Seismic Fragility Analysis of a Nuclear Building based on Probabilistic Seismic Hazard Assessment and Soil-Structure Interaction Analysis

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Abstract

Seismic fragility analyses are conducted as part of seismic probabilistic safety assessment (SPSA) for nuclear facilities. Probabilistic seismic hazard assessment (PSHA) has been undertaken for a nuclear power plant in eastern Canada. Uniform Hazard Spectra (UHS), obtained from the PSHA, is characterized by high frequency content which differs from the original plant design basis earthquake spectral shape. Seismic fragility calculations for the service building of a CANDU 6 nuclear power plant suggests that the high frequency effects of the UHS can be mitigated through site response analysis with site specific geological conditions and state-of-the-art soil-structure interaction analysis. In this paper, it is shown that by performing a detailed seismic analysis using the latest technology, the conservatism embedded in the original seismic design can be quantified and the seismic capacity of the building in terms of High Confidence of Low Probability of Failure (HCLPF) can be improved.

Keywords: Seismic fragility, risk assessment, seismic hazard, ground motion, uniform hazard spectra, soil-structure interaction, dynamic model, CANDU 6.

1. Introduction

A Probabilistic Seismic Hazard Assessment was performed in 2015 that provided a Uniform Hazard Spectra (UHS) to represent the seismic hazard for Point Lepreau Generating Station (PLGS). Foundation Input Response Spectra (FIRS) was calculated for key safety-related buildings of the plant considering site-specific geological conditions [1].

The original seismic design for PLGS in the 1970s was based on a typical Canadian Standard Association spectral shape with anchorage to a peak ground acceleration of 0.2g. The Design Basis Earthquake (DBE), defined for a hazard equivalent to an approximate 1 in 1,000-year return period, is energy rich in the frequency range from 2.7 to 7 Hz. However, the FIRS, calculated for a total hazard at a mean 10,000-year return period (i.e. mean Annual Frequency of Exceedance of 1E-04), has a PGA of 0.344g and is energy rich in the high frequency range that is greater than 10 Hz. The DBE and FIRS are presented in Figure 1.

Updated seismic fragility evaluations are required based on the FIRS. The fragility analysis method, documented in EPRI Report TR-103959 [2] is followed to calculate the median seismic capacities of the critical failure modes of the building and their associated variability. High Confidence of Low Probability of Failure (HCLPF) calculation for the service building division II is presented in this work.

The scaling method suggested in EPRI reports [3] & [4] to make use of previous analysis results are not applicable because of the difference in the UHS and DBE spectral shapes. In addition, "as built" structure drawings including some modifications completed during the service life of the structure are used for the seismic fragility evaluation.

