INTRODUCTION

The evolution of safety standards due to evolution of knowledge, aging effect of installations, or feedback from recent events such as the Fukushima accident, require a continuous assessment of seismic resistance of structures. As a consequence more and more investigations conclude to deficiencies that in some cases lead to structural retrofitting. However, the experience from various studies, confirmed by observations made on site with structures exposed to severe earthquakes indicate that the effective resistance of structures is not always well assessed by classical linear elastic analysis. The purpose of this communication is to illustrate the benefits of applying appropriate methodologies based on advanced computational modeling, by considering various phenomena such as soil-structure interaction (linear and nonlinear), geometrical nonlinearity (uplift, collision), material nonlinearity (concrete cracking, rebar yielding, damping). These features allow performing reliable best estimate analyses by predicting the apparition of local degradations and their evolution to reach the ultimate capacity of a structure. Also essential to determine reliable fragility curves for other type of analysis such as seismic probabilistic risk assessment (SPRA). This paper, which is essentially an extension of previous works (Voldoire, 2006), presents the application of several methodologies elaborated to take into account different phenomena to the analyses of the SMART structure and of an actual nuclear facility.

DESCRIPTION OF THE METHODOLOGY

To simulate soil-structure interaction (SSI) in linear dynamic calculations for seismic analysis, a BEM-FEM coupling approach can be adopted, in particular, the chaining of Code_Aster (Open Source Finite Element code developed at EDF R&D) and MISS3D (Boundary Element code developed at Ecole Centrale Paris) can be used. A new hybrid Laplace-time domain approach (HLTA) allows performing nonlinear transient SSI calculations. Material behaviors of structural and nonstructural elements are of great interest to predict the ultimate capacity of structures. Several constitutive models dedicated to reinforced concrete elements have been developed for various types of applications (pushover or dynamic transient analyses). They concern beams, columns, shear walls and slabs, and non-structural elements such as masonry infill. The use of such material models allow to predict the ductility, resistance decay and the ability of elements to dissipate energy through cyclic loading. Geometrical nonlinearity modeling is necessary to describe various situations, such as foundation uplift and collision between structures located close to each other. Their modeling has required the elaboration of specific constitutive laws and guidelines to calibrate the numerical parameters. In some cases, the retrofitting of an existing structure may be obtained using isolation devices such as interstory bracing dampers. The assessment of their efficiency to reduce seismic response of structure requires the use of nonlinear viscous constitutive law by performing dynamic time history analyses. This matter is also elaborated in the communication, by describing the methodology and the presentation of results.
For critical structures, it can be important to have an accurate evaluation of seismic fragility based on numerical simulation. Fragility curves can be evaluated calculating N structural responses for different earthquake loads and taking into account uncertainties related to model parameters. To illustrate the benefits and advantages of such analyzing techniques in assessing the seismic resistance of structures, several case studies are presented in this communication. These different methodologies and tools have been applied to the SMART RC wall structure tested in the CEA Saclay seismic experimental facility (Richard et al., 2012, 2013). It is also important to show that the application to real installations of the nuclear industry which requires larger numerical models and computational effort is also feasible with reasonable cost and time-schedule.

**SEISMIC RESPONSE OF THE SMART STRUCTURE CONSIDERING MATERIAL NONLINEARITIES AND SOIL-STRUCTURE INTERACTION**

This section gives some insight on how much nonlinear phenomena, as well as soil-structure interaction effects, can modify the dynamic response of RC structures. In particular, the numerical model elaborated for the SMART-2008 benchmark (EDF and CEA, 2007) is used in the present discussion.

The nonlinear behavior is taking into account through a macroscopic damaging constitutive model developed for reinforced concrete (RC) plates elements. It is written according to the theory of generalized standard materials and named GLRC_DM (Markovic et al., 2007). In this model, it is assumed that the softening steps due to concrete degradation are avoided considering the reinforcement bars role. It is leading to computing efficiency and robustness.

In order to highlight the effects of material nonlinearities in the SMART RC specimen, the linear solution is first compared to the nonlinear case. In both analyses, zero-displacement conditions are assumed at the base of the building so that no SSI effects are accounted for. Figure 1 shows the corresponding pseudo-spectral accelerations (PSA) at the top of the SMART building.

![Figure 1. Linear and nonlinear SMART structure responses without SSI effects (x and y directions).](image)

Differences are observed between x and y directions because of the non-symmetric character of the building. Even if no significant differences are observed in terms of Peak Ground Acceleration (PGA), eigenfrequencies are slightly shifted to a lower frequency range and the overall impression is that more energy dissipation is introduced in the model.

