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APPLICATION OF SEISMIC MARGIN ASSESSMENT ON NATIONAL RESEARCH UNIVERSAL REACTOR FACILITY

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Abstract

As part of an Action Plan to ensure that the lessons learned from the Fukushima accident are applied in Canada, a Seismic Margin Assessment (SMA) of the National Research Universal (NRU) reactor facility was carried out. The existing multipurpose nuclear facility was built in the 1950's when limited seismic knowledge and less reinforcement details were required in the design. Several challenges in applying a SMA on the NRU facility are presented. The methodology used for the SMA is based on EPRI-NP-6041. Alternative criteria applied in SMA are discussed. The issues include the selection of parameters of ground motion to determine the seismic margin earthquake level. A parametric study on stiffness and load conditions was conducted to better simulate the NRU structure. The ageing effects on the compressive strength of concrete material were considered through non-destructive testing. The ductility factor based on the National Building Code of Canada (NBCC) 2010 is applied in the SMA to consider nonlinear behavior of the structure. Alterative criteria are carefully examined and the deviation from the criteria provided in EPRI-NP-6041 is discussed. It is recommended that attention should always be paid to scrutinize any alternative methodologies and justifications should be provided in the selection of assessment criteria and parameters in SMA.

Keywords: nuclear; SMA; building; dynamic; code

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1. Introduction

On March 11, 2011, a magnitude 9.0 earthquake struck off the coast of Japan and caused a large tsunami, resulting in the loss of thousands of lives and half a million homes. The accident at Tokyo Electrical Power Company's (TEPCO) Fukushima Daiichi nuclear power plant caused three nuclear meltdowns and the release of radioactive material. It is the largest nuclear disaster since the Chernobyl disaster of 1986 and the second disaster to be given the Level 7 event classification of the International Nuclear Event Scale.

After the Fukushima accident, the Canadian Nuclear Safety Commission **(**CNSC**)**, an independent federal government agency that regulates the use of nuclear energy and material in Canada, established a four-year Action Plan to ensure that the lessons learned from the Fukushima accident are applied in Canada to enhance the safety of nuclear facilities.

One project currently undertaken at the Canadian Nuclear Laboratories (CNL) is the assessment of the seismic margin of nuclear facilities taking into account lessons learned from the Fukushima accident. In this paper, the application of seismic margin assessment (SMA) of the National Research Universal (NRU) reactor facility is presented and the alternative criteria applied in SMA are discussed.

The NRU facility is composed mainly of an above-ground steel super-structure and a reinforced concrete basement sub-structure. The structure houses the reactor vessel, experimental facilities, various equipment, and working areas. Since most of the safety-related Systems, Structures and Components (SSC's) are within the concrete basement structure, the seismic assessment of the concrete basement structure is of particular concern and is discussed in this paper.

2. Methodology

Normally a nuclear plant is designed under Design Basis Earthquake (DBE) in which adequate assurance of nuclear safety is provided against an earthquake-induced failure. However, the convention design using DBE does not quantify the actual margin to failure, nor does it provide adequate information on the seismic risk or vulnerabilities. Beyond Design Basis Earthquake (BDBE) is then introduced to consider the earthquake at very low probabilities of exceedance during the lifetime of a nuclear plant. The seismic risk analysis under BDBE can quantify the redundancy and defence-in-depth of the SSCs.

There are two approaches available for seismic risk analysis for BDBE, i.e. the seismic probabilistic safety assessment (SPSA) and the seismic margin assessment (SMA). Since the SMA approach is more efficient, it is widely used in engineering practice. The SMA describes the additional seismic margin plants have, by virtue of their conservative design, to withstand earthquakes larger than the DBE. This margin can be defined in terms of the High Confidence (95%) of Low Probability (5%) of Failure (HCLPF) capacity of each critical SSC or the overall HCLPF of the plant.

The SMA method is used in the current investigation to study the concrete basement structure. The concept of EPRI-NP-6041 SMA method [1] is adopted, which consists of identifying both the success paths and the weakest links along each success path. The SMA was carried out in two stages:

Stage 1:

Determine the stress level of the structural elements forming the basement structure at DBE level as a baseline case.