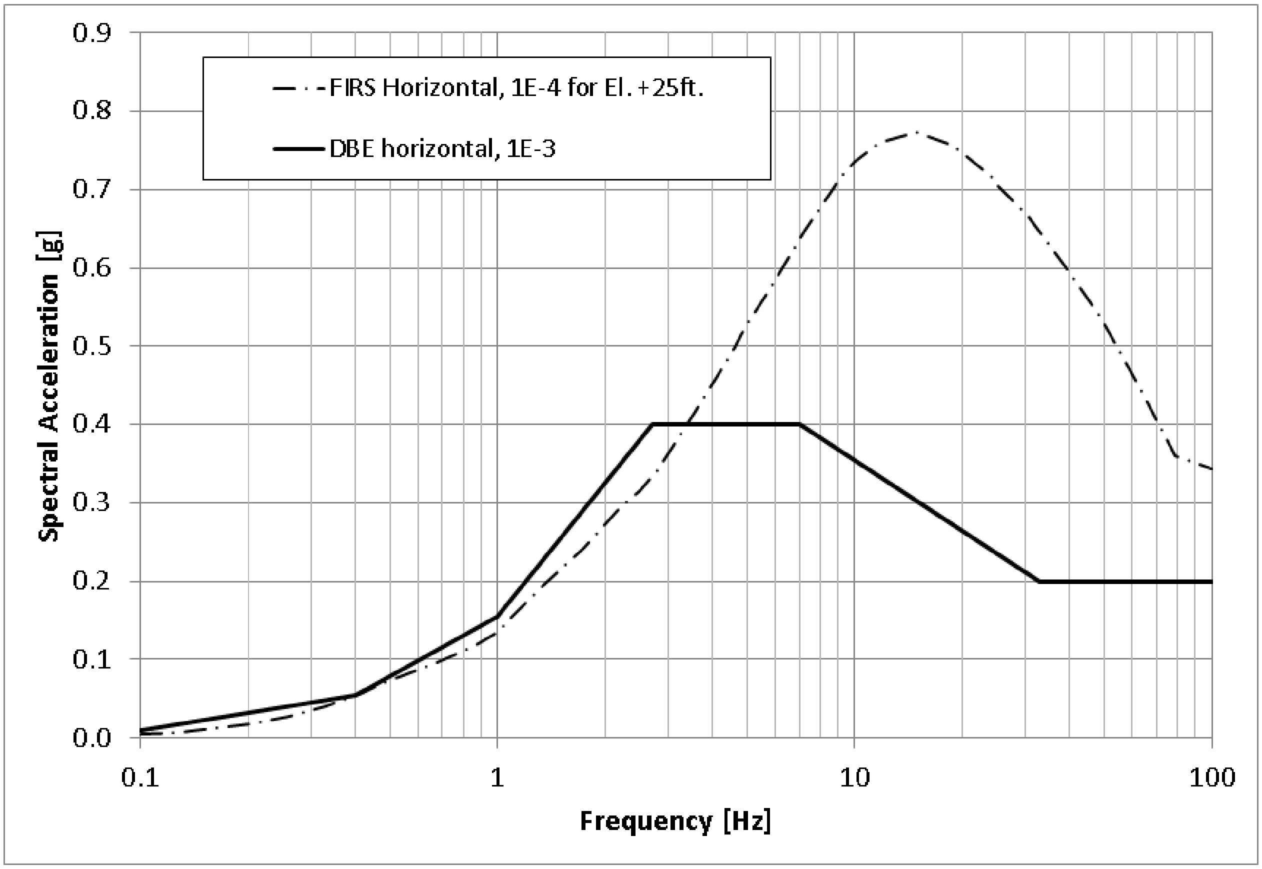

Figure 1 Comparison between PLGS Design Basis Earthquake, and Foundation Input Response Spectrum at a Mean Annual Frequency of Exceedance of 1E-04

2. Service building division II description

The service building division II is part of the structural complex adjacent to the reactor building that contains all supporting services for the equipment in the reactor building. The service building division II consists of a reinforced concrete substructure and a steel superstructure with floors composed by steel beams embedded in concrete. The building has a semi-circular shape on the side facing the reactor building. The approximate dimensions in plan are 41 m x 25 m. It is approximately 15 m tall from elevation 13.72 m (45 ft, the grade level of the plant), as shown in Figure 2. The service building division II comprises of: a floor at elevation 13.72 m (45 ft), a floor at elevation 22.94 m (75 ft - 3 in), a roof at elevation 28.73 m (94 ft – 3 in) and, a one story basement at elevation 7.62 m (25 ft). The roof supports some small steel superstructures: one for pipe supports and cable tray housing, another one for a filter room and the stairs enclosure.

3. Seismic demand

Soil Structure Interaction (SSI) analyses are conducted by the sub-structuring approach implemented in the computer program ACS SASSI [5]. It uses linear finite element modelling and the frequency domain solution methods. The complex soil-structure system is partitioned into two sub-structures, namely, the foundation and the structure. In this partitioning, the structure consists of the upper-structure plus the basement minus the excavated soil and the foundation consist of the soil layers including the excavated soil. Each sub-problem is solved separately and the results are combined in the final step of the analysis to provide the complete solution using the principle of superposition. The interaction occurs at all excavation volume nodes if the flexible volume method is selected or at the structure-soil interface nodes if the interface method is selected.

3.1 Mathematical model

A finite element model of service building division II, including post built additions to the structure is developed for soil structure interaction analyses. The model is built by using the computer program ANSYS [6], to take advantage of its excellent modelling capabilities, and then is transferred to ACS SASSI. Selection of element types and sizes are according to recommendations in ACS SASSI reference manual.

Openings indicated in structural drawings are accounted in the model. Enclosing walls are modelled through mass elements; their stiffness is neglected by engineering judgement. All main structural connections are considered to be hinge connections as assumed for the original structural design.

Snow mass and the roof mass are accounted through mass elements at the roof corner nodes per tributary area. Figure 2 shows the finite element model of the service building division II. The excavated soil in the embedded portion of the model is included in the model for soil structure interaction analysis. The soil is represented by solid type elements with sizes up to 1.0 m in vertical direction. Nine layers are modelled within the embedded portion of the model.

