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PERFORMANCE-BASED EARTHQUAKE ENGINEERING METHODOLOGY FOR NUCLEAR CABLE TRAY SYSTEM

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ABSTRACT:

The Pacific Earthquake Engineering Research (PEER) Centre has been developing a performance-based earthquake engineering (PBEE) methodology, which is based on explicit determination of performance, e.g., monetary losses, in a probabilistic manner where uncertainties in earthquake ground motion, structural response, damage estimation, and losses are explicitly considered. To carry out the PEER PBEE procedure for a components of the nuclear power plant (NPP) such as the cable tray system, hazard curve and spectra were defined for two hazard levels of the ground motions, namely, operation basis earthquake, and safe shutdown earthquake. Accordingly, two sets of spectral compatible ground motions were selected for dynamic analysis of the cable tray system. In general, the PBEE analysis of the cable tray in NPP was introduced where the resulting floor motions from the time history analysis (THA) of the NPP structure should be used as the input motion to the cable tray. However, for simplicity, a finite element model of the cable ray was developed for THA under the effect of the selected ground motions. Based on the structural analysis results, fragility curves were generated in terms of specific engineering demand parameters. Loss analysis was performed considering monetary losses corresponding to the predefined damage states. Then, overall losses were evaluated for different damage groups using the PEER PBEE methodology.

1. INTRODUCTION

Cable tray systems are very popular in nuclear power plants (NPP), which are generally multi-span steel structures suspended from the ceiling, or mounted on ground or floor levels of the building structure. The support types include structural frames, cantilever beams, and vertical posts. Trapeze support, L-shaped support, and cantilever support are frequently used in NPPs. They are typically made from cold-formed steel components and designed to provide support to the heavy cables in an earthquake for the electric cables to remain functional. Damage to the cable trays and their supports is permissible and may occur. However, the intent is that cable functionality is maintained and that cable tray or support damage does not jeopardize other nearby equipment. Cable tray belongs to seismic category I (C-I) safety-related structures where its seismic damage under any earthquake excitations should be limited to a certain level. The structural system should maintain the specified design function and structural integrity after safety shut down earthquake (SSE) (IEEE-344, 2013; WEC, 2011). As a result of past earthquake damage to cable tray systems, analytical and experimental studies were carried out by Shahin et al. (1978), Pearce et al. (1984), Eder & Yanev (1988), Smith et al. (1990), Kajitani et al. (2013), Reigles et al. (2016), and Huang et al. (2017). However, the seismic evaluation procedure of NPP cable tray systems is not fully developed.

Earthquakes occur at random times and regions with unforeseen magnitudes and vibration frequencies. Thus, probabilistic seismic assessment of the building structures and cable trays is rational.

Performance-based earthquake engineering (PBEE) is a framework to evaluate seismic hazard, structural response, and the resulting damage and losses (Moehle & Deielein, 2004). Recently, PBEE methodology has been updated to second generation (PBEE-2) by the Pacific Earthquake Engineering Research (PEER) Center. PBEE-2 correlates the probabilistic relationships to components damage and engineering demand parameters (EDPs) (Porter, 2003; Günay & Mosalam, 2013), employs fragility functions in a damage analysis, and quantifies the future seismic performance of buildings and other facilities in terms of probabilistic repair costs, life-safety impacts, and loss of function, summed up as the 3D's (Dollars, Deaths, and Downtime) (Porter et al., 2007, 2010; Yang et al., 2009; ATC, 2017). Unlike building structures, the performance requirement of NPP cable tray systems is extremely high. In this paper, seismic evaluation of these systems is carried out with PEER PBEE-2 methodology. Spectral compatible ground motions are selected for the time history analysis (THA) of the cable tray systems. Performance levels and EDPs are defined to perform the fragility analysis considering structural response, damage, and cost.