The methodology to compute the capacity of structures and components is based on the conservative deterministic failure margin (CDFM) approach as indicated in EPRI-NP-6041 [1]. The seismic margin is calculated based on the capacity-to-demand ratio for the inelastic response of DBE. The procedure is described as follows:

The stresses obtained from the analysis are identified as the demands for the seismic and non seismic loads, i.e. D_s and D_{ns}.

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Calculate the capacity of structures and components, i.e. C. It is noted that reduction in the capacity due to concurrent seismic loading ΔC_s must also be considered.

Calculate the scale factor by which the DBE can be scaled for the seismic margin earthquake (SME) level

$$FS_{inelastic} = \frac{C - D_{ns}}{K_{\mu}(D_s + \Delta C_s)} \tag{1}$$

Stage 2:

Establish the review level condition for the beyond design basis earthquake to ensure that the basement structure will not suffer any major deformations.

At this stage, several trials for the SME magnitude were carried out to identify the success paths and the weakest links. The trial SME magnitude should be set sufficiently high so that the capacity of some elements may be lower than the demand from the trial SME. Then the weak elements were further investigated to confirm their seismic margin capacity.

It is noted that the general criteria of the CDFM approach are outline in EPRI-NP-6041. The CDFM approach is based on the evolution of seismic criteria used in earlier seismic margin reviews. Since the conservatism can be introduced at various steps of the evaluation, the fundamental intention of the evaluation is to achieve HCLPF as expected. Also, the use of alternative criteria is allowed as long as the alternative criteria can approximately achieve the same HCLPF indicated in EPRI-NP-6041.

In fact, uncertainties always exist when dealing with an existing structure. The uncertainties may come from the lack of seismic performance information for the old structure in which seismic design is not fully considered, or change of stiffness and material properties as a result of the loading history and ageing affects, etc.

In addition, the inconsistency of past seismic engineering practice combined with improvement of seismic knowledge makes the SMA difficult to achieve consistent goals in HCLPF. The EPRI-NP-6041 document provides the guideline of the SMA method but does not restrict the use of alternative criteria to achieve the goal. In the current SMA, while the philosophy of EPRI-NP-6041 SMA method is maintained, the alternative criteria applied in SMA are examined and further discussed in the paper.

3. Seismic Ground Motion

The NRU facility is located in the Canadian Nuclear Laboratories - Chalk River site. Most seismic events in the Chalk River site have been small to moderate in magnitude. Initially, the DBE parameters used for seismic analysis of SSCs were derived based on the method recommended by the National Building Code of Canada (NBCC) of 1990 [2]. The probability of occurrence of DBE was characterized by a return period of 1000 years (10^{-3}/y).

The ground motion parameters were determined based on a probabilistic approach as follows:

At bedrock level:

Peak Ground Acceleration (PGA)	0.24 g
Peak Ground Velocity (PGV)	130 mm/s

It is noted that the definition of DBE was redefined in CSA N289.3-2010 [3]. The probability of exceedance of $1\text{x}10^{-4}$ per year is specified for DBE in new design. However, it is still allowed that DBE for some of the old plants to be based on an estimated probability of exceedance of 1×10^{-3} per year or established deterministically (i.e., without probabilistic measures).

The latest site-specific uniform hazard spectra (UHS) were developed for the Chalk River site based on NRCAN-NBCC methodology. The response spectral acceleration values were determined using model SECan_2015clC.model (the same as model used for determining NBCC-2015 [4] localities and grid values).

Fig.1 shows the comparison of response spectra based on DBE and SECan_2015c1C.model for return periods of 10000 and 2500 years, which is corresponds to annual probabilities of 0.0001 and 0.000404. All the response spectra shown in Fig.1 are at 5% damping. The hazard model values for the SECan_2015c1C.model are presented in Table 1.

