THE SEISMIC FRAGILITY ANALYSIS FOR MULTI-STORY STEEL STRUCTURE IN CANDU NUCLEAR POWER PLANT

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The Wolsong Unit 2 is CANDU-6 type plant and being constructed in the Wolsong site, where Design Basis Earthquake (DBE) was determined to be 0.2g. A seismic PSA for Wolsong Unit 2 is being performed as one of the conditions for the Construction Permit. One of the issues in the seismic PSA is the availability of the seismically non-qualified systems, which are located in the Trubine Building(T/B). Thus, the seismic fragility analysis for the T/B was performed to estimate the operability of the systems. The design seismic loads for the building were based on a ground response spectrum scaled down from the DBE to horizontal peak ground acceleration (pga) of 0.05g. The seismic fragility analysis for the building was performed using a factor of the safety method. It is estimated that the most critical failure is that of masonry walls and its High Confidence and Low Probability of Failure (HCLPF) capacity is 0.13g. The critical failure mode of the structure is identified to be tensile yielding failure of grip angle, and its HCLPF capacity is 0.34g.

1. INTRODUCTION

The Wolsong Units 2, 3, and 4, CANDU-6 type plants, are now under construction. Construction of the plants was started in 1991, and are scheduled to be in commercial operation by 1997, 1998, and 1999, respectively. One of conditions for the construction permit is the seismic IPEEE (Individual Plant Examination for External Events) for the plants. The DBE of the site is 0.2g, the same as other nuclear power plant sites in Korea. However, the seismic hazard for the site is higher than that of any other sites in Korea.

Since no seismic IPEEE study had been performed for the CANDU plants worldwide, the seismic IPEEE was performed in two phases, 1) feasibility study phase and 2) main seismic IPEEE phase. The main purpose of the feasibility study was to determine the applicability of the seismic IPEEE approach for PWR type plants and if applicable, to establish the approach for the CANDU plants. This was performed by : 1) comparing the design features of the CANDU plants with those of PWRs, 2) determining the applicability of the approach, and 3) establishing the approach/strategy of the analysis. The first phase study was completed in January, 1994. The second phase "The main seismic IPEEE" study is to perform the seismic risk analysis based on the approach and the strategy established from the first phase study. The second phase started in September, 1994 and is estimated to take about 36 months.

The feasibility study identified that the seismic design features of the CANDU plants are quite different from that of the PWRs. but concluded that the IPEEE methodology for PWRs could be applied in the CANDU plants with some proper modifications (Beom-Su Lee, et al). One of the important recommendations of the study is to perform seismic fragility analyses for the seismically non-qualified components to get some credits for the availability of non-qualified systems, which can perform safety function, during/after an earthquake. Most of those systems are located in the turbine building. Thus, one of the essential task for the seismic IPEEE is to perform the seismic fragility analysis for the turbine building.

This paper presents the process and results of the seismic fragility analysis for the turbine building. The resulting seismic fragility value will be used as an input to estimate the seismic induced core damage frequency.

2. DESCRIPTION OF THE STRUCTURE

2.1 Physical Description

The turbine building (T/B) has horizontal dimensions of 97.6 m x 65.1 m (320 ft x 214 ft) and consists of a main turbine hall and auxiliary bay. The main turbine hall houses the turbine generator. The auxiliary bay houses non-safety related electrical equipment (e.g. batteries, battery chargers, inverters, etc.), deaerator, and feedwater tank. The T/B foundation consists of reinforced concrete raft extending over the entire area of the building with the top of the foundation raft at the elevation of 83.76 m. The raft is surrounded by reinforced concrete retaining walls along three sides and by the service building foundation along the fourth side. The steel superstructure with its main columns supported on the retaining walls and the raft foundation encloses the main turbine hall and supports several floors of the auxiliary bay. The T/B is structurally separated into two parts (Divisions I and II) by an expansion joint along the mid-length of the building from the roof level down to the raft foundation.

The lateral-force resisting vertical systems of the T/B superstructure consists of braced steel frames in the longitudinal direction and moment frames (main turbine hall) and braced frames (auxiliary bay) in the transverse direction. The lateral-force resisting horizontal systems of the auxiliary bay consist of reinforced concrete floor slabs supported on steel decks which distribute floor inertia loads to the lateral-force resisting vertical systems.

2.2 Design Analysis for the Structure

The Wolsong DBE horizontal input ground motion is defined by the 90th percentile ground response shape (CSA CAN3-N289.3-M81) anchored to a peak ground acceleration of 0.2g. The vertical input ground motion is taken to be 2/3 of the horizontal input ground motion across the entire frequency range of interest. For bolted steel structures such as the T/B superstructure, damping value of 5% of critical was used for design ground response spectrum.