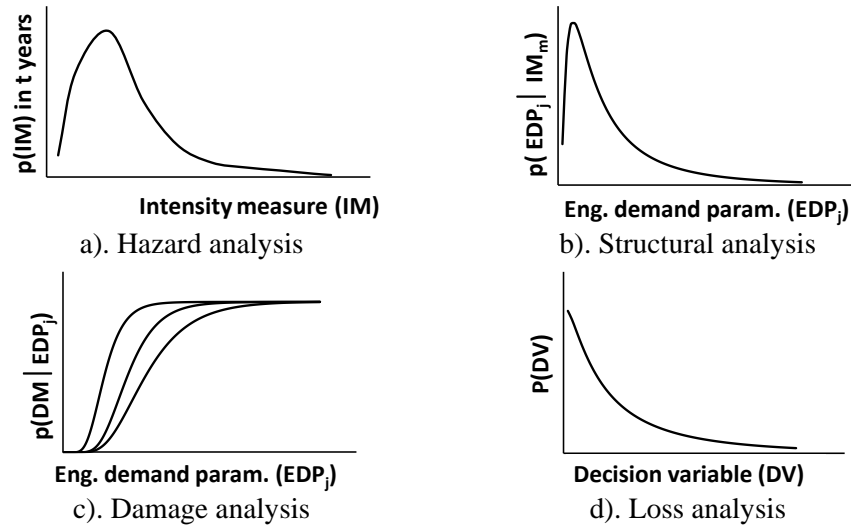


Figure 1. PEER PBEE framework (after Günay & Mosalam 2013)

2. PEER PBEE FRAMEWORK

As demonstrated in Figure 1, the PEER PBEE methodology consists of four analyses: hazard, structural, damage, and loss (Yang et al., 2009; Günay & Mosalam, 2013), where the outcome of each analysis is either a probability or probability of exceedance (POE) distribution (Der Kiureghian, 2005). In earthquake loading, it combines ground motion hazard (IM , Intensity Measure), structural response (EDP , Engineering Demand Parameter), prediction of structural damage (DM , Damage Measure), and prediction of structural loss (DV , Decision Variable). Note that IM , EDP , DM , and DV can be vectors. This methodology is probabilistic in nature, reflecting the uncertainty associated with the seismic performance prediction, and it has been implemented in the updated version of FEMA P-58 (FEMA, 2018a,b). The probabilities determined in each stage are combined using the total probability theorem as described in Eq. (1).

$$P(DV^n) = \sum_m \left(\frac{\sum_j \sum_i \sum_k P(DV_j^n | DM_k) p(DM_k | EDP_j^i) p(NC | IM_m) +}{P(DV^n | C) p(C | IM_m)} \right) p(IM_m) \quad (1)$$

where $p(IM_m)$ is the probability of the m^{th} value of the earthquake IM , determined as an outcome of hazard analysis; $p(EDP_j^i | IM_m)$ is the probability of the i^{th} value of the EDP utilized for the j^{th} damageable group, when the m^{th} value of IM occurs (outcome of structural analysis), $p(DM_k | EDP_j^i)$ is the probability of the k^{th} DM when subjected to the i^{th} value of the EDP utilized for the j^{th} damageable group (outcome of damage

analysis) and $P(DV_j^n | DM_k)$ is the POE of the n^{th} value of the DV for the j^{th} damageable group when the k^{th} DM occurs (outcome of loss analysis). Moreover, $p(C | IM_m)$ and $p(NC | IM_m)$ are the probabilities of having and not having global collapse, respectively, under ground motion IM_m . Finally, $P(DV^n | C)$ is the POE of the n^{th} value of the DV in the case of global collapse. Note that the POE of the n^{th} value of a DV in Eq. (1), $P(DV^n)$, is interpreted as a weighted average of the POE of DVs from all possible cases, $P(DV_j^n | DM_k)$, where the weight of each case is defined by the probability of a specific IM , EDP , and DM combination, i.e. $p(DM_k | EDP_j^i) p(EDP_j^i | IM_m) p(NC | IM_m)$. Another useful indicator is the expected value of DV (Der Kiureghian, 2005), $E(DV)$, which can be calculated using Eq. (2).

$$E(DV^n) = \sum_m \left(\sum_j \sum_i \sum_k E(DV_j^n | DM_k) p(DM_k | EDP_j^i) p(NC | IM_m) + E(DV^n | C) p(C | IM_m) \right) p(IM_m) \quad (2)$$

Although, the PEER PBEE methodology is able to produce a full probability distribution of DV during a considered time span, e.g. 50 years, in some situations, it may be useful to determine the loss in case of a specific hazard level taking place, especially for the investigation of an important public facility. In such cases, the POE and expected value of the DV are given by Eqs. (3) and (4), respectively, with $p(IM_m) = 1.0$.

$$P(DV^n) = \sum_j \sum_i \sum_k P(DV_j^n | DM_k) p(DM_k | EDP_j^i) p(NC | IM_m) + P(DV^n | C) p(C | IM_m) \quad (3)$$

$$E(DV) = \sum_j \sum_i \sum_k E(DV_j | DM_k) p(DM_k | EDP_j^i) p(NC | IM_m) + E(DV^n | C) p(C | IM_m) \quad (4)$$