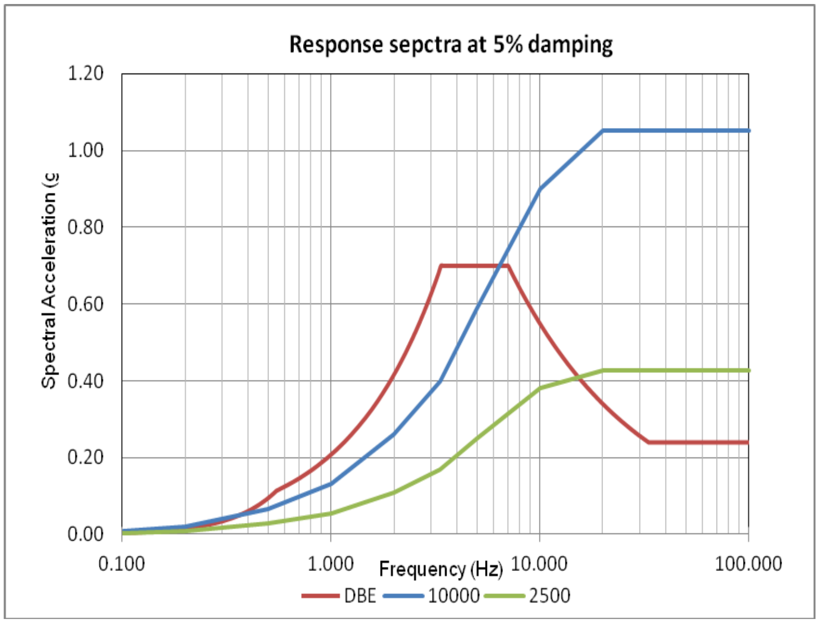

Fig. 1 – Comparison of Ground Response Spectra

Table 1 - GSC 5th Generation seismic hazard model values for NPP facilities

	Conversion factor C to A		1.0816	0.8186	0.6450	0.5754	0.5292	0.5379
Location	Probability	Percentile	0.05	0.1	0.2	0.3	0.5	1
Chalk River	0.0001	mean	1.0506	0.8992	0.5861	0.3987	0.2611	0.1307
Location	Probability	Percentile	0.05	0.1	0.2	0.3	0.5	1
Chalk River	0.000404	mean	0.4284	0.3812	0.2506	0.1694	0.1101	0.0557

	Conversion factor C to A		0.5545	0.5881	0.6638	0.9917	0.5937		
Location	Probability	Percentile	2	5	10	PGA	PGV	Latitude	Longitude
Chalk River	0.0001	mean	0.0646	0.0187	0.0072	0.5586	0.2494	46.052	-77.363
Location	Probability	Percentile	2	5	10	PGA	PGV	Latitude	Longitude
Chalk River	0.000404	mean	0.0273	0.0076	0.0032	0.2474	0.1019	46.052	-77.363

Site Class A (Hard Rock), Vcs30=2000m/s), Annual probabilities of 0.0001 and 0.000404, for mean

Spectral and Peak Acceleration in units of g, peak velocity in m/s
Values determined using model SECan_2015c1C.model (same as model used for determining NBCC 2015 localities and NBCC2015 grid values)

It is apparent that inconsistencies exist between the response spectra shown in Fig.1. The DBE curve is originally developed based on CSA N289.3-M81 [5] and still consistent with CSA N289.3-10 [3]. In accordance

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with CSA N289.3-M81, the fundamental parameters for the Design Basis Seismic Ground Motion (DBSGM) are peak acceleration, velocity and displacement. The amplification factors are used to create the standard response spectra which are based primarily on strong ground motion data available up to and including the San Fernando earthquake of February 9, 1971.

It is noted that the parameters of peak acceleration, velocity, and displacement used for the hazard analysis were for the specified probability level in CSA N289.3-M81 and N289.3-10. The response spectrum used for engineering design was constructed based on the standard spectral shape. The resulting spectrum did not necessarily have a uniform probability of exceedance at all periods.

The methodology of specifying ground motions in NBCC 2015 [4] is based on UHS. The UHS is a representation that plots, for each spectral period, the spectral amplitude that has a specified probability of exceedance. Thus the probability of exceeding a UHS is constant (or uniform) as a function of period.

The EPRI-NP-6041 document provides four alternatives to select the seismic margin earthquake level. One alternative is to specify the SME in terms of UHS which is consistent with NBCC 2015. This option is very appealing because it can produce HCLPF statements in terms of annual frequency of exceedance as oppose to PGA. However, the alternative ties the SMA and seismic hazard assessment together in which the disagreement in hazard issues may be introduced. Another difficulty in using this alterative is that UHS reflects uncertainty in the seismological parameters and the randomness in the response spectral shape. The meaning of HCLPF SME level statements may be confusing because the additional conservatism exists in UHS.

