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SEISMIC ASSESSMENT OF CANDU6 NPPs AFTER FUKUSHIMA ACCIDENT

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ABSTRACT

In response to the Fukushima Daiichi Nuclear Power Plant Accident following the 2011 Great East Japan Earthquake and Tsunami, many nuclear power plants around the world initiated lessons learned programs and performed various seismic assessments in order to determine the seismic margin in the nuclear power plants. This paper discusses the seismic assessment of two CANDU6 Nuclear Power Plants (NPPs) as a follow-up to the Fukushima Daiichi Nuclear Power Plant Accident. Two widely accepted approaches were followed respectively for these two CANDU6 NPPs: one used European Union's Stress Test approach; the other used US EPRI Seismic Margin Assessment (SMA) approach. This paper compares these two approaches and discusses how these two approaches were applied to two existing CANDU6 NPPs. It also summarizes the major findings of the seismic assessments of these two NPPs. It is concluded that the two CANDU6 NPPs assessed meet their safety targets regardless of the approach and method used.

INTRODUCTION

Two different sets of requirements for the seismic assessment in response to the Fukushima Accident were applied to two CANDU6 NPPs respectively: one followed the requirement of European Union's Stress Test (ENSREG 2011), termed as "EU Plant" hereafter; the other followed EPRI SMA approach (EPRI 1991), termed as "EPRI Plant" hereafter .

Stress Test Approach

European Union "Stress Tests" Specifications (ENSREG 2011) define a stress test as a "targeted reassessment of the safety margins of nuclear power plants in the light of the events which occurred at Fukushima: extreme natural events challenging the plant safety functions and leading to a severe accident". The reassessment of the stress test will report on the response of the plant and on the effectiveness of the preventive measures, noting any potential weak point and cliff-edge effect, for each of the considered extreme situations. By their nature, the stress tests will tend to focus on measures that could be taken after a postulated loss of the safety systems that are installed to provide protection against accidents considered in the design. The initiating events considered in a stress test are earthquake and flooding. The current paper discusses earthquake exceeding design basis earthquake only and focuses on the evaluation of the seismic margins as scoped out in the EU "Stress tests" Specifications (ENSREG 2011), e.g., indicate which are the weak points, specify any cliff edge effects according to earthquake severity, and indicate what is the range of earthquake severity the plant can withstand without losing confinement integrity etc. According to the EU "Stress tests" Specifications (ENSREG 2011), the evaluation of the seismic margin should be based on the available information, which could include Seismic PSA, Seismic Margin Assessment (SMA) or other seismic engineering studies to support engineering judgment.

EPRI SMA Approach

The EPRI SMA approach consists of identifying success paths and finding the weakest links along each success path (EPRI 1991). A success path is defined as a set of Structures, Systems, and Components (SSCs) that are required to bring the plant to, and maintain, a safe shutdown condition following a seismic event. It is necessary only to demonstrate the operability and survivability of a success path of SSCs that will bring the plant to a stable condition (either hot or cold shutdown) and maintain that condition for at least 72 hours after the earthquake. Success path determination is based on the selection of components for which it will be easiest to demonstrate an adequate seismic margin; however, the success path shall be compatible with plant operation. The success path of SSCs that is required to perform the safety-related functions corresponds to a subset of the seismic classification list. Success paths are evaluated using the seismic walkdown procedures as specified in EPRI NP-6041. By comparing a success path component's in-structure seismic demand, design, anchorage, and potential seismic interactions to earthquake experience and the seismic-test-based criteria in the SQUG *GIP*, seismically rugged components can be identified and screened out. Screened-out components have HCLPF (High Confidence of Low Probability of Failure) capacities greater than or equal to the *GIP* bounding spectrum peak ground acceleration. Components determined not to meet the *GIP* screening criteria are further evaluated using the Conservative Deterministic Failure Margin (CDFM) method (EPRI 1991). The components having the lowest HCLPF capacity are the weak links, and determine the plant-level HCLPF capacity. Component HCLPF capacity may be increased by design changes, replacement, anchorage modifications, etc., as appropriate. Modifications to plant design, procurement, operations, and maintenance procedures might be necessary to ensure that seismic qualification of success path SSCs is maintained for the life of the facility.