In hazard analysis, the resulting hazard curve is produced by examining the seismic environment (e.g. nearby faults, their magnitude-recurrence rates, fault mechanism, source-site distance, site conditions, etc.) of the considered site, to describe the earthquake hazard in a probabilistic manner and provide the POE of each possible value of an IM (e.g. peak ground acceleration, or spectral acceleration at a specific period). In the structural analysis, a finite element model of the cable tray is used to determine the nonlinear dynamic response using the selected ground motions. In damage analysis, the DM of each damage group with different values of the EDP associated with that damage group is determined probabilistically resulting in different fragility functions, which are described herein by lognormal distributions with median and coefficient of variation (COV) values experimentally determined. In the loss analysis, the POE of different values of the chosen DV , e.g. economic losses, downtime and fatalities, is described for all the damage measures of all damageable groups. Lack of information on the detailed probabilistic distribution model of occupants and indirect losses (such as repair time and electricity generation interruption) for the cable tray system, monetary loss corresponding to components' replacement cost is the considered DV in this study.

3. PBEE ANALYSIS

3.1 Cable Tray in NPP Building

If the cable tray is installed in an NPP building, floor response analysis of the primary NPP structure should be performed with selected ground motions. For the NPP structure, seismic hazard analysis should be performed to define the target response spectrum for ground motion selection. Alternatively, code recommended spectrum is applicable considering the site-specific and hazard properties of the NPP structure. Regulatory Guidelines issued by the U.S. Nuclear Regulatory Commission (NRC) (NRC 2014) was used to define target response spectra in the horizontal and vertical directions. The design-level target spectra are set at a peak ground acceleration of 0.1g. Response spectra of selected 20 records are shown in Figure 2. A simplified two-stick OpenSees model (2014) was developed based on information from Shao et al. (2017) (Figure 3), where, the shorter and taller sticks represent an auxiliary building (AUX) and a reactor containment building (RCB), respectively. The origin of the coordinate system for the analysis model is located at the center of the RCB at ground elevation. Beam-column elements were used to model this lumped-mass NPP superstructure. The common base slab was modeled using shell elements; therefore, it had distributed weight and mass properties. The base slab was fixed and soil-structure-interaction effects

were not considered in this parametric analysis. Eigen value analysis shows that the fundamental frequencies of the AUX and RCB substructures are 4.284, and 5.010 Hz, respectively.

The floor motion time histories of each floor can be acquired under the excitation of each selected ground motion. As an example, the floor responses of the AUX model under the San Fernando earthquake (Shao et al., 2017) were computed and the corresponding time histories and response spectra are shown in Figure 4 (input peak ground acceleration is 0.1g). From this figure, it is shown that the acceleration responses in the higher stories are larger than those in the lower stories. The corresponding floor response spectra have the same trend. Ignoring the dynamic interaction between the main structure and the cable tray, THA of the cable tray structure (Figure 7) can be carried out using the resulting floor motions from the dynamic analysis of the main structure for the period range from 0.1 to 0.4 sec (Figure 4b), where response approaches zero after 0.5 sec. Then, PEER PBEE can be performed using the approaches discussed in Sec. 2 of this paper.

The cable tray in the NPP does not involve the seismic hazard analysis and ground motion selection. Thus, to carry out the complete process of PEER PBEE, the cable tray mounted on the ground level is analyzed completely in next section. The other PBEE analyses, i.e. structural, damage, and loss are similar in the two scenarios of the cable tray being in NPP or on ground.

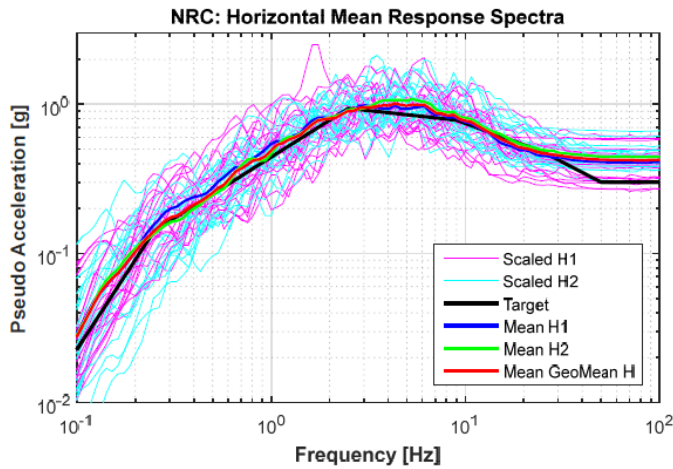


Figure 2. NRC compatible response spectra (H1 and H2 denote the two horizontal directions) (Shao et al., 2017)

