text{ g} \) at top of soil deposit) was likely to have produced very low strains.

Figure 6 shows a layout of the SASSI model, which was developed using the soil properties given in Table 1. The original site (i.e., without excavation and structure) is modeled by means of a collection of horizontal layers, which are infinite in the horizontal direction, over a halfspace, which is represented by means of the variable depth method \([1]\). The excavated area is represented by means of 6 and 8 node finite elements and its stiffness matrix is “subtracted” from the one obtained for the original site. The size of the mesh is dictated by the largest frequency to be accurately represented by the model considering that the maximum element size must be less than \(1/6\) of the shear wavelength. Since the mesh is extruded in the vertical direction, the lowest velocity layer (i.e., uppermost in this case) controls the mesh size. However, parametric studies showed that, for the present case, the uppermost layer does not significantly affect the overall soil-
structure system response. Hence, in order not to have an unnecessary fine mesh, the second layer is considered to establish the maximum mesh size for the FE model. In this case, the largest element size yields:

\[
\Delta h_{\text{max}} = \frac{1}{6} \frac{V_s}{f_{\text{max}}} = \frac{1}{6} \frac{1350 \text{ m/s}}{12 \text{ Hz}} = 4.86 \text{ m}
\]

(4)

The model considers the excavated soil region to be rigid, as there are several shear walls in this region confined by the foundation and level ground floor slabs. An interaction node is defined at the center of the foundation slab where the superstructure is connected to the soil-structure interaction model (Figure 6).

<table>
<thead>
<tr>
<th>Depth [m]</th>
<th>Site 1</th>
<th>Site 2</th>
<th>Site 3</th>
<th>Ave.</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 2.5</td>
<td>257</td>
<td>206</td>
<td>172</td>
<td>212</td>
<td>180</td>
</tr>
<tr>
<td>2.5 – 5</td>
<td>500</td>
<td>450</td>
<td>550</td>
<td>500</td>
<td>350</td>
</tr>
<tr>
<td>5 – 7.5</td>
<td>750</td>
<td>461</td>
<td>750</td>
<td>654</td>
<td>460</td>
</tr>
<tr>
<td>7.5 – 10</td>
<td>900</td>
<td>823</td>
<td>900</td>
<td>874</td>
<td>865</td>
</tr>
<tr>
<td>10 – 12.5</td>
<td>1,000</td>
<td>1,000</td>
<td>1,000</td>
<td>1,000</td>
<td>1,000</td>
</tr>
<tr>
<td>12.5 – 15</td>
<td>1,100</td>
<td>1,100</td>
<td>1,100</td>
<td>1,100</td>
<td>1,100</td>
</tr>
<tr>
<td>15 – 17.5</td>
<td>1,200</td>
<td>1,200</td>
<td>1,200</td>
<td>1,200</td>
<td>1,200</td>
</tr>
<tr>
<td>&gt; 17.5</td>
<td>1,500</td>
<td>1,500</td>
<td>1,500</td>
<td>1,500</td>
<td>1,500</td>
</tr>
</tbody>
</table>

Figure 5. Average shear wave velocity profile

Figure 6. SASSI model (not to scale)
MODEL VALIDATION

A minor earthquake (PGA < 1.5%) occurred in the vicinity of the NPP in December, 2003, which triggered the seismic monitoring system that had been installed at the plant sometime before the event. The earthquake motions were recorded in terms of time histories of acceleration at two locations within the reactor building (S2 and S3) and at ground surface about 150-m from the building (S1). Figure 6 shows the approximate location of locations S1 to S3 in the SASSI model.

INPRES assigned a moment magnitude $M_w = 5.0$ to the event, whereas USGS estimated the Compressional Body Wave Magnitude, at $m_b = 4.1$. The location of the epicenter has been estimated between 6 and 12 km (USGS and ISC, respectively), whereas the hypocenter of the event has been located at a depth of 47 km by ISC. Table 2 shows the peak acceleration values recorded at locations S1-S3, where direction X is 120° South and Z is vertical.

In order to validate the reactor building model, the rock base motion was determined by means of the free field record at S1. A SASW test performed at the free field location, S1 (SASW location 5, Figure 3), indicated that the ground conditions were similar to the rest of the site. Hence, an amplification study was carried out in order to filter out the site amplification effects to obtain the actual rock base motions. The amplification study was performed by means of the computer program SHAKE91.