Candu Energy Inc. (Candu) has extensive experience and a long history of performing seismic capacity assessment for nuclear power plants. Before the Fukushima Daiichi Nuclear Power Plant Accident, Candu had successfully completed seismic capacity assessment for several nuclear power plants using either the CDFM method or the Fragility Analysis (FA) method (EPRI 1994). In response to the Fukushima accident, Candu has helped several nuclear power plants to perform seismic capacity assessment to meet their regulatory requirements. This paper discusses the seismic assessment of two CANDU6 NPPs as a follow-up to the Fukushima Accident to demonstrate that the NPPs has adequate seismic margin above their Design Basis Earthquake (DBE). One applied the EU Stress Test Approach, i.e., the EU Plant, and the other followed the EPRI SMA Approach, i.e., the EPRI Plant. As discussed above, the seismic evaluation in the EU stress test approach can be based on the available information, which could include previous Seismic PSA, SMA or other seismic engineering studies to support engineering judgment; while the seismic assessment in the EPRI SMA approach requires the new reassessment of the whole NPP and apply only the SMA method. Due to the space limitation, this paper focuses only on some of the elements of these two approaches, i.e., the selection of the Review Level Earthquake (RLE), the seismic walkdown, and the seismic capacity assessment of the SSCs. The other elements, e.g., the selection of the success path equipment or the Safe Shutdown Equipment List (SSEL) are not discussed in the current paper.

SELECTION OF REVIEW LEVEL EARTHQUAKE

The main objectives of both of the EU Stress Test and the EPRI SMA are to show that the NPP can withstand the Review Level Earthquake (RLE) with high confidence and to identify seismic vulnerabilities and any potential weak links. Seismic capacities of SSCs on the success paths are estimated in terms of the High Confidence of Low Probability of Failure (HCLPF) values. SSCs with high HCLPF values will be screened-out from further consideration. SSCs having HCLPF values lower than the RLE are flagged, and recommendations to improve their HCLPF values will be made if they do not meet their safety targets. This process ensures that the NPP has sufficient seismic "margin".

The RLE is an engineering representation of earthquake ground motion chosen to have a lower probability of exceedance than that of the DBE. The probability of exceedance of the RLE is generally agreed upon by the owner/licensee and the regulatory authority. A probability of exceedance of 1×10^{-4} per year or less is typically selected as the RLE. The RLE is chosen to challenge seismic design of the NPP over the DBE. A RLE will be established at a reasonably high and achievable level based on site seismicity and plant specific design features. Typically, there are two approaches to develop a RLE. One current trend is to specify the RLE in terms of a uniform annual probability of exceedance response spectral shape, the so-called Uniform Hazard Spectral (UHS) shape. This approach requires a site specific Probabilistic Seismic Hazard Analysis (PSHA). The other is to specify the RLE simply in terms of a horizontal PGA higher than that of the DBE. This approach has been accepted by USNRC and Europe Union Requirements for the existing NPP evaluations in the absence of a PSHA

EU Plant RLE

For the CANDU6 NPP that followed the EU Stress Test approach, the RLE was based on the previous SMA performed in the 2000s. It has a Peak Ground Acceleration (PGA) of 0.33g and has spectral shape similar to the standard NUREG/CR-0098 Ground Response Spectrum (GRS). Although this RLE of 0.33g has already exceeded the plant's DBE that has a PGA of 0.2g, in order to determine the capacity of the plant to sustain the maximum level of earthquake possibly beyond even the 0.33g, the targeted level of earthquake considered in the seismic evaluation is taken as having a PGA of 0.4g based on Candu's knowledge and experiences with the plant's seismic design.