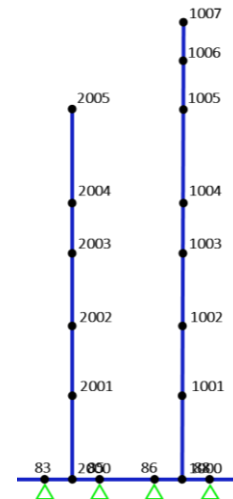
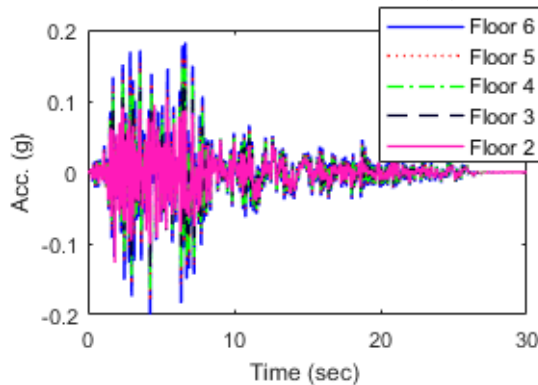
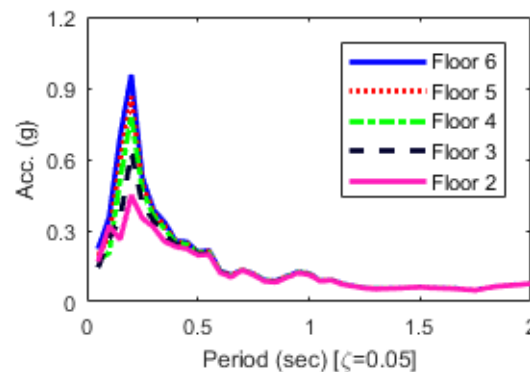


Figure 3. Simplified OpenSees model of the NPP structure



a) Floor motion time histories



b) Floor response spectra (Damping ratio = 0.05)

Figure 4. Floor response under San Fernando earthquake (AUX model)

3.2 Cable Tray Mounted on the Ground Level

3.2.1 Seismic Hazard and Ground Motions

In the seismic performance evaluation of the cable tray in NPPs, two levels of earthquakes are considered, namely, the operation basis earthquake (OBE) and safe shutdown earthquake (SSE). The corresponding

probability of occurrences are 10%, and 1%, respectively, in 50 years (WEC, 2011). This also meets the specifications in ASCE 43-05 (2005). Current codes provide details to perform seismic hazard analysis, and define the design response spectrum (ASCE, 2005; USNRC, 2007, 2014). Moreover, site specific seismic analysis is suggested for more reliable hazard probability. This study assumes the cable tray systems to be located in San Francisco, where the latitude is 37.8° , longitude is -122.417° , and the site is Class B. The spectral acceleration at the period of the first mode, $S_a(T_1)$, is one of the commonly used parameters as *IM*. Shaking table testing results showed that the fundamental periods were 0.15 (longitudinal) and 0.1 (transversal) sec (Huang et al., 2017), which were used as the spectral acceleration (S_a) periods in the Hazard Curve Calculator application of OpenSHA (Field et al., 2003), Figure 5a, where the mean annual frequency of exceedance of S_a in the longitudinal direction was slightly larger than that in the transversal direction. To obtain the site-specific hazard spectra, and carry out PBEE-2 procedure, OpenSHA (Field et al., 2003) was used and two hazard response spectra were generated, Figure 5b. Subsequently, two sets of ground motions were selected from the PEER NGA-West2 database (Ancheta et al. 2013) with 15 motions for OBE and 12 motions for SSE (Tables 1 and 2). In each set, only one ground motion record with the smallest scale factor was selected from each earthquake scenario. The response spectra of the selected motions matched to the target hazard spectra well without scaling as suggested by Sommerville and Porter (2005) (Figures 5c and d). These motions were used in THA of the cable tray system.

