Seismic Margin Assessment of a Reactor Building

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ABSTRACT

The main objective of this paper is to evaluate the seismic behavior of reactor building 7 at the Kashiwazaki-Kariwa Nuclear Power Station for the 2007 Niigataken-chuetsu-oki earthquake. The paper summarizes the work done during the KARISMA Benchmark: generation and verification of the Finite Element model of the reactor building, analysis of the building taking into account the soil-structure-interaction and assessment of the seismic margin using two nonlinear analysis methods, namely the Pushover Analysis and a nonlinear time history analysis.

The results show that a simplified consideration of the soil-structure-interaction with frequency-independent equivalent springs and dampers as elastic support can lead to comparable results to a full soil-structure analysis.

The seismic margin assessment using pushover analysis and nonlinear dynamic time-history analysis allows the conclusion, that the building (without consideration of failure of the soil) has additional margins in strength of about 4 to 6 times the occurred earthquake. However, a definite criterion for a failure cannot be specified.

INTRODUCTION

On 16 July 2007, a strong earthquake, the Niigataken-chuetsu-oki earthquake (NCOE), affected the Tokyo Electric Power Company (TEPCO) Kashiwazaki-Kariwa Nuclear Power Station (NPS), the biggest nuclear power plant in the world, located at about 16 km away from the epicenter. The large amount of observations and data collected led to the organization of the Kashiwazaki-Kariwa Research Initiative for Seismic Margin Assessment (KARISMA).

The earthquake was accompanied by very high peak ground accelerations of up to 0.68 g at the foundation, reaching the 1.1 to 3.6-fold design values of the NPS. The 3 blocks which were in operation and another block in the start-up phase were shut down. During and after the earthquake all reactors were in safe condition.

In view of the large number of data measured inside and outside the power plant – acceleration time histories in/at the ground, in the buildings and at plant components – the idea matured to evaluate within the scope of an IAEA-program the seismic behavior of the plant with consideration of national rules and regulations, i.e. the seismic safety of an existing plant on the basis of the measured data.

The recorded measurement data and the “reanalysis” thus make it possible to investigate the real behavior of the plant during the earthquake. The damage or non-damage is evaluated in relation to the ground motions which were recorded by the seismic instrumentation and the occurrence of which could thus be confirmed, and to the accelerations measured in the plant itself.

The following paper summarizes the work done by the authors during the KARISMA benchmark Task 1 – Structure, starting with the generation and verification of the Finite Element (FE) model of the reactor building. With this FE model an analysis of the structure response corresponding to the excitation
of NCOE is performed taking into account the effects of the soil-structure-interaction (SSI). In a last step the seismic margin is assessed with the help of two nonlinear analysis methods, namely the Pushover Analysis and a nonlinear time history analysis. The investigations of the reactor building are carried out on the basis of the documentation made available by the TEPCO Guidance Document.

NUMERICAL MODELS

In the framework of KARISMA Benchmark two different approaches are used to represent the reactor building and supporting medium. On one hand SASSI a computer code that models structure and soil and that solves the complete SSI problem in the frequency domain is applied. On the other hand the Finite Element (FE) program SOFISTIK is used. Since SOFISTIK solves the problem in the time domain nonlinear structural effects can be considered. In this approach the SSI is approximately taken into account by equivalent springs and dampers that represent the soil. In both approaches the embedment of the structure is taken into account.

Figure 1 shows the general overview of the reactor building 7 and figure 2 the corresponding FE-models. For calculations with SASSI a model with a relatively coarse mesh (left hand side) and for SOFISTIK analyses a finer mesh (right hand side) is used. The basemat is built up with solid elements. The FE-mesh of the rest of the structure mainly consists of shell elements for walls and slabs and beam elements for columns and beams. Geometry data and material parameters are taken from the Guidance Document. Shear walls are included in the model except for auxiliary walls. Sloshing of the spent fuel storage pool is modeled by equivalent impulsive and convective masses and springs. The interior of the containment vessel is modeled as a stick model according to the Guidance Document.

