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PRELIMINARY ESTIMATION OF FRAGILITY CURVES FOR THE APR1400 TURBINE BUILDING UNDER SEISMIC LOADING

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ABSTRACT

Seismic probabilistic safety assessments are the state-of-practice for evaluating the potential impact earthquakes can have on a nuclear power plant. Seismic probabilistic safety assessments can basically be de-aggregated into two components, seismic hazard analyses and construction of a fragility curve. These two components are used to estimate failure frequencies in nuclear power plant systems, structures, and components. In this preliminary investigation, an empirical seismic fragility curve is constructed for the APR1400 nuclear power plant turbine building. A suite of seismic ground motions is selected from global databases to cover a range of frequencies to be used in the simulation. Motions are also amplified to help make alternative events available. These ground motions are input into a structural analysis program with the APR1400 turbine building model to simulate earthquake excitation. Failure was considered when any framing element reached its yield stress, excessive member deformation, or exceedance of allowable story drift. A consolidation of these results is then mapped into a fragility curve to be regressed into the generic fragility function given by the Electric Power Research Institute.

INTRODUCTION

Seismic probabilistic safety assessments are the state-of-practice for evaluating the potential impact earthquakes can have on a nuclear power plant. Seismic probabilistic safety assessments can basically be de-aggregated into two components, seismic hazard analyses and construction of fragility curves. These two components are used to estimate failure frequencies in nuclear power plant systems, structures, and components. In this preliminary investigation, an empirical seismic fragility curve is constructed for the APR1400 nuclear power plant turbine building. The APR1400 turbine building houses the turbine generator, condenser systems, feedwater systems, and additional balance of plant systems through four main floor levels referred to as the basement, ground level, operating level, and deaerator level. Additionally, the turbine building is classified as non-safety related and has no major non-seismic structural interfaces with other structures. Although it is noted that the APR1400 is designed to safely operate without access to systems, structures, and components in the turbine building (Kim 2009), a seismic fragility curve can expand the database (USNRC 1998, EPRI 2002) and help engineers perform a more complete and rigorous safety assessment of the APR1400.

The remainder of this paper describes the use of a computer program to generate a virtual model of the APR1400 turbine building framing, subjecting the model to several ground motions of varying peak ground accelerations (PGAs), and compiling the numerical results to form a fragility curve. This study is preliminary in the sense that (1) only structural steel framing is considered based on Korea Hydro & Nuclear Power (KHNP) architectural drawings, (2) the internal cranes, which are coupled with the steel framing, are not modelled, (3) the turbine generator pedestal is not modelled, (4) additional non-structural systems are not modelled, and (5) multi-directional shaking is not a feature in the computer program used for modelling. The inclusion of such items, especially the rail crane system, would make the model more reflective of actual structural design, but it is assumed that the differences may be negligible.

COMPUTER MODELLING

Since it is obviously not feasible to impart a suite of varying strong ground motions to multiple APR1400 turbine buildings, a computer model was generated instead. A finite element modelling program STAAD Pro v8i was used to develop the APR1400 turbine building model (Bentley 2012) with an example shown in Figure 1.