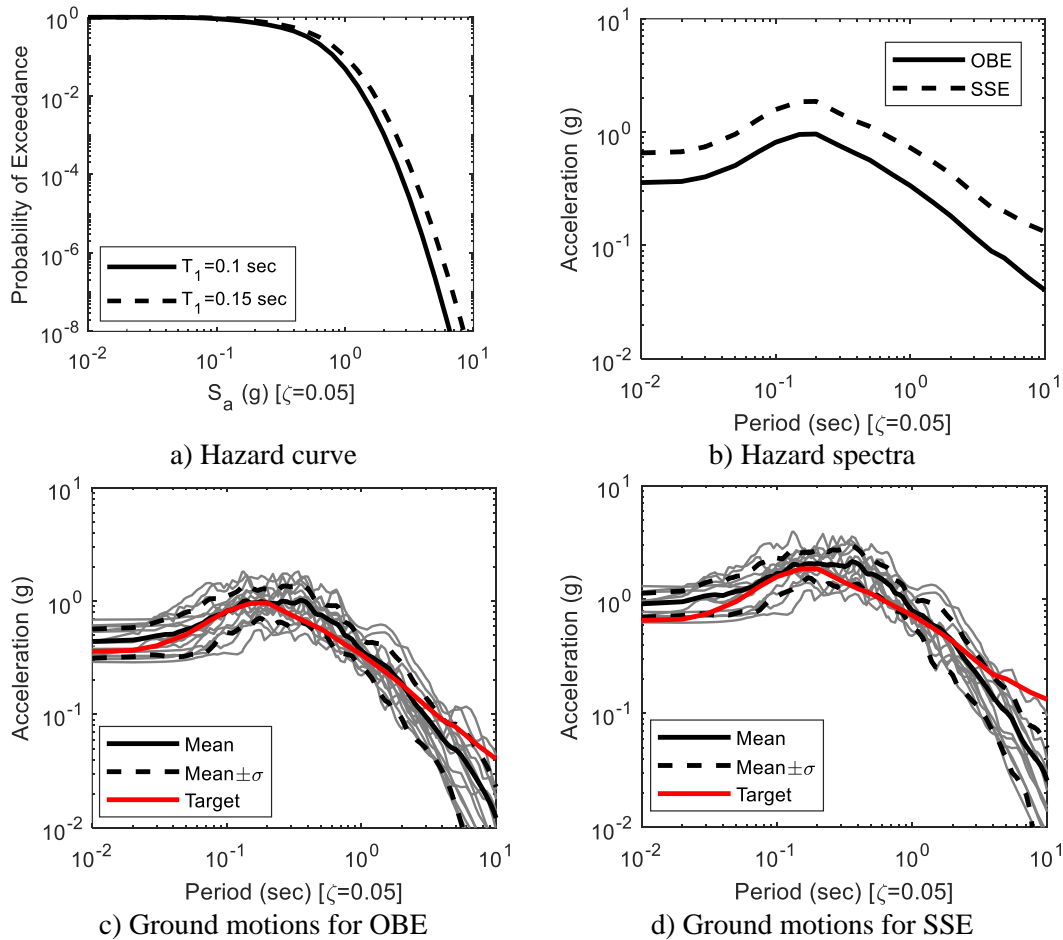


Figure 5. Hazard curve and spectra

3.2.2 Structural Analysis

The purpose of the structural analysis is to determine the response of a structure to various levels and characteristics of earthquake hazard in a probabilistic manner. A multi-span steel cable tray system with 1.067 m width, 1.5 m height and 7.2 m length, was considered in this study, Figure 6a, with two types of

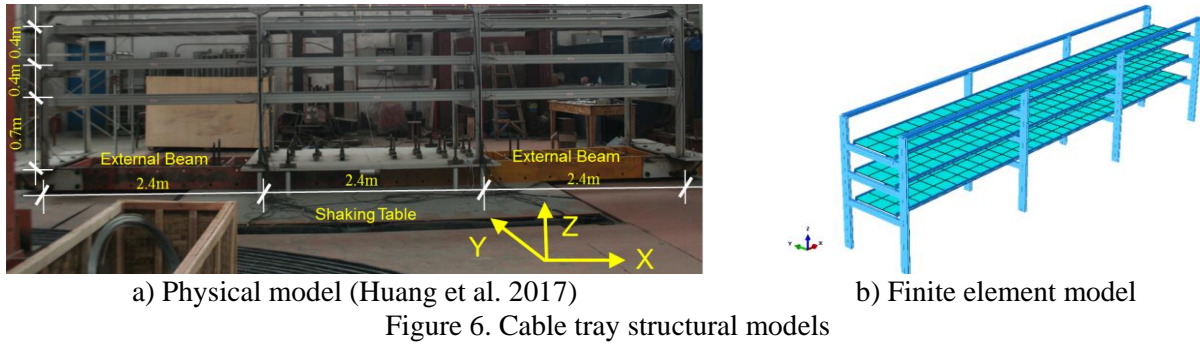
trays situated in three tiers: the ladder-type tray in the top tier, and the trough-type tray in the first two tiers. The self-weight of the cables in each tier was 468.0 kg corresponding to 100% loading (Huang et al., 2017). A finite element model, Figure 6b was developed in ABAQUS (Dassault Systemes 2016). *B31* elements were used for columns and beams, and the cables and pallets in each story were simulated with *S4* linear shell elements with 3.0 mm depth. The mass was modeled with nonstructural mass element. The bolted connections in the beam-column joints had a certain capability of rotation in seismic load, thus they were modeled with *joint-rotation* elements with a rotational stiffness of 10^4 N/rad. Shaking table testing results demonstrated the relative displacement between the pallet and support beams (Huang et al., 2017) in direction X of Figure 6a. This effect was represented by *bushing* and *slot-rotation* elements which fail once the dislocation exceeded 10.0 mm. The mechanical properties of the steel were simulated with elastoplastic model in (Esmaily and Xiao 2005). The damping ratio of the whole model was 5% as suggested in ASCE 43-05 (2005) and in (Huang et al. 2017). The eigen value analysis demonstrated that the fundamental vibration frequencies were 6.8 and 9.8 Hz, for 0% loading ratio, and 5.1 and 7.0 Hz, for 100% loading ratio, respectively, which are almost identical to those obtained from the shaking table tests (Huang et al., 2017).