Another option is to specify the SME simply in terms of the horizontal peak ground acceleration, velocity, and displacement. The arguments may be that the parameters do not fully reflect the seismic hazard as understood based on the current seismic knowledge. However, the alternative to use PGA is still consistent with DBE based on CSA N289.3-10 and the current regulated practice for the facility. Therefore, the alternative to use PGA parameter is adopted in the study. The standard response spectrum shape is based on CSA N289.3-10.

4. Structural Model

A 3D finite element model was created using STAAD.Pro software [6]. A linear dynamic analysis using the response spectrum method was utilized in the seismic analysis of NRU facility. The dynamic analysis procedure is in accordance with CSA-N289.3-10 and EPRI SMA methodology.

Fig.2 shows a 3D model view of the NRU concrete basement structure. The structure is approximately 302-ft long by 160-ft wide by 123-ft high. Beam and plate elements are used to model the beams/columns and slabs/walls, respectively. The size of the plate element after meshing is approximately 3ft x 3ft. The steel structure above the main floor is not included in the model. However, the loads transferred from the steel structure to the concrete basement structure are considered through the decoupling.

The structural model used in CDFM should be best estimate (at media) plus uncertainty variation in frequency. A parametric study is carried out to examine and determine the influence of the mass and stiffness on the dynamic response of the basement structure.

In accordance with EPRI-NP-6041, the SME loadings are only combined with normal operating loads without extra conservatism. In accordance with CSA N291-08 [7], all possible live loads should be considered; the maximum live load reduction shall not be more than 50%. Therefore, two load conditions are considered, i.e. the full live loads and live loads with 50% reduction to study the effect of operating loads on the dynamic response of structure.

In accordance with ASCE 43-05 [8], the element stiffness shall be modified using the effective stiffness factor for the dominant response parameters. Since the actual stiffness of the existing structure varies due to past stress level, loading condition, and material ageing, etc., the gross section and effective stiffness section are considered to assess the stiffness impact on the dynamic response of structures. The effective stiffness property is to be used as a fraction of the gross section property as shown in Table 2 in accordance with CSA A23.3-04 [9]. Table 3 summarizes the models created for the current parametric study.

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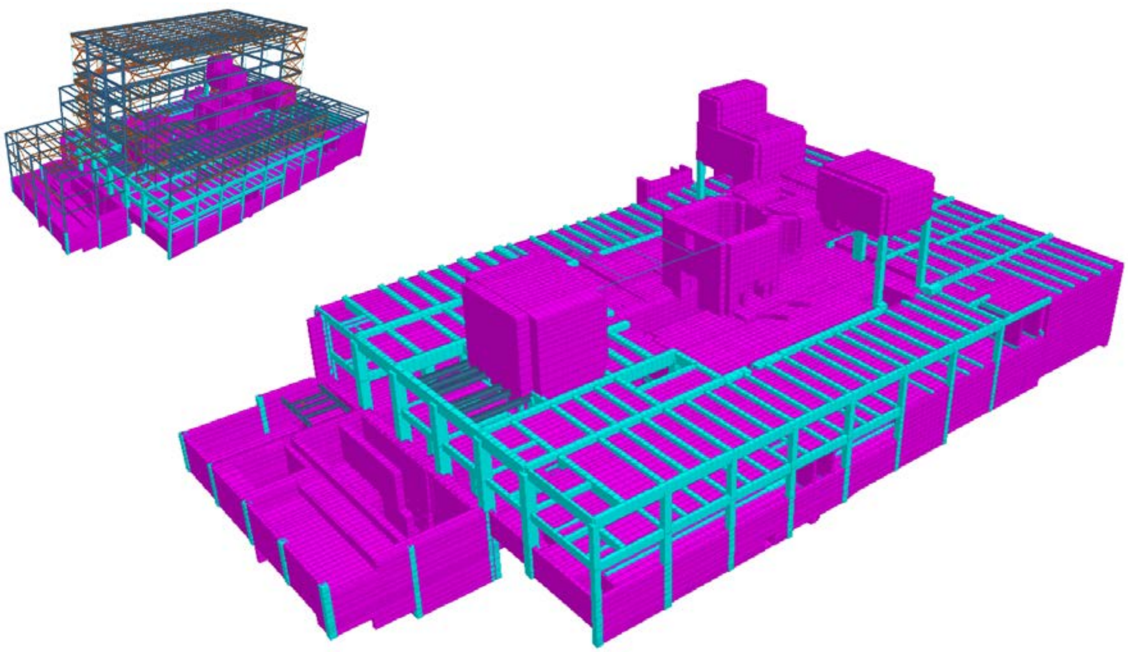