When SSI is accounted for with MISS3D, similar statements can also be concluded if the PSA are compared at the top of the building (see figure 2) within a linear analysis. However, besides the observed frequency shifting, the present case shows important levels of attenuation not only in terms of amplitude but also in terms of PGA.
If, in addition to SSI effects, nonlinear analysis is performed, the structural response can still be modified. In the following, two nonlinear cases are considered and compared. The first one deals with material nonlinearities only within the SMART structure whereas the second case includes nonlinear phenomena also in the soil surrounding the structure.

The first case has been studied in previous works (Nieto Ferro et al., 2013) by coupling Code_Aster and MISS3D within a Hybrid Laplace-Time domain Approach (Nieto Ferro et al., 2012). Some results involving ultimate damage levels and response spectra are given in Figure 3 in order to illustrate the obtained overall agreement when compared to a full-FEM solution.

Figure 3. Nonlinear SMART building responses obtained using the HLTA and a full-FEM approach.
The second case, where also the soil exhibits nonlinear behavior, is briefly addressed in figure 4. The nonlinear soil constitutive model of Hujeux (1985) is assumed for the FE region of soil close to the structural foundations. The HLTA has been used for this calculation so the rest of the soil, extending to infinity, is assumed to behave linearly.

![Figure 4. Fully linear response with SSI effects and nonlinear SSI responses with a near-field soil exhibiting linear and nonlinear behavior.](image)

Therefore, it can be concluded that accounting for SSI as well as for nonlinearities in both, structure and near-field soil, can significantly modify the RC building response (at least for the SMART soil-structure interaction system) and it is thus recommended to be taken into account if best-estimate assessments have to be carried out.

**DETERMINATION OF SEISMIC FRAGILITY CURVES OF RC BUILDINGS AND EQUIPEMENTS**

Fragility curves express the conditional probability of failure of a structure or component for a given value of seismic input motion parameter, such as PGA, CAV or spectral acceleration Sa. If a lognormal fragility model is chosen, then the fragility curve is entirely defined by two parameters, which are the median capacity $A_m$ and the log-standard deviation $\beta$.

In the nuclear industry, the fragility of a structure or component is defined with respect to its capacity, denoted by $A$. Capacity is defined as the limit seismic load before conventional failure criterion is reached and is modelled by a random variable. Moreover, the capacity is supposed to have lognormal distribution. Then, the fragility curve, that is the failure probability conditioned on ground motion parameter $\alpha$, is given by the cumulative distribution function of capacity $A$, yielding

$$P_f(\alpha) = \Phi\left(\frac{\ln(\alpha / A_m)}{\beta}\right)$$

where $\Phi(.)$ is the standard Gaussian cumulative distribution function. The parameters $A_m$ and $\beta$ can be evaluated by means of maximum likelihood estimation (Zentner 2010) or regression analysis (Zentner et al 2011). Both methods will be applied and compared in what follows. In the latter case, when regression is used, then the structure or component is supposed to fail when damage level $s$ has occurred. Given the model output $Y$ and the threshold $s$, the fragility curve can be expressed as the probability that $Y$ exceeds the threshold $s$ at ground motion level $\alpha$:

$$P_f(\alpha) = P(Y > s | \alpha).$$
The model output \( Y \) is generally considered to be log-normally distributed so that \( \ln(Y) \) is a normal random variable. Adopting a simple seismic demand model of the form \( Y = \tilde{c} \alpha^b \eta \), we can further write \( \ln(Y) = c + b \ln(\alpha) + \varepsilon \) where \( \varepsilon = \ln(\eta) \) is a centred normal (Gaussian) random variable with standard deviation \( \sigma_\varepsilon \) and \( c = \ln(\tilde{c}) \), \( b \) are constants that can be evaluated by means of linear regression. With these notations and using the expression of the median threshold \( s = \tilde{c} A_m^b \), the corresponding lognormal fragility curve reads:

\[
    P_f(\alpha) = \Phi \left( \frac{\ln(\tilde{c} \alpha^b / s)}{\sigma_\varepsilon} \right) = \Phi \left( \frac{\ln(\alpha / A_m)}{\beta} \right)
\]

where \( \beta = \sigma_\varepsilon / b \), with \( \sigma_\varepsilon \), the standard deviation of regression error, and the median capacity as can be evaluated from the threshold value \( s \) as \( A_m = \exp[\ln(s) - c / b] \).