Table 1. Ground motions for OBE

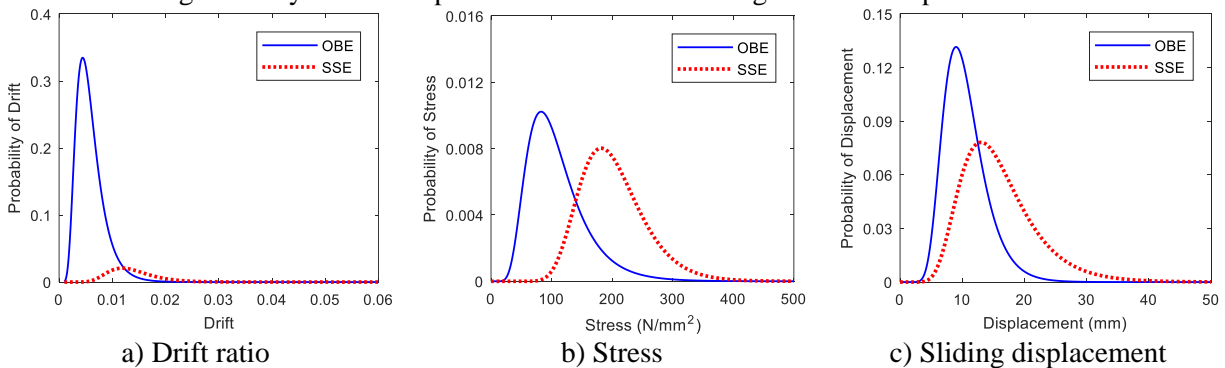
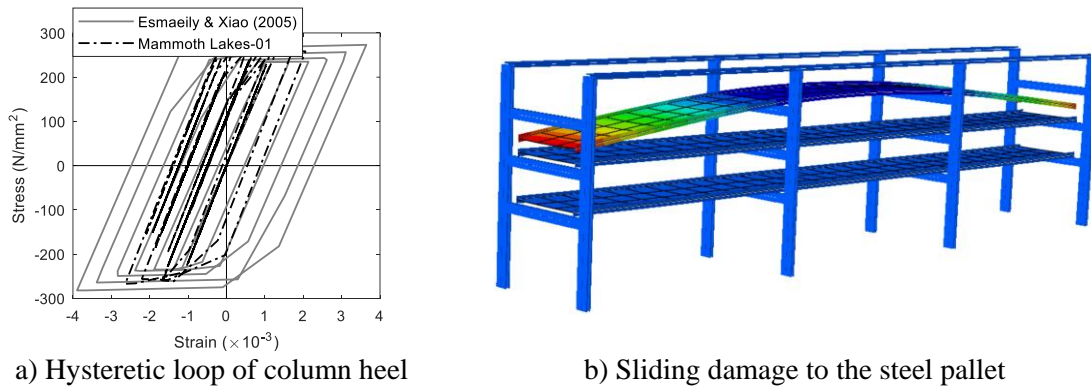
No.	Earthquake Name	Year	Magnitude	R_{rup} (km)	V_{s30} (m/sec)
1	Helena_Montana-01	1935	6.00	2.86	593.35
2	Imperial Valley-02	1940	6.95	6.09	213.44
3	Northern Calif-01	1941	6.40	44.68	219.31
4	Borrego	1942	6.50	56.88	213.44
5	Kern County	1952	7.36	38.89	385.43
6	Northern Calif-03	1954	6.50	27.02	219.31
7	Parkfield	1966	6.19	9.58	289.56
8	Borrego Mtn	1968	6.63	45.66	213.44
9	San Fernando	1971	6.61	22.77	316.46
10	Managua_Nicaragua-01	1972	6.24	4.06	288.77
11	Friuli_Italy-01	1976	6.50	15.82	505.23
12	Gazli_USSR	1976	6.80	5.46	259.59
13	Tabas_Iran	1978	7.35	13.94	471.53
14	Imperial Valley-06	1979	6.53	10.45	231.23
15	Mammoth Lakes-01	1980	6.06	6.63	382.12