Fig.2 - 3D model view of NRU concrete basement structure

Table 2 - Section properties for analysis

Element type	Effective property	
	Flexural	Shear
Beam	$I_e=0.4I_g$	$A_e=A_g$
Column	$I_e=0.5I_g$	$A_e=A_g$
Wall	$I_e=0.6I_g$	$A_e=0.6A_g$
Slab	$I_e=I_g$	$A_e=A_g$

I_e = effective moment of inertia
I_g = moment of inertia of gross concrete section
A_e = effective area of section for shear forces
A_g = gross area of section

Table 3 - Analysis models for parametric study

Parameters	Model 1	Model 2	Model 3	Model 4
Stiffness	Effective	Effective	Gross	Gross
Operating Load	100%	50%	100%	50%

Table 4 presents the natural frequencies of the first 10 modes for the different models. The results of the analysis show the effect of operating loads and stiffness on the natural frequencies. The natural frequencies are shifted to the higher values with increase of stiffness and reduction of operating load. Except for Model 1, the spectral acceleration for the fundamental natural frequency is within the peak value of 0.7g between 3.339 Hz – 7.0 Hz. The response spectral acceleration of Model 1 based on the fundamental natural frequency is slightly less than 0.7g.

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Using the gross section stiffness provides the maximum stiffness in structural analysis. Consequently, the higher fundamental natural frequency is obtained. On the other hand, the structure performs non-linearly when the stiffness is less than the effective stiffness. Since the ductility factor has been used in the SMA to account for the non-linear behaviour, the effective stiffness shall be consistent with the ductility factor for the conventional shear wall construction as indicated in NBCC 2010 [10]. It is noted that CSA N289.3-10, NBCC 2010 and CSA A23.3-04 are applicable to the NRU SMA project.

Table 4 - Natural frequencies for the first 10 modes

Mode	Model 1	Model 2	Model 3	Model 4
1	3.298	3.499	3.958	4.196
2	4.483	4.752	4.601	5.606
3	4.546	4.987	5.607	5.880
4	4.599	5.198	6.027	6.425
5	4.679	5.221	6.051	6.572
6	4.841	5.374	6.191	6.726
7	4.874	5.397	6.200	6.893
8	4.957	5.407	6.219	6.912
9	5.057	5.490	6.571	7.032
10	5.391	5.604	6.731	7.067

Also, the fundamental natural frequency based on the effective stiffness is still within the peak. The shift of natural frequency is not significant to the correspondent spectral acceleration in dynamic analysis. Therefore, the effective stiffness based on the current CSA A23.3-04 is used in the final model.

The live loads indicated on the applicable drawings of the NRU structure are the maximum operating loads which are allowed to put on the floors. The loads are useful to check the beams and slabs for the worst loading scenario. However, only the realistic operating loads are required for the SMA without extra conservatism. After further evaluation of the existing load condition, using 50% reduction in full live load matches the true load condition. Also the fundamental natural frequencies for both load conditions are almost at peak with only slight differences. The shift of natural frequency is not significant to the corresponding spectral acceleration in dynamic analysis. As a result, 50% reduction in full live load is used as the operating load in the dynamic analysis.

In summary, the parametric study provides an overview of the dynamic response of the structure under the seismic load. The performance of the structure under a real earthquake should be within the upper and lower bounds of mass and stiffness. In the current SMA, Model 2 based on the effective stiffness and 50% reduction of live load is used as a baseline model for the study.

5. Concrete Ageing

The concrete compressive strength increases with age over the first several years. In accordance with EPRI-NP-6041, the increase of compressive strength due to ageing can be considered in the SMA.

The concrete compressive strength specified in the design document is 3000 psi (20MPa). However, the results of rebound of the Schmidt hammer test [11] show that the compressive strength is significantly higher than that specified in the design document. The rebound of the Schmidt hammer test is a tool that is commonly used in Non-destructive testing (NDT) of hardened concrete. Although coring concrete cylinders from the existing concrete is more desirable because the ageing factor is based on the results of cylinder compressive strength tests, NDT is more practical in this case because of less damage to the NRU structure. The test provides a rough measure of the quality of the concrete and can be used to provide a sense of the uniformity of the concrete compressive strength across the entire basement structure.