The SMART benchmark model is studied. The output variable of interest is the maximum interstory drift. Two damage levels were considered: light damage \((s=0.003m)\) and controlled damage \((s=0.006m)\). Three mechanical parameters were modeled as lognormal random variables. These are: concrete Young modulus \( E_b \), damping \( \delta \) and additional masses on the slabs of the three levels of the building \( M_{\text{add}} \). Uncertainty is propagated using Hypercube Latin sampling (LHS). We had at our disposal 50 pairs of correlated horizontal ground motion time histories that were provided during the benchmark. These PGA values of these accelerograms ranged between 0.1g and 1.2g. Peak to value variability, close to the one observed for real scenario accelerograms retrieved from ground motion databases, is already included in the signals. According to common practice in the framework of seismic PRA (EPRI), the mean value of the two horizontal PGA values is used as ground motion indicator for the fragility curves.

The \( N_s=50 \) given pairs of accelerograms allowed for the same number of transient seismic analysis. In order to increase the output sample size and thus to improve the quality of the statistic analysis, we perform \( k \) replications, which leads to a total of \( N=kN_s \) simulations. Here, 4 replication have been performed. One model run takes 4 to 7 hours CPU time, according to the case (mechanical parameters and accelerogram). The design of experiment and distribution of the analysis via the Code_Aster user interface allowed performing the whole study in about 150 hours instead of 1500 hours (without distribution). We evaluate fragility curves for the SMART benchmark model using both linear regression and maximum likelihood estimator (MLE). As shown in Table 1, the results obtained wit both methods are very close. The confidence intervals for the parameters estimated by MLE are provided in Table 2. Such confidence intervals can be evaluated by profiled likelihood estimation (Zentner & Poirion 2012). The \( \beta \)-values estimated with the MLE increase with increasing threshold. Likely, the profiled confidence intervals becomes larger. This is due to the lesser failure data that is available for higher thresholds and thus uncertainty in the statistical estimation. The regression method provides a model that does not depend on the threshold. It thus might provide a more robust estimation for higher thresholds.

<table>
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<th>Table 1: Fragility curve parameters determined for the two damage levels by regression and using the maximum likelihood estimator (MLE).</th>
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<tr>
<td>( A_m )</td>
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<td>( \beta )</td>
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Table 2: Confidence intervals for parameters $A_m$ and $\beta$ of MLE determined by profiled likelihood.

<table>
<thead>
<tr>
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<tr>
<td>$A_m$</td>
<td>0.38, 0.44</td>
<td>0.82, 1.0</td>
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<tr>
<td>$\beta$</td>
<td>0.53, 0.7</td>
<td>0.75, 0.95</td>
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The fragility curves for the two damage levels are shown on Figure 5. The maximum likelihood estimator further allows to assess the adequateness of the lognormal model by performing statistical tests. For this purpose, BIC (Bayesian information criterion) and Akaike’s criterion (AIC) are determined for other possible distributions such as Weibull and exponential. These indicators are based on the a posteriori likelihood of the different distribution models and depend on the number of parameters necessary to fit (two for the lognormal and the Weibull model, one for the exponential fragility curve). The resulting fragility curves are reproduced on Figure 6 for light damage (left) and controlled damage (right). It turned out that the lognormal model (blue curve) is the most plausible model for the controlled damage level since it obtained best scores with both the BIC and the Akaike’s criterion. The red curve is the resulting fragility model obtained as a combination of the proposed models, where each model is weighted by its likelihood. For the light damage level, the Weibull model (green curve) performs slightly better but the lognormal model (blue) is very close.

Figure 5. Fragility curves obtained by regression for the two damage levels.

Figure 6. Fragility curves for the two damage levels (left: light damage, right: controlled damage) with 3 different distributions: lognormal, Weibull and exponential. The red curve is the resulting best-estimate model (weighted average of proposed probability models).
APPLICATION TO AN ACTUAL NUCLEAR FACILITY

The studied structure is a reinforced concrete building, 40m x 50m width and length, 40m high. It is constituted of 7 main stories and the top roof, with intermediate partial slabs (Figure 7). The structure is mainly built of slabs, in some areas with beams, a grid of orthogonal walls and a raft foundation of 1.5m thick built on a good soil quality. The building is laterally braced by shear walls, 4 peripheral continuous walls, and other inner walls with several opening. Floor slabs ensure the diaphragm function at each level.

The design of the structure is quite regular regarding seismic design codes. More specifically, horizontal shear walls are quite redundant, with as many as 8 rows of main walls in each direction.

For the purpose of the study, a three-dimensional meshing is created. Walls and slabs are represented by DKT shell elements. Beams and columns are meshed using Euler beam elements. Only the mass of secondary structural elements is included in the model and not their stiffness.

In order to encounter for their material non linear behaviour, shell element are based on multi-layered description of their thickness, and beams as multi-fiber cross-section. Each layer or fiber represents a fraction of the concrete of the elements. Steel reinforcement rebars are represented using 1D truss elements and 2 x 1D grid elements with the required eccentricity. The modelling technique allows encountering for bending and axial non linear behaviour of beams and columns, and in plane and out of plane bending, shear and membrane behaviour of slab and walls.