EPRI Plant RLE

For the CANDU6 NPP that followed the EPRI SMA approach, Candu recommends a RLE with 0.3g PGA and a spectral shape that is the same as NUREG/CR-0098 median shape based on the following considerations:

1. According to the descriptions in Safety Analysis report, the site is considered geologically stable and free of any active faults.
2. The spectral shape of the plant's DBE is similar to those of most of the NPP sites in Central and Eastern United State (CEUS). Based on NUREG/CR-0098, the use of 0.3g PGA RLE with NUREG/CR-0098 median rock spectral shape was chosen for a number of NPPs in CEUS and Canada.
3. The IPEEE RLEs of the CEUS NPPs are generally defined as 1.4-1.67 times of the DBE. The frequency of interest for the SSCs in a nuclear power plant ranges from 2-10 Hz. The average spectral acceleration for EPRI Plant DBE is about 0.43g over the frequency range of 2-10 Hz. The peak spectral acceleration for 5% NUREG/CR-0098 median rock spectrum (recommended RLE) is 0.64g, which is 1.49 times of the average spectral acceleration of 0.43g. This value falls into the factor range of 1.4-1.67.

The RLE with 0.3g PGA for the EPRI Plant was also selected based on Candu's experience from other similar CANDU6 NPPs. Candu's experience is based on CANDU6 plant specific design features, international practice for the selection of RLE, various seismic capacity assessment experiences for several CANDU6 sites with higher seismicity than the EPRI Plant discussed in this paper. The selected RLE with 0.3g PGA for the EPRI Plant is considered to have a probability of exceedance lower than or equivalent to 1×10^{-4} per year.

RLE-BASED FLOOR RESPONSE SPECTRA

Once the RLE is determined, it is necessary to develop either preliminary or final estimates of the seismic demand based on the RLE, i.e., the RLE-based Floor Response Spectra (FRS), since it is one of the factors which must be considered during the screening process of the seismic walkdown and must be used for the subsequent seismic capacity assessment. The seismic demand based on the RLE can be obtained by either scaling previously performed design or reevaluation building response analyses or by performing new building response analyses. For the seismic margin assessment, considerable benefit in reducing computed demand levels is likely to be achieved by performing new SMA building response analyses, particularly for soil sites where the design analyses are likely to be excessively conservative in their original design. As noted in the EPRI NP-6041 and verified in various SMA performed by Candu, the REL-based FRS computations are one of the primary locations where substantial margin has been shown to exist and is one of the most beneficial spots to attack in order to demonstrate greater margin. In many cases, it is expected that the RLE-based FRS will often be less than the DBE-based FRS when the RLE is less than about 1.5 to possibly 2 times the DBE because the DBE-based FRS are likely to contain a number of sources of conservatism relative to the SMA criteria.

EU Plant RLE-based FRS

For the CANDU6 NPP that followed the EU Stress Test approach, the RLE-based FRS was based on the previous SMA and was derived in the 2000s. It was observed that the 0.4g PGA based FRS are close to those based on the 0.2g DBE, i.e., the seismic responses based on the GRS with 0.4g PGA are close to the FRS based on the DBE with 0.2g PGA in terms of amplitudes and shapes. One of the reason is because the RLE-based FRS were generated using larger damping for the primary structure, a more detailed and less conservative Soil-Structure Interaction (SSI) analysis, and 15% peak clipping for the narrow peaks. The other reason is that the EU Plant DBE-based FRS had utilized the FRS from other similar CANDU6 plants for seismic qualification and therefore are inherently conservative.