Figure 1. General overview of reactor building 7
For wave propagation calculations and for consideration of nonlinearity due to the strain dependency of the soil the program SHAKE is used. Therefore, the soil layers are assumed to be horizontal and of infinite extent. The reactor building 7 is embedded in the sedimentary rock layer of the Nishiyama formation and thus, the building reaches 25.7 m into the ground. For a realistic analysis of the building the consideration of the SSI is necessary. As mentioned above, for consideration of SSI two approaches are followed, namely the complete analysis of the coupled soil-structure model in SASSI and the approximate consideration in SOFISTIK. For the approximation frequency-dependent impedance functions are determined with SASSI for the embedded foundation structure (see figure 3). Based on these impedance functions frequency-independent equivalent springs and dampers as elastic support are derived. The discrete springs and dampers are distributed at all surfaces embedded in the soil. For the distribution of the springs it is assumed that the soil at the front surface (corresponding to the direction of excitation) and at the bottom surface is activated as support.

Additionally, for the structure a mass and stiffness proportional Rayleigh damping is applied. The damping coefficients $\alpha$ and $\beta$ are chosen to get a damping ratio of about 5% in the range of the main eigenmodes of the structure.
MODAL ANALYSIS

For the SOFISTIK model with equivalent springs a modal analysis is performed. Table 1 shows the first twelve modes which cover over 97% of the modal mass.

Table 1: FE model of the reactor building with equivalent soil springs and corresponding modes

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz)</th>
<th>Modal participating mass ratios (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>UX</td>
</tr>
<tr>
<td>1</td>
<td>0.17</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>0.17</td>
<td>1.22</td>
</tr>
<tr>
<td>3</td>
<td>0.19</td>
<td>0.00</td>
</tr>
<tr>
<td>4</td>
<td>0.23</td>
<td>0.42</td>
</tr>
<tr>
<td>5</td>
<td>2.08</td>
<td>0.00</td>
</tr>
<tr>
<td>6</td>
<td>2.11</td>
<td>68.63</td>
</tr>
<tr>
<td>7</td>
<td>2.20</td>
<td>0.00</td>
</tr>
<tr>
<td>8</td>
<td>3.46</td>
<td>0.00</td>
</tr>
<tr>
<td>9</td>
<td>4.54</td>
<td>0.06</td>
</tr>
<tr>
<td>10</td>
<td>4.87</td>
<td>0.00</td>
</tr>
<tr>
<td>11</td>
<td>5.15</td>
<td>0.07</td>
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<tr>
<td>12</td>
<td>5.22</td>
<td>27.32</td>
</tr>
<tr>
<td>SUM</td>
<td></td>
<td>97.72</td>
</tr>
</tbody>
</table>

The first four modes correspond to the sloshing of the spent fuel storage pool. The first main modes of the reactor building are at about 2 Hz.

To compare the SOFISTIK model with equivalent springs with the SASSI model with full SSI the response spectra at the basemat and the mezzanine locations are calculated. The excitation used for the analysis is described in the next section. Figure 4 shows typical results exemplary for the y-direction. The results show a good correlation especially in the frequency content. The amplitude depends on the damping and it is not possible to achieve a perfect correlation.

Figure 4. Typical response spectra - comparison SASSI with SOFISTIK
SEISMIC EXCITATION

For the seismic excitation the intended procedure is to use the recorded signal at Unit 5 free field seismometer (5G1) as input for the response analysis of reactor building 7. The soil column analyses under main- and aftershocks are performed with SHAKE. SHAKE computes the response in a horizontally layered soil-rock system subjected to vertical travelling shear waves. The cyclic behavior is approximated by an equivalent linear model. The soil properties for Unit 5 free field are based on the provided data in the Guidance Document.

To check whether consistent results can be obtained from the SHAKE analysis two analyses are performed. In the first one the signal from seismometer 5G1 (nearest to the surface) is used as input motion and the responses inside the soil at locations corresponding to seismometer positions (G51-G55) are determined. The second analysis is conducted the other way around. Thus, recordings from borehole position G55 serve as input for the analysis and responses at the seismometer locations are determined. Figure 5 exemplarily shows the response results at level -24.0 m TMSL (G52) derived from both approaches as well as the response spectrum of the recorded signal under Aftershock I. Especially in the region of soil deposit eigenfrequencies the one-dimensional SHAKE analysis provides considerable exceedances to the recorded responses.