Table 2. Ground motions for SSE

No.	Earthquake Name	Year	Magnitude	R_{rup} (km)	V_{s30} (m/sec)
1	Imperial Valley-02	1940	6.95	6.09	213.44
2	Kern County	1952	7.36	38.89	385.43
3	Northern Calif-03	1954	6.50	27.02	219.31
4	Parkfield	1966	6.19	9.58	289.56
5	Borrego Mtn	1968	6.63	45.66	213.44
6	San Fernando	1971	6.61	22.77	316.46
7	Managua_Nicaragua-01	1972	6.24	4.06	288.77
8	Friuli_Italy-01	1976	6.50	15.82	505.23
9	Gazli_USSR	1976	6.80	5.46	259.59
10	Tabas_Iran	1978	7.35	13.94	471.53
11	Imperial Valley-06	1979	6.53	2.66	223.03
12	Mammoth Lakes-01	1980	6.06	6.63	382.12



The nonlinear THA of the numerical model was conducted using the selected ground motions in Tables 1 and 2 to investigate the probability distribution of *EDPs* defined consistent with damageable groups as a part of structural analysis. In this study, two damage groups were considered, namely the steel pallets and the columns, including the heels. The stress-strain hysteresis loops of the column heel under Mammoth Lakes-01 earthquake was very large showing stress hardening, Figure 7a, where the largest stress exceeded the design strength of 210 N/mm². Due to the weak connection between the steel pallet and the frame, the dislocation of the steel pallet took place and fell from the support beams, Figure 7b. Based on the properties of the dynamic response of the cable tray system, the *EDPs* were defined as drift ratio (peak displacement of the column top divided by the column height), stress of the column heel, and relative horizontal sliding displacement between the steel pallet and the tray. In the SSE, the dynamic responses corresponding to the *EDPs* were larger than that in the OBE. For each *EDP*, lognormal distribution was assumed with the median and coefficient of variation values calculated from the nonlinear THA with the selected ground motions. The resulting probability distributions of drift, stress, and sliding displacement for the considered hazard levels (OBE and SSE) are shown in Figure 8, where the magnitude of the drift in OBE was larger than that in the SSE. These probabilities correspond to $p(EDP_j^i | IM_m)$ in Eq. (1).



3.2.3 Damage Analysis

The objective of the damage analysis is to estimate physical damage at the component or system levels as a function of the structural response. *DM* is generally defined in terms of damage states corresponding to the repair measures necessary to restore the components of a facility to its original conditions (Porter, 2003). In the cable tray system, damage states of the steel pallets and columns should be defined prior to damage analysis. Accordingly, specific values of *EDP* corresponding to various *DMs* with different probabilities were determined with fragility functions representing the POE of a *DM* for different values of *EDP*. Fragility functions were obtained for the three damage groups, i.e. steel columns, column heels, and steel pallets. Based on the experimental tests (Huang et al., 2017) and numerical analysis results, the damage states for the considered steel columns were slight, moderate, and severe with corresponding *EDPs* represented by drift ratios in Table 3 with respective COVs. The resulting fragility curves are shown in Figure 9a. For the steel column heels, the resulting damage states were defined as elastic, elastoplastic, and plastic with correlated *EDP* represented as the loading stress of the steel. Based on the Chinese code provisions (TJU, 2013) and finite element analysis results, the median stress for these states were 160, 220 and 240 N/mm². Considering the advanced steel manufacturing technology and qualified construction procedures, the corresponding COVs were taken as 0.10, Table 3. With these parameters, the resulting fragility curves are shown in Figure 9b. For the steel pallets, the damage states were functional, bolted connection damage, and falling of the cable, and represented by the sliding displacement mentioned in Sec. 3.2.4. From the shaking table tests (Huang et al., 2017) and time history analysis, the corresponding median values were 8.0, 15.0, and 30.0 mm, respectively, Table 3. The associated COVs were 0.2, 0.26, and 0.15. Then the fragility curves were generated as shown in Figure 9c. It should be noted that sever damage to the steel columns is expected to lead to the scenario of collapse of the cable tray system.