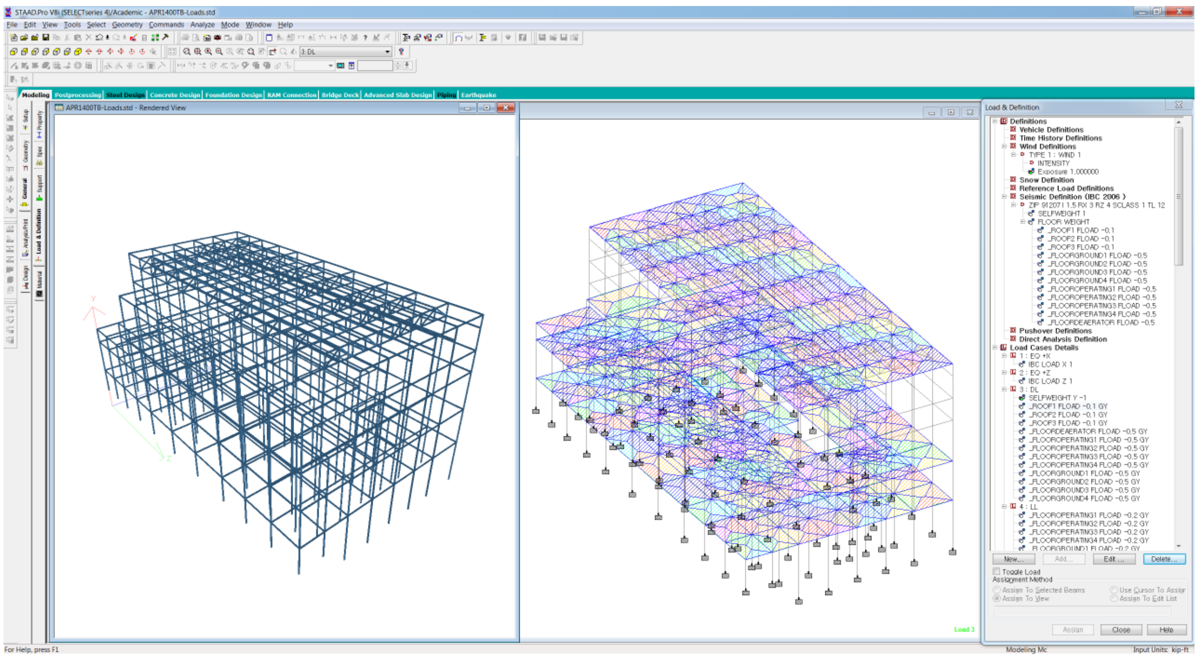

Figure 1. Example of APR1400 turbine building model in STAAD Pro v8i.

Steel members were designed to load resistance and factor design (LRFD) standards as specified in IBC (2009) and AISC 360 (2010). Dead loads for each floor were designated at 4.79 kPa to reflect machinery and 9.58 kPa for the floor housing the deaerator, and 2.39 kPa for the roofing. Roof live loads were considered as 4.79 kPa for maintenance and floor live loads were estimated at 4.79 kPa. Although typically not dominant, wind loads were also considered according to IBC standards, with a basic wind speed of 38 m/s, importance factor of 1.5, and exposure category D. Seismic loads were also estimated in accordance to IBC standards, utilizing the equivalent lateral force procedure, however STAAD uses a zip code location parameter to estimate additional seismic analysis procedure parameters. A zip code for the Los Angeles area, California, USA was used as input and site class was considered as site class A, since most nuclear power plants are founded on rock sites. Seismic bracing was not considered since they are not indicated in the KHNP architectural drawings. Instead, steel special moment resisting frames were assumed, resulting in a seismic response modification factor, R = 8 in both horizontal directions. The author notes that operating APR1400s have concentric bracing in the turbine building and that seismic design is highly site specific. STAAD allows for the automatic generation of load combinations making analyses much easier. These loads and load combinations are used to select structural members, primarily based on minimal material weight criteria. STAAD has a feature that allows the program to select steel members for the user given certain design parameters.

After designing the steel framing, the STAAD model was subjected to a series of ground motions through the time history analysis feature. Characteristics of the ground motions used in this study are shown in Table 1 below (Hashash 2016), with an example ground motion shown in Figure 2. These ground motions will be scaled to cover the spectrum of PGAs needed to derive a preliminary fragility curve.

Table 1: Ground motions used for analysis.

Ground motion name	Date of event	Magnitude	Distance to fault rupture (km)	USGS site class	PGA (g)
Chi-Chi	1999/09/20	7.6	15.29	B	0.183
Coyote	1979/08/06	5.7	17.2	B	0.124
Imperial Valley	1979/10/15	6.5	26.5	B	0.169
Kobe	1995/01/16	6.9	0.6	B	0.821
Kocaeli	1995/08/17	7.4	17.0	B	0.218
Loma Gilroy	1989/10/18	6.9	19.9	B	0.170
Loma Gilroy 2	1989/10/18	6.9	11.6	B	0.357
Mammoth Lake	1980/05/25	6.3	15.5	A	0.430
Nahanni	1985/12/23	6.8	16.0	A	0.148
Northridge	1994/01/17	6.7	26.8	A	0.217
Northridge 2	1994/10/17	6.7	43.4	A	0.098
Parkfield	1966/06/28	6.1	9.9	B	0.357
Whittier Narrows	1987/10/01	6.0	21.1	A	0.186