EPRI Plant RLE-based FRS

For the CANDU6 NPP that followed the EPRI SMA approach, the Reactor Building (RB) RLE-based FRS were generated through new seismic analysis to reduce the seismic demand. The RLE-based FRS for other buildings were obtained using the scaling approach and engineering judgment based on the DBE-based FRS. For the EPRI SMA, the scaling approach is very cost effective over performing new building analyses. However, they are approximate and is conservative to cover uncertainty. It was found that the Reactor Building 0.3g RLE-based FRS are close to the existing RB FRS which is an envelope of the DBE-based FRS and the standard reference CANDU6 RB FRS that are based on a standard GRS with a PGA of 0.2g and a standard spectral shape. The 0.3g RLE-based FRS for the EPRI Plant is close to the 0.4g PGA based FRS for the EU Plant in terms of amplitudes for the reactor building. This is because the EPRI Plant is founded on very hard rock for which the soil-structure interaction effect is insignificant given the standard shape GRS that is high in low frequency but low in high frequency. It is also discovered that the RLE-based FRS for other buildings obtained through the scaling approach, when used in the seismic margin assessment, do not give the seismic margin expected for some SSCs in these buildings. Therefore, to increase the HCLPF capacity of these SSCs, it was recommended to regenerate the RLE-based FRS for these buildings through new seismic analysis.

SEISMIC WALKDOWN

For the seismic evaluation of the two CANDU6 NPPs discussed in this paper, a key common element is the seismic walkdown, which was carried out to identify any as-built construction deviations and to assist in seismic capacity analysis of components and structures. The seismic walkdown followed the procedures in SQUG GIP and EPRI NP-6041. These procedures have been developed using earthquake experience database, seismic qualification test, and past seismic assessments of NPPs.

The objectives of the seismic walkdown are to identify all equipment that is expected to have sufficiently high seismic capacity and to clearly define failure modes for components not expected to have high seismic capacity. The seismic walkdown team or the Seismic Review Team (SRT) reviews and gathers detailed information and measurements on equipment and structures for performing seismic capacity evaluations. The SRT observes and records any deficiencies, e.g., missing anchor bolts, loose mounting of relays, excessive cracking of concrete that can reduce the seismic capacity of components. Significant spatial interactions, e.g., nonseismically qualified equipment located above or beside seismically qualified equipment, heavy equipment, or ceiling fixtures etc., are identified during the seismic walkdown. Areas with potential for seismically induced fire, e.g., storage of flammable liquids or gases, are also noted. The SRT also evaluates fire protection systems in the plant for inadvertent actuation, e.g., potential seismically induced flooding and capability to mitigate seismically induced fires, and assess the condition of other piping systems whose failure could cause seismically induced flooding.

Major Seismic Walkdown Findings

For the two CANDU6 plants discussed in this paper, in general, the DBE qualified mechanical equipment and distribution systems including valves were judged to be seismically rugged and screened at 1.2g spectral acceleration subject to further anchorage evaluation and natural frequency of the equipment. This is because these mechanical components are typically located at lower elevations of the building where seismic response is low and because the normal operating loads imposed on the mechanical components and their anchorage could be more severe than seismic loads. For mechanical components located at high elevations in the RB or other buildings, further screening and/or evaluation is required. The instrument racks and I&C panels are well constructed. Most of the SQ instrument racks could be screened at 1.2g spectral acceleration subject to minor housekeeping actions. All the safety related structures are not screened out during the seismic walkdown due to the complexity of the structures. Major components of the reactor systems such as Calandria, pressure tubes, the Calandria tubes, and shutoff rods, etc. are not covered in EPRI NP-6041. Therefore, screening of these items relies upon the design reports and seismic capacity calculations.

Comparing with other CANDU6 NPPs, it was observed that the EPRI Plant has relatively better seismic design since it is the most recent built CANDU6 NPP incorporated with mature seismic good practice. The wide use of embedded parts rather than anchor bolts for the anchorage of most of the SSCs has increased the seismic ruggedness of the NPP significantly.

SEISMIC CAPACITY CALCULATION

Seismically rugged SSCs were screened out from any further consideration in the seismic capacity calculations during and after the seismic walkdown. For the remaining SSCs, i.e., the SSCs cannot be screened out, seismic capacity calculations are required.