Two seismic design analyses were performed for the non-safety related T/B based on operational reliability and nuclear safety requirements. For operational reliability, the T/B was designed to perform its function under all normal operating loads and under the earthquake loads based on the National Building Code of Canada (NBCC, 1985) seismic zone 2 requirements. The T/B design seismic loads were determined based on a ground response spectrum scaled down from the design basis earthquake (DBE) of 0.2g to a horizontal pga of 0.05g. This earthquake was referred to as General Design Earthquake (GDE).

For nuclear safety, the T/B was designed not to cause damage to the adjacent service building under the DBE. Elastic response spectrum analysis was performed using the DBE ground motion parameters. The structural steel members of the superstructure were designed on the basis of the computed elastic member forces reduced by a factor of 2.0. This factor is the force modification factor specified in the National Building Code of Canada reflecting the capability of a braced frame with nominal ductility to dissipate energy through inelastic behavior. This force modification factor was not used for the T/B floor response spectra generation. Results from the modified DBE seismic analysis were used for evaluating the seismic fragility of the T/B.

The structural steel members were designed according to the Limit States Design Method as defined in CAN3-S16.1-M89. All structural elements were designed to have sufficient strength and stability so that the factored resistance was greater than the effects of factored loads.

Three-dimensional finite element models were used for the T/B Division I and II design analyses. Major structural members such as columns, floor beams and diagonal members of the vertical braced frames were included in the detailed models, and the concrete slabs were also modeled appropriately using finite elements to incorporate their stiffness. The columns are assumed to be hinged at their base. Vertical retaining walls along three sides of the T/B were represented as equivalent vertical cantilever members embedded at the elevation of 88.81 m.

Dominant natural frequencies of the auxiliary bay were determined as 0.8 Hz for overall translational mode in the transverse direction with torsional response, 0.95 Hz for overall translational mode in the longitudinal direction with torsional response, and 2.5 Hz for 2nd mode of the overall response of the T/B superstructure.

3. SEISMIC FRAGILITY EVALUATION

3.1 Seismic Fragility Model

The seismic fragility evaluation of the T/B followed the separation of variables methodology discussed in EPRI-TR-103959 (EPRI, 1994). The following structure capacity and response factors were considered in the seismic fragility evaluation:

- Capacity Factors
  - Strength factor
  - Inelastic energy absorption factor
- Response Factors
  - Spectral shape factor
  - Damping factor
  - Modeling factor
  - Modal combination factor
  - Earthquake components combination factor
  - Soil-structure interaction factor

For the controlling failure mode of the structure, the median factor of safety (F_i) and the associated uncertainty (β_U) and randomness (β_R) variabilities are determined for each of the above factors. The median seismic capacity (A_m), expressed in terms of peak ground acceleration and the associated β_R and β_U of the governing failure mode are defined by the equations below:

$$A_m = (\Pi F_i) * (DBE);$$
$$\beta_R = [\Sigma(\beta_R)^2_i]^{1/2}$$
$$\beta_U = [\Sigma(\beta_U)^2_i]^{1/2}$$

The high confidence of low probability of failure seismic capacity (HCLPF) of the controlling failure mode is given as the 95 percent confidence of less than 5 percent probability of failure. The HCLPF capacity is calculated from the median seismic capacity and its variabilities as shown below:

$$HCLPF = A_m e^{-1.65(\beta_R + \beta_U)}$$

3.2 Structure Capacity Factors

A comparison of the modified DBE design demand plus normal operating load to the code capacity ratios was performed for the T/B superstructure major columns, vertical brace members, member connections, and column base anchorage to identify the controlling failure modes. The diagonal braces of the lateral-force resisting vertical systems in the longitudinal direction of the T/B auxiliary bay were identified to be the controlling structural elements.

The typical T/B superstructure diagonal bracing consists of wide flange steel members connected to the main columns and beams through four steel grip angles. The steel grip angles are connected to the web of the wide flange diagonal member at one end and to the gusset plate at the other end with A325 friction type high strength bolts. The bolted connection was designed such that its capacity is higher than the calculated code capacity of the member. The following potential failure modes of the diagonal bracing members were evaluated:

- Tensile yielding and compression buckling of the diagonal bracing member and the grip angles
- Shear failure of the bolted connection

The governing failure mode was determined to be the tensile yielding of the grip angles. A median strength factor of 0.93 was calculated for this failure mode. The median strength factor is defined as the ratio of the median tensile yield capacity to the DBE design member force. It is noteworthy to point out that the elastic seismic member force (i.e. bracing member force without the force modification factor of 2) was used here, whereas the reduced inelastic force was used for the member design.