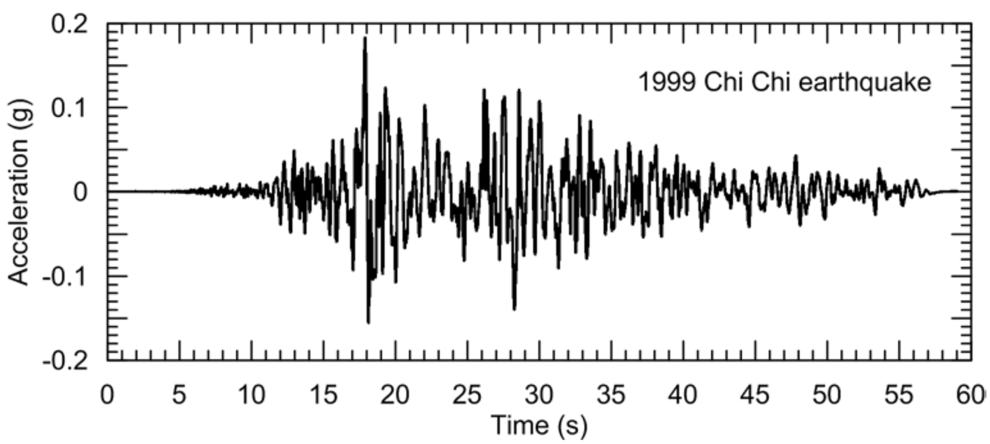

Figure 2. 1999 Chi-Chi ground motion used for time history analysis.

The aforementioned strong ground motions were scaled to varying PGAs such that a range of different PGAs could be utilized for the preliminary fragility curve database. For this study, ground motions were scaled to PGA = 0.05, 0.10, 0.15, 0.20, 0.25, 0.30, 0.35, 0.40, 0.45, 0.50, 0.55, 0.60, 0.65, 0.70, 0.75, 0.80, 0.85, 0.90, 0.95, and 1.00 g and damping was set at 5% for 30 modes, which accounted for approximately 90% participation. Although the practice of scaling ground motions to produce additional ground motions is potentially unrealistic and has a variety of issues in itself, it is a useful procedure to populate the fragility database. Failure was determined when any structural member reached or surpassed the yield strength of steel, exhibited unreasonable deformation, or exceeded code specified allowable story drift. For this preliminary model, the yield strength of steel was 250 MPa and the ultimate strength of steel was 400 MPa and unreasonable deformation was estimated using a code prescribed L/240, where L = length of the steel member. Allowable story drift was estimated from the code as being less than or equal to 0.020h, where h is the story height. Drifts above this value were considered as failure. Drifts were calculated based on the following code prescribed equation:

$$\Delta_x = (\delta_{e2} - \delta_{e1})\frac{C_d}{I} \leq 0.020h_{1,2} \tag{1}$$

where δ_{e2} = average story deformation for level 2 from elastic analysis, δ_{e1} = average story deformation for level 1 from elastic analysis, deflection amplification factor, C_d = 5.5 for special moment frames, importance factor, I = 1.5, $h_{1,2}$ = story height where 1 denotes the level below and 2 denotes the level above confining the story. Story drifts had to be manually compared at each relevant story since STAAD does not differentiate between the turbine housing and the frames housing the balance of plant, such as the deaerator and pumps.

SIMULATION RESULTS AND REGRESSION

Since an elastic analysis was performed, all members did not fail from dynamic loading. Member deformations were also within limits since the turbine building was initially designed to satisfy serviceability requirements. As a result, failure was dictated by exceeding allowable story drift limits. A summary of the conditional probability of failure against PGA is shown in Figure 3 below.

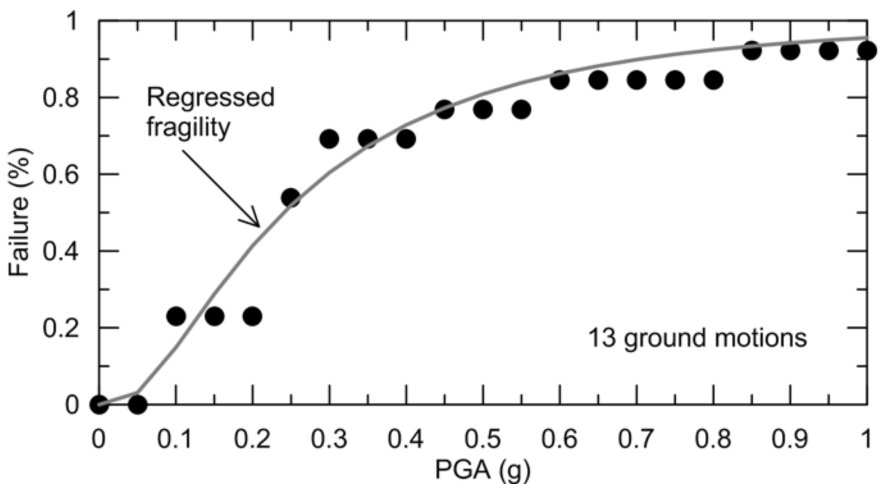

Figure 3. Conditional probability of failure from 13 ground motions for the APR1400 turbine building.