A commonly used value, which describes the seismic capacity of a component or structure, is the High Confidence of Low Probability of Failure (HCLPF) value. The HCLPF represents, with a 95% confidence, that the probability of failure of a component or structure does not exceed 5%. The results of the analysis can be presented in terms of specified variables, e.g., PGA or ground response spectral acceleration at a specified frequency. In general, there are two methods for calculating the seismic HCLPF capacity, one is the Conservative Deterministic Failure Margin (CDFM) method and another is the Fragility Analysis (FA) method. The CDFM method is documented in EPRI NP-6041 and the FA method is documented in EPRI TR-103959. In addition to these two methods, the EU Stress Test Approach also allows the use of other seismic engineering studies to support engineering judgment.

The CDFM method is a deterministic approach based on seismic demand of 84% non exceedance probability. The seismic capacity calculated using the CDFM method can be expressed as HCLPF84. The CDFM method estimates the seismic capacity in terms of HCLPF value based on the following equation:

$$\text{HCLPF84} = (\text{Seismic Capacity})/(\text{RLE-based Seismic Demand}) * \text{RLE} \qquad (1)$$

In the FA method, median seismic capacity of 50% non-exceedance probability or mean capacity is applied. The seismic capacity calculated using the FA method can be expressed as HCLPF50. The HCLPF capacity in FA method is calculated by:

$$\text{HCLPF50} = A_m \exp\,[1.65(\beta_R + \beta_U)] \qquad (2)$$

where A_m is the median seismic capacity in terms of a ground-motion parameter (e.g., PGA and spectral acceleration); β_R is logarithmic standard deviation due to randomness; and β_U is logarithmic standard deviation due to uncertainty.

According to the Seismic Fragility Applications Guide Update in EPRI 1019200, the definition of HCLPF is updated to remove the distinction between HCLPF84 and HCLPF50 if the PSHA have included the response spectral peak and valley variability βrs as part of the aleatory variability. If βrs is included when developing seismic hazard estimates as a function of annual frequency of exceedance (AFE), the CDFM seismic HCLPF capacity and the HCLPF capacity calculated from the FA method should be essentially the same (EPRI 2009).

Because of different SSI effects among different CANDU6 NPPs as well as the different approaches used to derive the RLE based seismic demand, the HCLPF values for an exactly the same SSC could be different due to the differences in the RLE based FRS. This is also evidenced by the seismic capacity evaluation of the two CANDU6 NPPs discussed in this paper. Although the EPRI Plant has relatively better seismic design in general since it is a newer plant, the seismic capacity evaluation results shows the HCLPF values for some SSCs of the EPRI plant are lower than those of the EU plant. That is partially because the RLE-based FRS for these two plants were derived differently, especially for buildings other than the RB, for which the EPRI plant used scaling approach as it is cost-effective in deriving FRS, while the EU plant used other approach to remove the conservatism inherited in the scaling approach.

Another finding is that, typically, for the HCLPF calculation for the same SSC in the same plant, the use of the FA method, although more involved, gives an improved HCLPF value over the one obtained through the CDFM method. A sample HCLPF calculation for the containment building of one typical CANDU6 NPP is given below using both the CDFM method and FA method respectively.

Sample Comparison of the HCLPF Calculation between CDFM and FA

In this Section, the seismic capacity assessments of a typical CANDU6 containment building are performed to illustrate the applications of CDFM and FA methods. For a typical CANDU6 NPP, the Reactor Building (RB) consists of a prestressed reinforced concrete cylindrical Containment Structure (CS) and Internal Structure (IS). The CS and the IS are supported on a 5 ft (1.6m) thick prestressed concrete slab. The base slab is founded on a sub-base slab of reinforced concrete. A sliding membrane is provided between the two slabs to facilitate radial deformation of the base slab under prestressing and shrinkage loads. A central shear key and a set of radial shear keys provide the required stability under seismic and wind loads during construction. These shear keys provide stability against large lateral (horizontal) load acting on the CS. The base slab, containment wall, ring beam and the upper outer dome are prestressed to minimize leakage. The inner dome, which serves as a reservoir for the dousing water is not part of the containment boundary, is made of reinforced concrete.