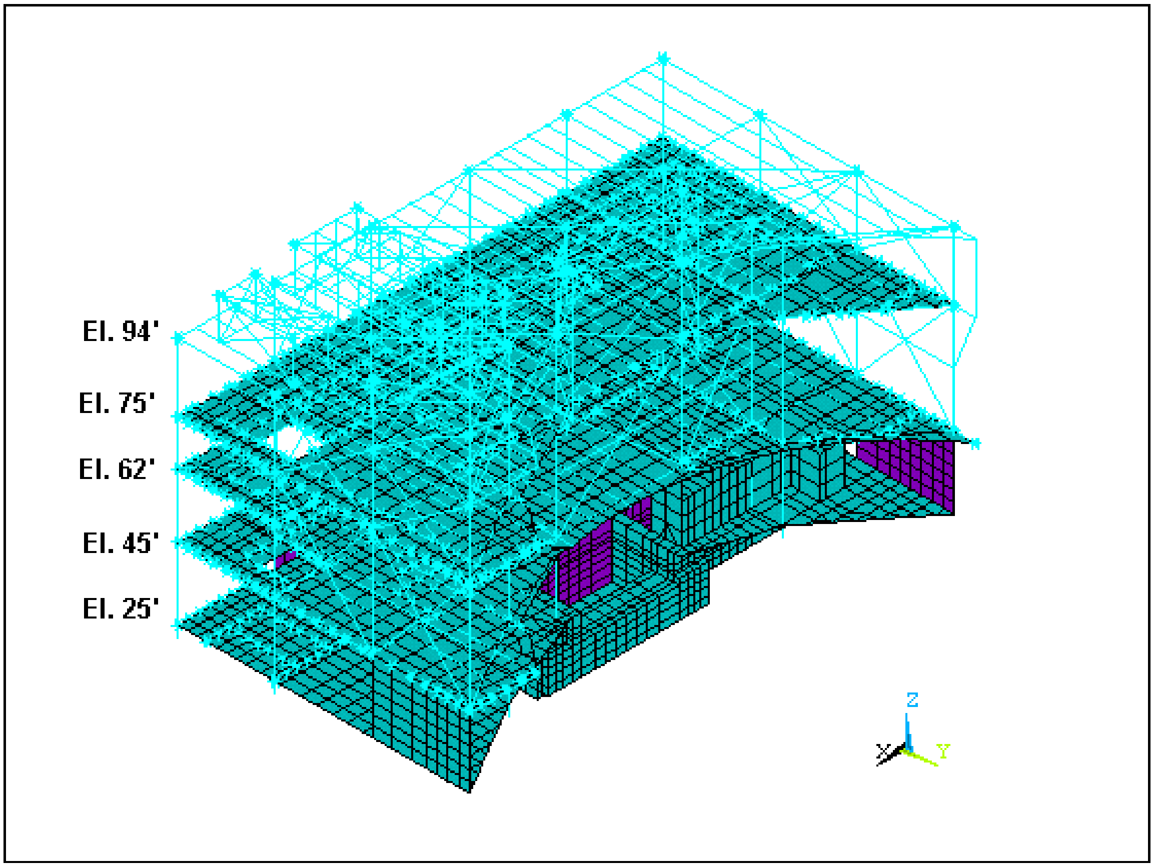

Figure 2   Service Building Division II Finite Element Model

3.2 Foundation medium

The plant is sited on competent rock. The stiffness of any soil above elevation 7.62 m (25 ft) is not accounted for in the analysis with conservative engineering judgement. The foundation medium is modelled using discretized soil layers overlying a uniform elastic half space.

3.3 Modal analysis

Modal analysis is conducted in ANSYS to compute the main mode frequencies of the service building structure. The boundary condition analysed is the fixed-base. The model is fixed at elevation 7.62 m (25 ft) for modal analysis. The fixed-base natural frequencies give some insights on the approximate location of the peaks in the transfer functions to be computed in the SSI analyses and allow model checking after transferring it to ACS SASSI. Table 1 presents the main frequencies comparison between the fixed-base model and SSI model. The main frequencies in both analyses are very similar because the soil profile is hard rock. As expected, the structural dynamic behaviour indicates that the steel superstructure is more flexible than the concrete portion of the structure. The vertical modes are more localized than the horizontal modes as can be concluded from the mass participation.