The tests that were carried out in-situ represent all types of concrete elements that exist in the basement structure, i.e., walls, columns, and slabs. Due to the difficulty in reaching undersides of floors and beams, in most cases the topside of the floor slabs were tested. Only a few concrete beams were possible to test from underside. Based on the field test data, averages and standard deviations are calculated and shown in Table 5.

In accordance with EPRI-NP-6041, concrete compressive strengths are likely to range from 10% to 45% over the minimum specified 28-day strength when cylinder test data show a covariance of 0.14. Due to the lack of recorded test data during the construction, the results of rebound of the Schmidt hammer test are used to account for increased strength of concrete due to ageing. An ageing factor of 1.5 is used, which increases the concrete compressive strength to 4500 psi (30MPa).

Table 5 - Average and Standard Deviation of Test Results

Element	Average (MPa)	Standard deviation
Column	44.5	4.8
Wall	42.6	4.4
Slab and beams	55.7	2.5
All elements	45.9	6.4

It is noted that the material strength used in CDFM is either the code-specified minimum strength or 95% exceedance actual strength if test data available. It is obviously too conservative to use the compressive strength specified in the design document for the current SMA. However, the uncertainty of the material properties based on the NDT may result in an argument of consistency of HCLPF capacity.

6. Ductility Factor

In accordance with EPRI-NP-6041, the nonlinear behavior of the structure can be assessed using nonlinear time history analysis and compare the maximum element demand ductility to a conservative estimate of its ductility capacity. This method is time consuming and the results are arguable. Hence a simplified nonlinear analysis is preferable.

The ductility factor K_μ is used to consider the nonlinear behavior of structure in CDFM. It is the same as the energy absorption factor $1/F_\mu$. The requirement for inelastic energy absorption is that for non-brittle failure modes and linear analysis, use 80% of the computed seismic stress in capacity emulation to account for ductility or perform nonlinear analysis and go to 95% exceedance ductility levels.

Alternatively, multiple modification factors based on the configuration of the structure and the construction material can be used to consider the nonlinear response of the structure. The factors are provided in the building codes and are similar to the ductility factor used in EPRI-NP-6041.

The ductility factor K_μ considered in the SMA of the basement of the NRU reactor facility is defined as follows:

K_μ, Ductility factor = $1/R_d\ R_o$

- R_d is ductility-related force modification factor = 1.5, shear walls for conventional construction
- R_o over strength-related force modification factor = 1

It is noted that the seismic performance of the facility is more like a building structure. As a result, the ductility-related force modification factor of shear walls for conventional construction as indicated in NBCC 2010 is used. The effect of over strength-related force modification is not considered in accordance with EPRI-NP-6041. Since the existing NRU facility did not fully incorporate seismic design at the time, the ductility factor in terms of probability of exceedance still needs to be verified to meet CDFM criteria.

7. Conclusions and Recommendations

Seismic margin assessment was conducted for the NRU reactor facility in Canada. The existing multipurpose nuclear facility was built in the 1950's when limited seismic knowledge and reinforcement details were implemented in the design.

The SMA method used in the current study is based on the EPRI-NP-6041 procedure. However, the alternative criteria in terms of seismic ground motion, structural model, concrete ageing and ductility factor are considered in the SMA. The parameter of PGA is used for selection of seismic margin earthquake level. A parametric study on stiffness and load conditions is conducted to find a best estimated structural model for analysis. The ageing effect on the compressive strength of concrete material is considered through Non-destructive test. The ductility factor based on NBCC 2010 is applied in the SMA. The alterative criteria are carefully examined and the deviations from the criteria provided in EPRI-NP-6041are justified.

The difficulties in application of SMA on the existing structure come from uncertainties and inconsistency in the selection of assessment criteria and parameters. There are always alternatives in SMA which provide different meanings in terms of probabilities and conservatism. Attention should always be paid to scrutinize the alternatives and justifications should be provided in selection of assessment criteria and parameters. Then, the statements based on SMA can be more meaningful and less questionable.

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