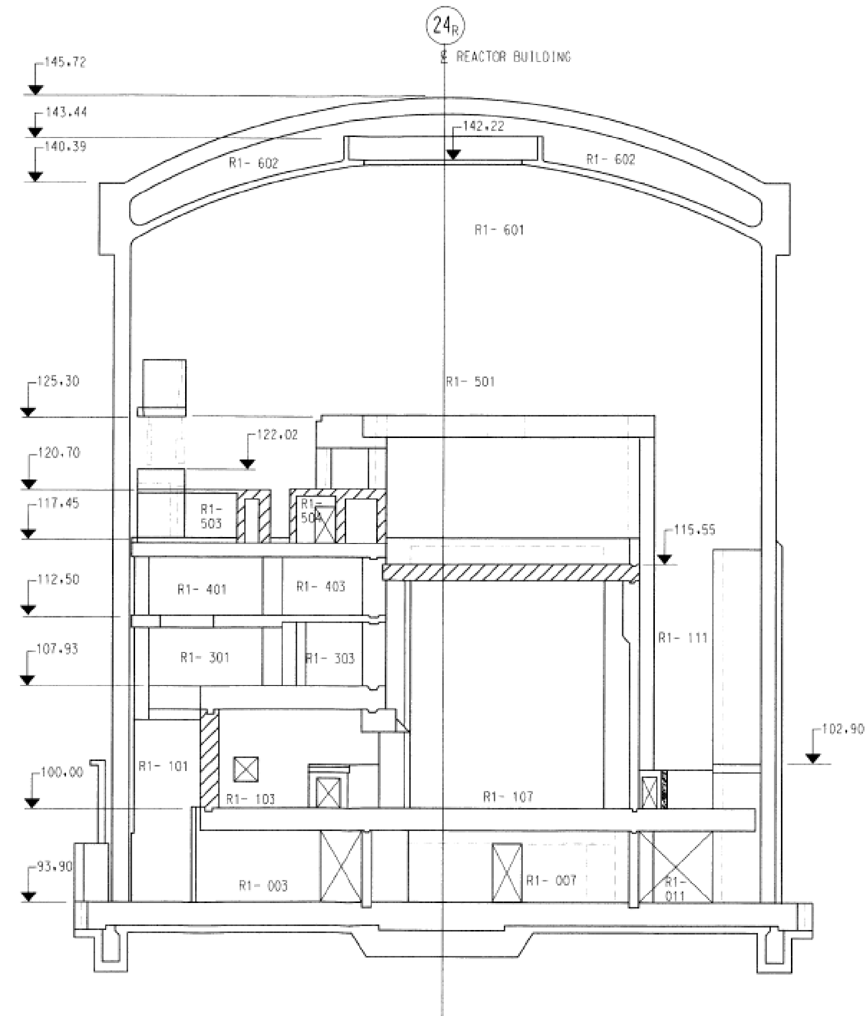

Figure 1 A typical CANDU6 Reactor Building

As shown in Figure 1, the CS has an outside diameter of 143 ft (43.6 m) and a height of 139 ft (42.4 m). The dome is approximately 20 ft (6.1 m) high and is of a torispherical shape. At the junction between the cylindrical structure and the dome, a circular ring beam provides transition from the perimeter wall to the dome. The containment wall is 3.5 ft (1.1 m) thick. For a practical design, both the flexural and shear strengths were evaluated for the containment structure. The containment shell was reinforced by placing additional reinforcing steel and by thickening the wall locally at major penetrations such as the equipment hatch and personnel airlock. As a result, stresses at these locally reinforced sections are low such that failures at these locations do not govern.

Under seismic events, the seismic loads lead to a distributed meridional membrane and a tangential shear stress. The maximum bending moment and base shear occur near the base of the containment wall. Based on the detailed review of seismic forces and reinforcing steel along the height of the containment wall, it is concluded that the critical failure modes of the CS under seismic loading conditions are shear failure and flexural bending failure near the base. Furthermore, based on various studies and an approximate calculation for flexural capacity of the containment wall, it is judged the governing failure mode is the shear failure for the super-structure.