Table 1 Main Frequencies of Service Building Division II

Mode Number	Frequency Fixed base model (Hz)	Frequency SSI model (Hz)	Direction	Remarks	Mass Participation Fraction
2	2.6	2.6	Y (A-C)	Steel Structure	29%
3	2.7	2.8	X (B-D)	Steel Structure	36%
7	3.6	3.5	Z (Vertical)	Steel St. (local mode)	1%
14	4.9	5.0	Z (Vertical)	Steel St. (local mode)	2%
21	5.5	5.5	Z (Vertical)	Steel St. (local mode)	2%
125	8.3	7.5	Y (A-C)	Concrete Structure	4%
172	10.3	9.6	X (B-D)	Concrete Structure	14%
233	13.4	13.4	Z (Vertical)	Concrete Structure	2%

3.4 Seismic input

The seismic inputs for the seismic analyses are acceleration time histories in three perpendicular directions compatible with the FIRS. The time histories meet the acceptance requirements in CSA N289.3 [7] and tightly match the FIRS developed at elevation 7.62 m (25 ft) based on site response analysis [1]. In addition, the site response analysis is developed based on the UHS at the bedrock level developed in the probabilistic seismic hazard assessment.

The control point or location of the seismic input is selected at elevation 7.62 m (25 ft). The time-step of all input time-histories is 0.005 seconds. The time-histories are defined by 8192 points including a quiet zone. In the frequency domain, this translates to a frequency increment of $\Delta f = 1/(0.005*8192) = 0.0244$ Hz.

3.5 Cut-off frequency

The cut-off frequency used in the analysis is 50 Hz; it is assessed by ensuring that the computed transfer functions are satisfactory up to the selected cut-off frequency. The transfer functions are computed at discrete frequency points. Interpolation schemes implemented in ACS SASSI program are used to compute interpolated transfer functions from the discrete transfer functions. A large number of frequency points (97) are used in the SSI analyses for the computation of transfer function curves to ensure that the interpolated transfer functions are well defined over the frequency range of interest following recommendations in Reference [5]. Figure 3 shows transfer functions at some nodes, for the analysis of the structure, in B-D horizontal direction. The blue and black curves show transfer functions of nodes located in the concrete substructure and the red and light blue show transfer functions of nodes located on the steel structure.