The inelastic energy absorption factor, which accounts for structure capability to absorb earthquake energy was calculated using the system ductility corresponding to failure. The selection of the ductility ratio corresponds to the onset of severe strength degradation as the median failure ductility is expected to provide a substantial margin against building collapse. The median story drift (relative lateral displacement divided by the story height) at which failure of the T/B auxiliary bay may occur is assumed 0.7% story drift. For the multi-degree-of-freedom T/B structure, it was necessary to establish system ductility rather than using the story ductility directly. The system ductility was determined from:

$$\mu_{system} = \frac{\sum W_i \Delta_{\mu,i}}{\sum W_i \Delta_{e,i}}$$

where W_i is the tributary story weight, $\Delta_{\mu,i}$ is the story drift at failure and $\Delta_{e,i}$ is the elastic story drift at yield. From this system ductility, the factor of safety of inelastic energy absorption was obtained using the Newmark-Riddell procedure (Riddell and Newmark, 1979).

The median inelastic energy absorption factor of safety of the T/B was determined to be 2.05 with associated randomness and uncertainty variabilities of 0.09 and 0.19, respectively.

3.3 Structure Response Factors

Only the spectral shape factor is presented in details here since it contributes significantly to the T/B seismic fragility. The spectral shape factor is to account for conservatism in the design ground response spectrum when compared to the site-specific median ground response spectrum. Two significant modes of the auxiliary bay along the longitudinal direction of the T/B were identified from the modal analysis of the T/B. The corresponding natural frequencies of these two modes are 0.95 Hz and 2.5 Hz, respectively. Ratios of the design spectral acceleration at the design damping (5%) to the median spectral acceleration at the median structure damping (10%) for each of these two modes were determined to be 2.9 and 2.73, respectively. The final spectral shape factor was determined to be 2.87 when the participation factors of these two modes were considered. The associated randomness and uncertainty variabilities are 0.20 and 0.24, respectively. Factors of safety and the associated variabilities of other response parameters such as damping, modeling, modal combination, earthquake components combination, and soil-structure interaction are shown in Table 1.

3.4 Masonry Block Walls

There are masonry block walls throughout the auxiliary bay of the T/B from the basemat to the floor at the elevation of 120.54m. The walls are typically 10 in. thick with vertical and horizontal reinforcing steel. All the masonry walls were designed to be vertically spanning structural elements for the out-of-plane seismic load and/or the dynamic steam pressure load. Seismic-induced failure of these masonry block wall would pose a threat to the components in the auxiliary bay which are included in the PSA study.

There are several full story-high (7 meters) masonry walls on the floor at the elevation of 120.54 m that provide enclosure to the inverter room and battery rooms. These walls are vertically reinforced with No. 7 bars spaced at 4 feet on center. At the base, the wall is connected to the concrete floor slab with No. 7 dowels. The walls are laterally braced to the roof beams at the top. The fundamental frequency for the out-of-plane response of the wall was estimated to be 2 Hz. The 10% damped floor design spectral acceleration at 2 Hz is 1.2g. At this acceleration level, the median strength factor of the masonry block wall was determined to be 0.71. The controlling failure mode of the masonry wall is hinging at the mid-height of the wall due to the out-of-plane inertial effect. The factor of safety of the inelastic energy absorption of the wall was determined to be 1.15.

Other masonry walls in the T/B that separate the auxiliary bay from the main turbine hall were designed for a dynamic steam pressure of 8 to 13 KPa and thus determined to have higher seismic capacity than the above discussed controlling walls.

4. RESULTS

The final median seismic capacity of the T/B auxiliary bay was determined to be 1.07g with β_R of 0.32 and β_U of 0.42 and the HCLPF capacity was estimated to be 0.32g. The critical failure mode was the tensile yielding of the grip angles. Since the diagonal bracing was lateral-force resisting vertical systems in the longitudinal direction, the failure is judged to result in collapse of the building in the longitudinal direction and there would be little impacts on the service building.

The final median seismic capacity of the masonry walls was determined to be 0.46g with a HCLPF capacity of 0.13g. Since the masonry wall enclose the inverter room and battery rooms, the failure of the masonry walls would impact the availability of batteries or electrical equipment in the inverter rooms. In the inverter room there are most of class I and class II electrical system electrical cabinets. Thus the collapse of the masonry walls into the direction of inverter room would cause loss of class I and/or II electrical systems, and it would limit the availability of non-qualified systems.

The fragility analysis results in the conclusion that the most critical limitation of the availability of non-qualified systems is the seismic-induced failures of masonry walls of turbine building, and it limits the availability of the systems to the 0.13g HCLPF.

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Table 1. Median Factors of Safety and Uncertainty Parameters for Turbine Building

Fragility Parameters	Median F.S.	β_R	β_U
Strength Factor	0.93	0	0.13
Inelastic Energy Absorption	2.05	0.09	0.19
Spectral Shape	2.88	0.20	0.24
Damping	1.0	0	0.12
Modeling	1.0	0	0.22
Modal Combination	1.0	0.17	0
Earthquake Components Combination	1.0	0.15	0
Ground Motion Incoherence	1.0	0	0
Soil-Structure Interaction	1.0	0.	0.05
Total	2.35	0.32	0.42

Median Capacity = 1.10g

HCLPF Capacity = 0.32g