The plot shows a sudden increase in story drift failures after PGA = 0.2 g and does not suggest a “normal” looking cumulative distribution function. The study scaled ground motions to a PGA of 1.0 g since earthquakes above this level are considered unrealistic and extremely rare. A majority of the failures (i.e. exceeding allowable story drift) occurred in the story on the side of the turbine building connecting housing the deaerator floor and in the story housing the operating floor; the story where the turbine generator is housed. Once a specific scaled ground motion forced the turbine building to exceed the allowable story drift limit, then all large scaled ground motions would also exceed the allowable story drift limit. Interestingly enough, all scaled versions of the Whittier Narrows ground motion could not impart enough deformation to exceed the allowable story drift, while the deformations imparted from the Chi Chi, Kobe, and Loma Gilroy ground motions exceeded allowable story drifts even at 0.1 g.

According to EPRI (2002), a median fragility function can be described by the following equation:

$$f' = \Phi\left[\frac{\ln\left(\frac{a}{A_m}\right)}{\beta_R}\right] \tag{2}$$

where a = given peak ground acceleration level, A_m = median ground acceleration capacity, β_R = unknown uncertainty (epistemic uncertainty). The curve shown in Figure 3 has regressed parameters of A_m = 0.24 and β_R = 0.84. The ground acceleration capacity is close to the marketed peak ground acceleration the APR1400 nuclear power plant is designed for, however the uncertainty parameter appears a bit high. This can be attributed to the sudden rise in failures where it seems plateau quite early in the PGA range considered.

DISCUSSION

As mentioned in the introduction, several important items were not considered in the modelling of the APR1400 turbine building that might have affected results, such as a lack of internal loading cranes and an inability to consider multi-directional shaking, would have had an impact on the end results. Although including multi-directional loading effects would most likely increase failure percentages at each PGA level, the larger steel members as a result of the loading cranes and additional support structures would have made the structure and framing more stiff in transverse directions. However, it should be noted that some beams were even sized to be W36x302, which is quite large. Design was difficult given the complexity of the model and lack of detail in the APR turbine building drawings.

To better model the structure, more details along with a more detailed loading plan would help. Additionally, an ability to model multi-directional loading would give a more accurate representation of the structural dynamic behaviour. A larger strong ground motion database would also lend more credibility to the results, covering a variety of ground motion parameters such as duration, frequency content, and perhaps even directivity effects. Moreover, a nonlinear analysis is required to truly evaluate if steel members reach or surpass yield when loaded, especially when ground motions are approaching higher and higher PGA levels. Since STAAD was used to model in one direction for this study, the use of an elastic analysis is acceptable, especially as the lower PGA levels. STAAD is able to perform a nonlinear analysis, but that would be for another study.

CONCLUSION

In this preliminary study, an APR1400 turbine building was modelled using a computer program called STAAD Pro v8i. Using drawings and loading criteria from the international building code, the steel turbine building was designed to withstand minimal wind loads, dead loads, live loads, and seismic loads from the equivalent lateral force procedure for an area in Los Angeles, California, USA. This design was then subjected to a suite of ground motions that were scaled to have a maximum PGA = 0.05, 0.10, 0.15, 0.20, 0.25, 0.30, 0.35, 0.40, 0.45, 0.50, 0.55, 0.60, 0.65, 0.70, 0.75, 0.80, 0.85, 0.90, 0.95, and 1.00 g with damping at 5%. The results showed a sharp increase in conditional probability of failure after PGA = 0.20 g. Using these results, a relationship, as recommended by the Electric Power Research Institute, was regressed for the APR1400 turbine building median fragility with parameters A_m = 0.24 and β_R = 0.84. Although the uncertainty appears large, it is not beyond the range of some nuclear power plant systems, structures, and components.

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