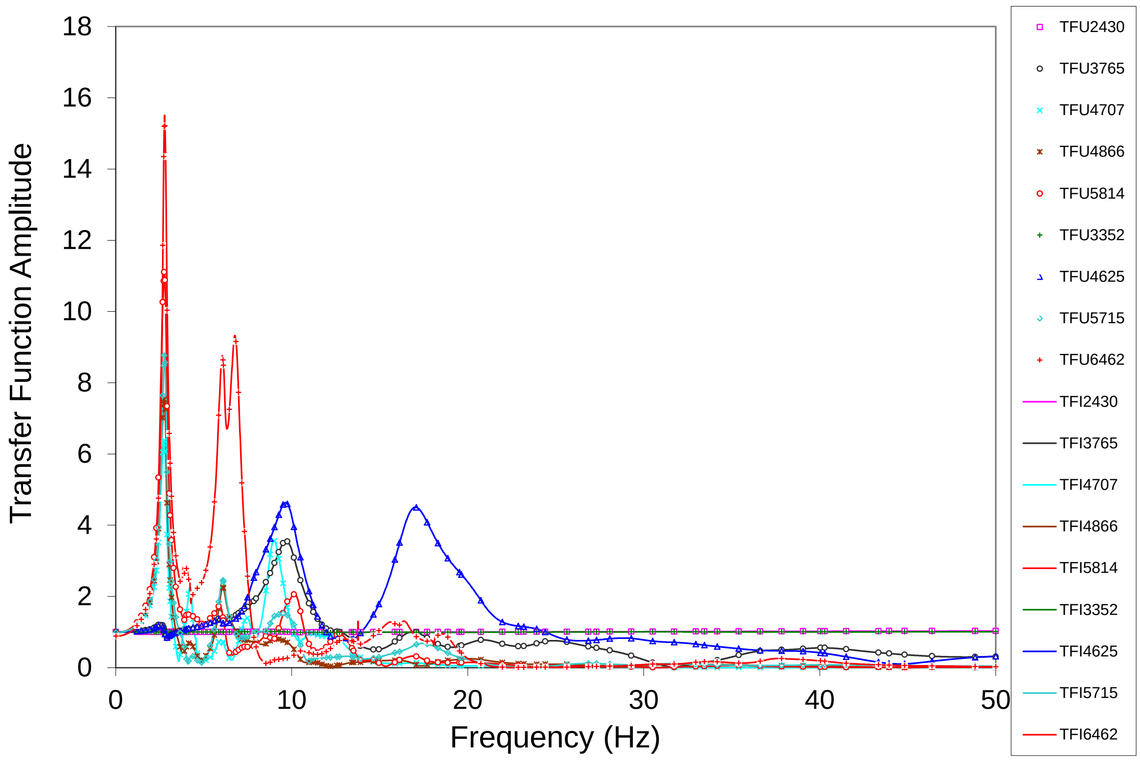

Figure 3 Transfer functions for B-D (X) direction

3.6 Seismic results

Floor response spectra, variation of structural acceleration and displacement along the height, and seismic forces in structural members are calculated for two horizontal directions each in combination with the vertical direction. The floor response spectra are generated in three orthogonal principal directions for five critical damping values including 2%, 4%, 5%, 7% and 10%.

Floor response spectra and displacements are the seismic inputs for fragility calculations of system and components in service building division II, while the seismic forces represent the seismic demands for fragility calculations of the structure.

The most stressed structural elements in the steel structure of the service building are the columns and bracing system between elevations 13.72 m (45 ft) and 22.86 m (75 ft). The bracing system along the grid lines “H” and “K” resists the highest seismic loads for the UHS seismic input. Consistently, the bracing system along the grid lines “H” and “K” presented the highest seismic forces for the DBE seismic input. Figure 3 shows grid lines and the columns location. Figures 4 to 7 present the structure main frames configuration.

The seismic forces calculated through the SSI analysis based on UHS are about 25% to 40% lower than the seismic forces based on the DBE seismic input used for the structure design. The forces distribution among members was similar in both analyses.

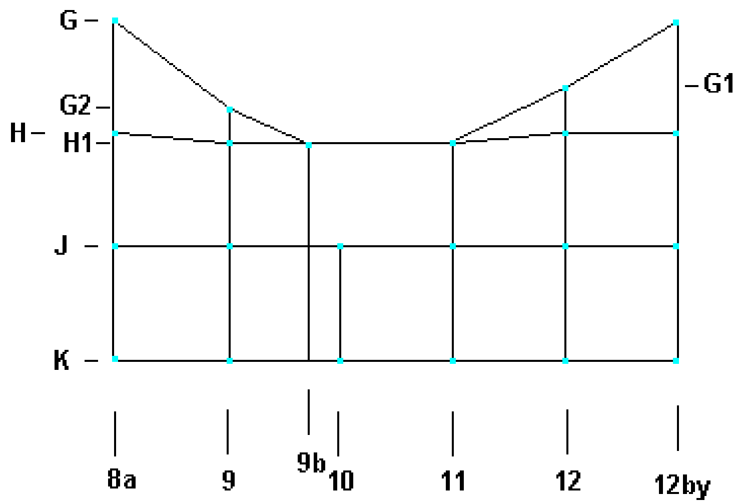

Figure 4 SB Division II main columns location

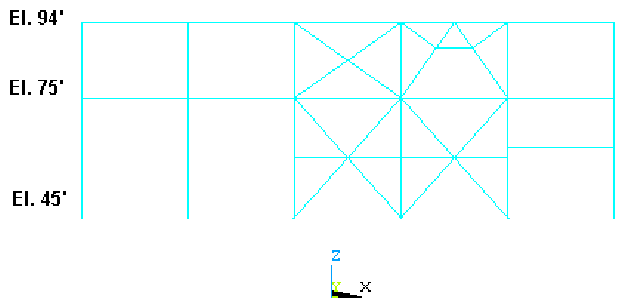

Figure 5 Section at Grid Line "K"

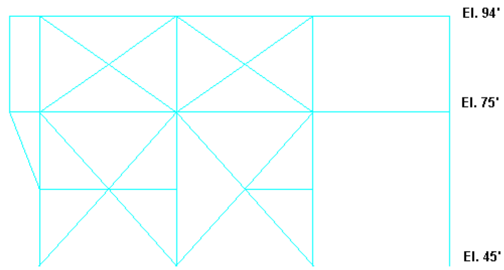

Figure 6 Section at Grid Line "8a"