<table>
<thead>
<tr>
<th>Direction</th>
<th>S1</th>
<th>S2</th>
<th>S3</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>1.46%</td>
<td>0.70%</td>
<td>1.06%</td>
</tr>
<tr>
<td>Y</td>
<td>1.27%</td>
<td>0.56%</td>
<td>1.22%</td>
</tr>
<tr>
<td>Z</td>
<td>2.33%</td>
<td>0.73%</td>
<td>2.27%</td>
</tr>
</tbody>
</table>

The rock base motions were thus input as control motion at bedrock level in the SASSI model and the response was calculated in the generalized degrees of freedom considering a hysteretic structural damping ratio of 2%. The structural damping considered reflects the low strains imposed by the earthquake. The seismic environment assumed vertically propagating shear waves polarized in horizontal planes (SH) for the horizontal motions and vertically propagating compressional body waves (P) for the vertical component of the control motion. Once the SASSI model is run, the response in the generalized degrees of freedom is obtained. The response in the superstructure DOFs may be obtained by the coordinate transformation given in Eq. (1). An alternative, and equally rigorous approach, is to obtain the motions from the SASSI model at the interaction node (Figure 6) and run the SAP2000 model for a prescribed support motion at the interaction node. The response calculated at locations S2 and S3 is compared to the actual records in terms of both transfer functions and response spectra, as described below.

Transfer functions
Transfer functions are defined in the frequency domain as the quotient between the steady-state response and input for each frequency. The analytical transfer functions are evaluated directly by means of the SASSI model. For the evaluation of the experimental transfer functions, the response is defined as the Fast Fourier Transform (FFT) of the transient acceleration records at S2 and S3 whereas the input is defined as the FFT of the rock base motions calculated by means of SHAKE91. E.g., the transfer function for location S2 in the x direction is evaluated as $H_{2x} = S_2x(\omega)/S_{1x}(\omega)$. Since the only input considered for the response is $S_{1x}$, this definition assumes that there is no coupling between directions x, y, and z. This is considered to be a reasonable approximation.

One shortcoming often encountered in the evaluation of the experimental transfer function using transient response records, is that within certain frequency ranges the input may have very low signal-to-noise ratios, thus leading to inaccurate estimations. This shortcoming can be circumvented if a coherence function is calculated on the basis of several records. However, in the present case, the response to a single earthquake has been recorded. Nevertheless, the experimental transfer functions calculated on the basis of a single earthquake are considered to be an estimate and it is expected that they lack accuracy in certain frequency ranges. Figure 7 and Figure 8 show experimental vs. analytical transfer functions for locations S2 and S3, respectively.

![Figure 7. Transfer function at location S2](image-url)
It is seen that there is a reasonable match for the horizontal components in the X and Y directions, the X direction showing a closer agreement, whereas the match appears to be poorer in the vertical direction. It should however be noticed that outside the range where the vertical input motion (i.e., vertical acceleration records at S1) has low amplitudes (f = 4.5 to 8.0 Hz) the match is reasonable in the vertical direction as well. The remaining differences in the vertical direction may be attributed to the assumed wave field, which consists only of P-waves for vertical motions.

**Response spectra**

Figure 9 and Figure 10 show 5%-damped in-house elastic response spectra obtained by means of the numerical model and the experimental records. The 5% damping is arbitrarily chosen for response spectra comparison and does not reflect the structural damping, which was considered to be equal to 2% for the low strains induced by the earthquake.

It is noted that lack of input at certain frequencies does not affect model validation by means of response spectra comparisons, since both calculations and measurements should respond to the same input whereas transfer functions are independent of the input frequency content. It is seen that the model accurately captures both the magnitude and the frequency content of the response, the model being slightly conservative in its predictions.

**CONCLUSIONS**

A numerical model has been developed in order to represent the seismic response of the Embalse NPP including dynamic soil-structure interaction phenomena. The numerical model has been setup in SASSI and includes generalized stiffness matrices generated by means of SAP2000 in order to represent the dynamic behavior of the internal structure at frequencies below 24 Hz. The geometry and mechanical properties of the structure were obtained from drawings and design phase reports whereas the soil properties were obtained recently by means of the SASW method.

A minor seismic event that took place in 2003 triggered the seismic monitoring system of the plant, and the numerical model was validated by comparing calculated vs. recorded response of the building to this event. The
comparison showed that the model represents the behavior of the soil-structure system with sufficient accuracy in the frequency range of interest (f = 0 to 12 Hz).

A numerical model of the soil-structure system of the NPP building is thus available for studying the response of the structure to the originally defined earthquake of 0.35 g. Detailed studies of the applicability of the 0.35 earthquake, as well as the results of the structural response, are given in two companion papers.

REFERENCES