For simplicity of illustration, the seismic input for both CDFM and FA seismic capacity evaluations of CS is assumed the same in this sample calculation. However, for actual analyses, different sets of FRS at the base slab are needed to determine different seismic demands for CDFM and FA methods, respectively. The evaluation results are summarized as following,

For the seismic capacity evaluation of the containment building using the CDFM method, as it is shown in Table 1, the ratio of demand to capacity approaches to 1 as the PGA scaled from 0.3g to 0.81. i.e., HCLPF = 0.81g.

Table 1 Seismic Demand vs. Capacity by CDFM

Seismic input in PGA (g)	Shear Demand, Vd (kN)	Shear Capacity, Vu (kN)	Vu/Vd
0.30	1.50E+05	4.35E+05	2.91
0.81	4.03E+05	4.03E+05	1.00

For the seismic capacity evaluation of the containment building using the FA method, median seismic demand and median strength capacity should be used to determine the strength factor as shown in Table 2.

Table 2 Seismic Demand vs. Capacity by FA

RLE PGA (g)	Median Seismic Demand, Vm (kN)	Median Shear Capacity, Vc (kN)	Strength Factor, F_S (Vc/Vd)
0.30	1.50E+05	5.58E+05	3.73

The strength factor, together with other factors, is used to calculate the median capacity of the containment building, i.e., A_m in Equation (2). These other factors include the Inelastic Energy Absorption Factor (F_μ) related to the ductility of the structure, Spectral Shape Factor (F_{SS}) related to the difference in the response spectra used in for the design and the RLE, Damping Factor (F_D) to account for conservatism in the damping values used for the design, Modeling Factor (F_M) to account for either conservatism or unconservatism in the structure model that was used for the seismic design, Modal Combination Factor (F_{MC}) to account for either conservatism or unconservatism in the modal combination method used in the design analysis, Earthquake Component Combination Factor (F_{EC}) to account for either conservatism or unconservatism in the earthquake components combination method used in the seismic design, and Soil-Structure Interaction Factor (F_{SSI}). For each of these factors, the associated randomness and uncertainty parameters are also calculated and combined. The calculated median seismic capacity A_m, the composite logarithmic standard deviations due to randomness $\beta_{R,}$ and composite logarithmic standard deviations due to uncertainty β_U are given in Table 3. They are then substituted into Equation (2) to obtain the HCLPF of 1.03g.

Table 3 HCLPF Calculation by FA

Median Seismic Capacity, A_m (g)	Randomness Parameter (β_R)	Uncertainty Parameter (β_U)	HCLPF (g)
3.33	0.25	0.46	1.03

As observed in this sample comparison, the use of the FA method, although more involved, gives an improved HCLPF value over the one obtained through the CDFM method. However, this increase of HCLPF value by using FA instead of CDFM is not always guaranteed.

CONCLUSION

Both the EU Stress Test approach and EPRI SMA approach have been applied separately to the seismic evaluation of two CANDU6 NPPs successfully in response to Fukushima Daiichi Nuclear Power Plant Accident. Different methodologies in seismic capacity assessment of SSCs are discussed. It is concluded that the two CANDU6 NPPs assessed can meet their safety targets regardless of the approach and method used.

In the detailed seismic capacity assessment of the SSCs of the NPPs, both CDFM method and FA method are accepted as the methodology for evaluation of seismic capacity. Both methods have been used in the seismic assessment of the CANDU6 NPPs in the past. CDFM method is relatively simpler than the FA method. However, the median capacity cannot be obtained directly from the CDFM method and for the HCLPF calculation for the same SSC in the same plant, the use of the FA method, although is more involved, can provide median capacity, randomness and uncertainty parameters, and can typically give an improved HCLPF value over the one obtained through the CDFM method.

As noted in the EPRI NP-6041 and verified in various SMA performed by Candu, the REL-based FRS computations are one of the primary locations where substantial margin has been shown to exist and is one of the most beneficial spots to attack in order to demonstrate greater margin.

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