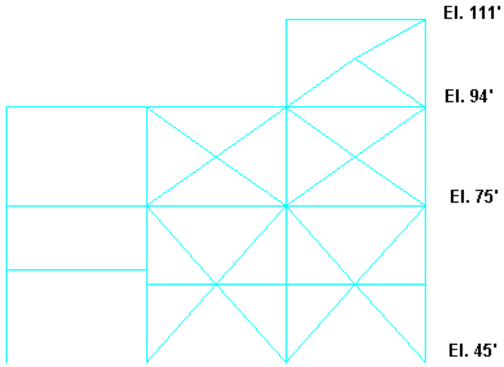

Figure 7 Section at Grid Line "12by"

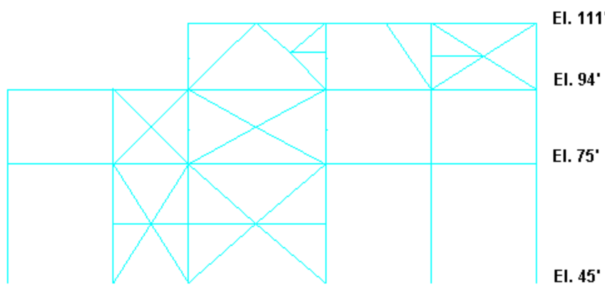

Figure 8 Section at Grid Line "H-H1"

4. Structure seismic capacity

The seismic capacity or fragility is defined as the conditional probability of failure that the structural response exceeds the structural capacity or resistance for a given peak ground acceleration. The methodology documented in EPRI Report TR-103959 [2] is followed to calculate the median seismic capacity of the building and its associated randomness and uncertainty variability.

The median seismic capacity is quantified by the High Confidence of Low Probability of Failure (HCLPF) value, which represents, with a 95% confidence, that the probability of failure of the structure will not exceed 5%. It is calculated by:

$$\text{HCLPF capacity} = \text{Am} \exp^{[-1.65(\beta_R + \beta_U)]} \quad (1)$$

The family of seismic fragility curves may be described by these three parameters, i.e., Am, β_R and β_U, where,

Am: is the median capacity and β_R and β_U are the variability defined in terms of randomness and uncertainty, respectively.

$$A_m = F.\ A_{UHS} \quad (2)$$

Where, A_{UHS} is the peak ground acceleration for the UHS and F is the median factor of safety for an identified mode of failure.

The median factor of safety and variability are calculated for factors listed below. The overall factor of safety (F) is the product of all factors of safety. The overall randomness and uncertainty is the square root of the sum of squares of all randomness and uncertainty values.

The capacity factor considers $F_C = F_S.\ F_\mu$ (3)

- Strength Factor (F_S)
- Inelastic Energy Absorption Factor (F_μ)

The structural response factor (F_{SR}) is based on: $F_{SR}=F_{SH}.F_D.F_M.F_{MC}.F_{EC}.F_{GMI}.F_{SSI}$

- Spectral Shape Factor
- Damping Factor
- Modeling Factor
- Modal Combination Factor
- Earthquake Components Combination Factor
- Ground Motion Incoherence Factor
- Soil-Structure Interaction Factor

F is represented by:

$$F = F_C\ .F_{SR} \quad (4)$$

$$\beta_R = (\beta_{RC}{}^2+\beta_{RSR}{}^2)^{1/2} \quad (5)$$

$$\beta_U = (\beta_{UC}{}^2 + \beta_{USR}{}^2)^{1/2} \tag{6}$$

β_{RC} = logarithmic standard deviation due to randomness associated with seismic capacity factor, it is the square root of the sum of squares of the randomness of the strength and inelastic energy absorption factors.

β_{RSR} = logarithmic standard deviation due to randomness associated with structure response factor, it is the square root of the sum of squares of the randomness of every factor accounted for the structural response factor.

β_{UC} = logarithmic standard deviation due to uncertainty associated with seismic capacity factor, it is the square root of the sum of squares of the uncertainty of the strength and inelastic energy absorption factors.

β_{USR} = logarithmic standard deviation due to uncertainty associated with structure response factor, it is the square root of the sum of squares of the uncertainty of every factor accounted for the structural response factor.

4.1 Failure modes in the service building

The reinforced concrete service building substructure is massive and seismically rugged. The superstructure however is a steel braced frame structure with pinned joints. The structure seismic analysis results indicated that the bracing system and columns between elevations 7.62 m (45 ft) and 22.86 m (75 ft) carry most of the seismic forces.