Concrete behavior is described by the stress-strain curve indicated in Eurocode 2. For multi-layered beam elements, at each concrete fiber the uniaxial strain is used to compute the corresponding stress level, while for shell elements it is the local principal strains $\sigma_1$ and $\sigma_2$ which are used to determine the stress condition. For shell elements stresses are calculated at Gauss points located on the surface of the element, as well as in the thickness within each layer (multi-layered shell element) allowing well predict the bending moment response of the structure.

Steel rebar behavior is also described by uniaxial material law, again using the indications from Eurocode 2. For beam elements, their treatment is similar to concrete fibers. However in shell elements, the uniaxial material model follows the orientation of rebar grids as indicated in the meshing. The eccentricity of grid elements to mid-plane surface is also taken into account in the calculation of bending moment.
Impedance of the soil foundation is calculated using dynamic soil properties of different layers. Soil-structure interaction is then modeled using spring elements lying beneath the raft foundation of the building. Stiffness of spring elements is determined considering a rigid body motion of foundation raft. Calculations are carried out using MISS3D software.

The pushover analyses are then carried out using Code_Aster. Seismic loading is then applied along each principal direction (X', Y') in order to determine the minimum capacity of the structure. Figure 8 plots indicate the capacity curve of the structure as well as the seismic spectrum corresponding to different intensities. Calculating the capacity curve of the structure to its ultimate resistance, characterized by predefined criteria, provides a useful indication to determine the maximum level of seismic loading that the structure can withstand. In addition, for a given seismic spectrum, the distance separating the performance point and the ultimate capacity can be considered as a safety margin. In this case, the results concluded that the structure is capable to withstand ground motions far beyond its design situation.

The presented FEM analysis methodology also provides other results indicating the level of damage within each structural element. These are local information such as maximum tensile and compressive strain values reached for each material (steel and concrete) and more global indications such as inter story drifts.

![Figure 8. Spectrum vs. capacity curve plot for different seismic load intensities](image)

From these results, the following observations can be made:

- At $a_{\text{design}}$, concrete cracking is quite limited to areas close to openings where stress concentration are produced. Strain values are low which indicate no rebar yielding. These reflect the relatively linear behavior of the structure visible on the capacity curve plot Figure 8. At this level, the secant stiffness of the structure has diminished by a factor of 1.4.

- Figure 9 indicates tensile strain mapping of concrete of two most important walls participating in the stability of the building parallel to the pushover analysis direction X'. Each column correspond to the result obtained for a given seismic load intensity (design acceleration level $a_{\text{design}}$, $2a_{\text{design}}$, $3a_{\text{design}}$, $4a_{\text{design}} + 1.5xM$ (M being the magnitude of the earthquake). By filtering strain values to only show values above the tensile limit ($0.03\%$), the mapping can be considered as an illustration of concrete cracking.
For higher seismic loading up to $a_{\text{design}} + 1.5 \times M$ cracking spreads all over the two main walls, resulting stronger softening of the structure as indicated in the capacity curve of the pushover analysis. At $a_{\text{design}} + 1.5 \times M$ the secant stiffness has been reduced by 4 compared to the initial elastic value.

Compressive strain values of concrete remain quite low, with highest values never exceeding $1.4 \, \varepsilon_c$. These are visible at RC beams / wall junction along the corridors.

Tensile strain values, even at $a_{\text{design}} + 1.5 \times M$ remain below the rebar yielding limit, except for a few mesh elements.

These local observations are in agreement with the global behavior of the structure where the mean inter story deflection of the building was calculated at 0.11%.

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<tr>
<th>Wall</th>
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<th>$3 \times a_{\text{design}}$</th>
<th>$a_{\text{design}} + 1.5 \times M$</th>
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Figure 9. Maximum tensile strain in concrete, indicating cracked areas of shear walls

The conclusion of the seismic assessment of this nuclear laboratory building is that its capacity to resist earthquakes is high, since even for ground motions with intensities as high 1.5 times the design magnitude predicted damages remain quite low. And the local verification of structural elements did not exhibit the presence of any abnormal weakness.

CONCLUSION

This paper has presented several examples of advanced techniques developed under the Code_Aster integrated Opensource tool to assess the seismic safety margin of NPP. Soil-structure interaction and material non linearities can be considered together for the assessment of the structures and the determination of the floor response spectra. The techniques to determine the seismic fragility have also been applied to the SMART specimen tested in the CEA Saclay shaking table. Finally an application of the non linear material constitutive laws to a current nuclear facility has shown the possibility of application to complex structure.
REFERENCES