Figure 5. Exemplary soil responses and contour lines of outcropping bedrock of Nishiyama formation

In the area north of reactor blocks 5, 6 and 7 an upwarp of the rock formation of up to 30 m can be observed. This is contrary to the assumption of horizontal layers of infinite extent that is an underlying requirement for SHAKE analyses. Hence, it is explainable that the one-dimensional SHAKE analysis can only approximately reproduce the soil behavior at the K-K site.

As a result a different approach, a best estimate, is chosen to get the input excitation for the response analysis. The structure is founded on a massive reinforced concrete basemat with a thickness of 5.5 m. This basemat is embedded in a relatively stiff rock layer. Due to that fact, the motion on top of the basemat does not differ much from the motion of the surrounding rock. So, the signal that is recorded on the basemat is assumed to be the input motion in the soil at this elevation (TMSL -11 m that corresponds to the center line of the basemat).

Figure 6 shows a comparison of recorded and calculated response spectra at the basemat and the mezzanine. The results show that with the best estimate approach a good agreement can be achieved.

As additional task, time histories at the positions of piping supports for the Residual Heat Removal (RHR) piping system are determined. The analysis of the RHR piping system is presented in a parallel paper, Block et al. (2013).
MARGIN ASSESSMENT

In a last step the margins in the design of the reactor building are assessed. For this purpose the nonlinear structural behavior is included in the model. For the nonlinear analysis special attention has to be taken for the material definitions. The utilized constitutive laws for concrete and reinforcing steel are based on stress-strain relationships according to DIN EN 1992 (German edition of Eurocode 2) and are adjusted to the measured material characteristics provided in the Guidance Document. This leads to the stress-strain-curves depicted in Figures 7 and corresponds to the following values for concrete strength and strain:

\[ f_c = 48 \text{ MPa}, \quad \varepsilon_{\text{min}} \approx -0.40 \% \]
\[ f_t = 3.5 \text{ MPa}, \quad \varepsilon_{\text{max}} = 0.02 \% \]

as well as yield strength and tensile strength with corresponding strains for reinforcing steel:

\[ f_y = 350 \text{ MPa}, \quad \varepsilon_y = 0.17 \% \]
\[ f_t = 550 \text{ MPa}, \quad \varepsilon_{\text{max}} = 5.00 \% \]
There are two different approaches for the nonlinear analysis investigated: one is the pushover analysis, a simplified static nonlinear procedure, and the other one is a nonlinear dynamic time history analysis. Since the structure does not show significant nonlinear behavior under the input motion time history derived from the records of the Mainshock (NCOE), the input signal is increased from 1xNCOE up to 6xNCOE. For the analyses the provided time histories at raft elevation (-13.7 m TMSL) are used.

A comparison of the pushover curve with and without consideration of the nonlinear material behavior of the columns inside the reactor building shows negligible differences. Thus the columns are assumed to remain elastic in the further analysis.

Soil-structure-interaction is simplified taken into account by using a system of springs and dampers based on impedance function determined with SASSI (see previous sections). The primary nonlinear effect of the soil is due to the strain-dependency of the soil parameters. This effect is taken into account in the equivalent linear analysis with SHAKE and hence is already included in the provided data. Impedance functions for soil corresponding to 1xNCOE and for soil corresponding to 6xNCOE are calculated, but the differences between those two limiting cases are found to be quite small. Thus linear springs based only on 1xNCOE impedance functions are used and the additional nonlinear effect due to an increase of the earthquake intensity level is neglected. Also no other nonlinear effect and failure mechanism are considered. The SOFISTIK model described in the previous sections is used.

**Pushover analysis**

For the pushover analysis the structure is set under horizontal loading that is increased until a failure of the structure is reached. There are different possibilities for the distribution of the load (see figure 8). Beside a uniform distribution of horizontal accelerations, that is proposed in the Guidance Document, a distribution of loading scaled according to the fundamental eigenmode is applied and the outcome of both is compared. The pushover analysis is performed according to the procedure provided in ATC 40.
The pushover curve is obtained by plotting the base shear (sum of all reaction forces) over the displacements of a specified node on elevation 23.5 m.