The bracing system along grid line “K” between grid lines 11 and 12 is selected for seismic fragility calculations considering the high acting seismic load and the largest slenderness ratio. Buckling of a diagonal bracing of this frame is judged to be one of the potential critical failure modes.

The connection between the diagonal member and the beam/column is another potential critical failure mode and, anchoring failure of columns base experiencing uplift forces is another potential failure mode that is reviewed in the service building seismic fragility calculations.

4.2 Capacities per failure mode

The capacities for each individual failure mode are determined and expressed in terms of the same parameters as the demands (i.e., forces, spectral accelerations). The seismic capacities are formulated as the products of the strength and inelastic energy absorption factors.

Axial compression capacity of the bracing member relative to the seismic axial compression demand gives a strength factor of 3.58 for the buckling of the selected bracing member. Seismic fragility calculations for DBE seismic input consistently showed a lowest strength factor of 1.71 obtained for buckling of the selected diagonal member, among other modes of failure evaluated.

Buckling of the bracing gives the lowest calculated factor of safety among the three evaluated modes of failure and therefore is assumed to be the dominant failure mode of the structure.

However; immediately after the bracing buckles in compression the seismic load is expected to be re-distributed to the remaining cross bracing which is in tension at that time. It is, therefore, a post buckling capacity that increases the structure strength for seismic fragility calculation. An inelastic energy absorption factor of 2 for tensile failure is then considered per ASCE 43-05 [8].

4.3 Seismic fragility calculation

Following the methodology above, seismic fragility calculations based on DBE seismic forces resulted in a service building division II HCPLF of 0.42g.

The total factor of safety, calculated as the product of all factors of safety is F=8.26, and the combined randomness and uncertainty variability are β_R=0.23 and β_U=0.42. Therefore, Am=2.84g and the service building division II HCLPF is estimated to be 0.98g for a Peak Ground Acceleration of 0.344g from the FIRS that was derived from the Uniform Hazard Spectra.

The improvement in HCLPF obtained for the FIRS seismic input compared with the DBE seismic input is gained from different sources:

a) The fact that the seismic input is rich in high frequency but low at the structure main range of frequencies (between 2 and 4 Hz) causes a decrease in the seismic forces imposed on the structure compared to the DBE seismic forces.

b) The conservatism embedded in the building original design related to the selection of structural member sizes.

5. Conclusions

In this work, the seismic fragility of the service building division II structure is calculated. The analysis is based on a finite element model and soil structure interaction analysis. The soil structure interaction effects are idealized using frequency-dependent impedance functions for the best estimate soil properties at the site.

Time histories that tightly match the UHS at the foundation level for Point Lepreau Generating Station is the seismic input used in the seismic analyses. The uniform hazard spectrum and foundation input response spectrum is rich in high frequency content which differs from the design basis earthquake. The floor response spectra and seismic forces in the structure are determined for use in seismic fragility calculations of the building. The UHS and foundation seismic input resulted in lower forces than the design forces by about 25 to 40%.

The critical elements in the structure are identified and the probability of failure is calculated. The resulting HCLPF value based on the UHS at the foundation level was determined to be higher than the HCLPF calculated in previous service building seismic fragility calculations that were based on a spectrum derived by scaling up the DBE to reflect a higher seismic demand for the 2008 Point Lepreau PSA-based Seismic Margin Assessment.

The main frequencies for the steel superstructure are between 2 and 4 Hz. Therefore, the structure is more stressed during a design basis type of earthquake than the uniform hazard type of earthquake even though the peak ground acceleration is higher for the latter (0.2g vs. 0.344g) and the probability of exceedance is about an order of magnitude lower for the UHS than the DBE type of earthquake.

The service building division II high confidence of low probability of failure (HCLPF) is 0.98g for the uniform hazard spectrum in contrast to 0.42g as calculated for the 2008 Point Lepreau PSA-based Seismic Margin Assessment. The results demonstrate that;

(a) relative flexible structures such as the steel super structure of the service building division II are not affected adversely by high-frequency motions;

(b) the service building division II seismic design for sites with seismic earthquakes rich in high frequency content and hard rock foundation condition is robust; and,

(c) detailed finite element modelling of the building is an efficient way to remove the conservatisms embedded in the original simplified design.

6. References

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