Figure 9 shows the pushover curves in y-direction, which is the decisive excitation direction, for a uniform load distribution that is asked for in the Guidance Document and a weighted load distribution according to the fundamental mode of the structure. For all intensity levels 1-6xNCOE performance points can be determined, which are marked on the curves. It appears that even higher intensity levels than 6xNCOE are bearable for the structure.

The different load distributions lead to quite different pushover curves. In spite of these differences of the pushover curves, the resulting performance points – maximum displacements – are in a similar range of value. A reason for that is that the structure is very compact and stiff and so the mode shape of the fundamental eigenmode does not show a pronounced deflection curve. This leads to similar capacity spectra after the transformation of the pushover curves to the ADRS format and consequently similar performance points.

Figure 9. Comparison of results corresponding to uniform loading (left) and weighted loading (right)

Figure 10. Performance point for 1xNCOE (left) and 6xNCOE (right), horizontal y-direction
**Dynamic response analysis**

For the margin assessment using a dynamic response analysis the input time histories corresponding to the different intensity levels are applied simultaneously in x-, y- and z-direction. For the margin assessment it is necessary to define a damage indicator which identifies structure collapse. One possible damage indicator proposed during the benchmark is the story drift. For different reasons this parameter does not seem to be appropriate. One problem is that the story drift – determined in the intended way – is calculated from values of maximum displacement. These maximum values do not necessarily happen at the same point of time. Uncertain is also the selection of the locations of evaluation. Above all in the case of the spring supported model a uniform rocking is included in the values that is not relevant for failure. This leads to unrealistic drift values. Furthermore, the introduction of a limit value for the performance level of failure is only specified in some regulatory guides and respective choice of this value can lead to a wide scattering of the results. It can be concluded that for more flexible structures, such as framework structures, the story drift can be advantageous and meaningful. Especially because of the compactness of the structure the use of story drift as damage indicator is limited for the regarded reactor building. Due to that fact, it is proposed to use the concrete compressive strain as the damage indicator. With the exceedance of the maximum concrete compressive strain concrete crushing is initiated. A collapse of the structure cannot be excluded after significant concrete crushing.

Nevertheless, if story drift is the decisive damage indicator, ATC 40 gives a maximum value for story drift for the Performance Level “Life Safety”. This value of 0.02 is exceeded for no intensity level, not even for 6xNCOE.

With the concrete compressive strain as a damage indicator, it appears that for 6xNCOE an exceedance of the maximum concrete strain appears. This exceedance occurs on level 12.3m at a floor that is connected to the reinforced concrete containment vessel (RCCV) as shown in figure 11. Since the RCCV is a sensible location this can be interpreted as an initiation of a collapse of the structure.

![Figure 11. Exceedance of maximum concrete compressive strain at t = 7.06 sec for 6 x NCOE](image-url)
CONCLUSIONS

The KARISMA Benchmark was a good opportunity to investigate the influence of different approaches for the seismic assessment of a reactor building. Usually the analysis methods are focused on the design of structures. So, some discrepancies between analysis results and records are inevitable as all the methods have underlying assumptions.

The main conclusions, respectively learning effects are:

- The one dimensional approach for the soil column analysis (SHAKE) is not sufficient for the Kashiwazaki-Kariwa site because the underlying assumption – one type of wave – is violated by the topology of the site.
- Using a best estimate assumption a very good correlation between measured and calculated responses in the building could be achieved. This applies equally for the structural model with equivalent springs and dampers and for the model with soil-structure-interaction. The calculated response time-histories and spectra serve as input for the RHR piping system.
- A full analysis of the soil-structure-interaction problem with e.g. SASSI can be time consuming and the calculations can sometimes be difficult to retrace. Furthermore, when nonlinearities should be taken into account a full analysis performed with SASSI is not possible. The investigations during the work on the benchmark showed that a simplified consideration of the soil-structure-interaction with equivalent springs and dampers can lead to comparable results to a full SSI analysis.
- A seismic margin assessment using pushover analysis and nonlinear dynamic time-history analysis allows the conclusion, that the building (without consideration of failure of the soil) has additional margins in strength of 4 to 6 times the occurred earthquake. However, a definite criterion for a failure cannot be specified.

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