Table 3. Median and COV of *EDPs* for different damage levels

Component	Damage state	EDP	Median	COV
Steel Column	Slight	Drift ratio	0.01	0.20
	Moderate		0.02	0.20
	Severe		0.05	0.20
Column heel	Elastic	Stress (N/mm ²)	160	0.10
	Elastoplastic		220	0.10
	Plastic		240	0.10
Steel pallet	Functional	Sliding displacement (mm)	8.0	0.21
	Connection damage		15.0	0.26
	Cable falling		30.0	0.15

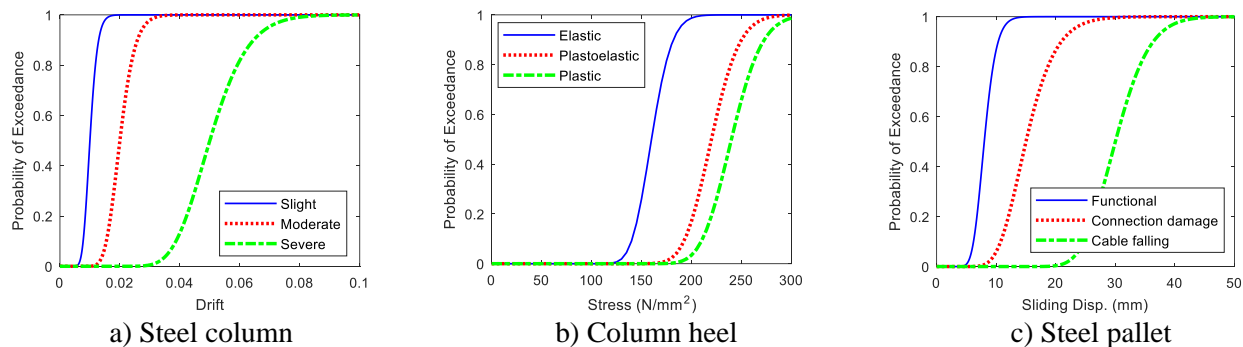


Figure 9. Fragility curves

3.2.4 Loss Analysis

Loss analysis is performed to translate damage quantities to *DVs* that can be used by stakeholders or engineers to make risk management decisions. In this stage, the POE of the chosen *DVs* such as economic losses, downtime, and fatalities, is described for the *DMs* of the damageable groups. For the predefined

damage groups of the cable tray system, monetary loss was considered in terms of repair cost, while indirect losses, e.g. repair time or social impact, were not considered herein. The repair cost correlated to the specific damage states is listed in Table 4, based on the experimental specimen elaborated in (Huang et al. 2017) for typical expenses in Shanghai, China. For the cable trays in a real NPP, the cost is expected to be higher because of longer physical distributions. Furthermore, the cost may vary among different construction companies, construction quality, labor, etc. However, the market in the cable trays has been fully developed and the variance in a certain district is small. Therefore, the COVs for the three damage groups were taken as 0.15, 0.15, and 0.10. The former two values were larger due to more uncertainties than the last one. In the monetary loss evaluation process of the steel columns, the repair cost of the column consists of the corresponding column heel, subsequently, only loss curves of the steel columns were generated. All loss functions were assumed of lognormal distributions with resulting fragility curves on repair in Figure 10.

The resulting total loss curve obtained using Eq. (1) is shown in Figure 10d. The POE of the monetary loss of No-collapse was very low such that the total loss was dominated by the collapse scenario. The No-collapse plot can be interpreted as the loss curve for a hypothetical case where collapse was prevented for all intensity levels. The significant reduction in economic loss as a result of eliminating collapse shows the effect of the “collapse prevention” mandated by the seismic codes from an economical perspective.

Table 4. Parameters used to define the repair cost function

Component	Damage state	Repair measure	Quantity	Repair cost (¥/\$)
Steel Column	Slight	Structural evaluation	8	1700/251
	Moderate	Strengthen the column	8	2800/413
	Severe	Replace the column	8	4900/723
Column heel	Elastic	Structural evaluation	8	500/74
	Elastoplastic	Strengthen the joint	8	800/118
	Plastic	Replace the joint	8	1700/253
Steel pallet	Functional	Retighten connection & cable	24	200/30
	Connection damage	Replace connection	12	1400/208
	Cable falling	Replace pallet	3	2900/431

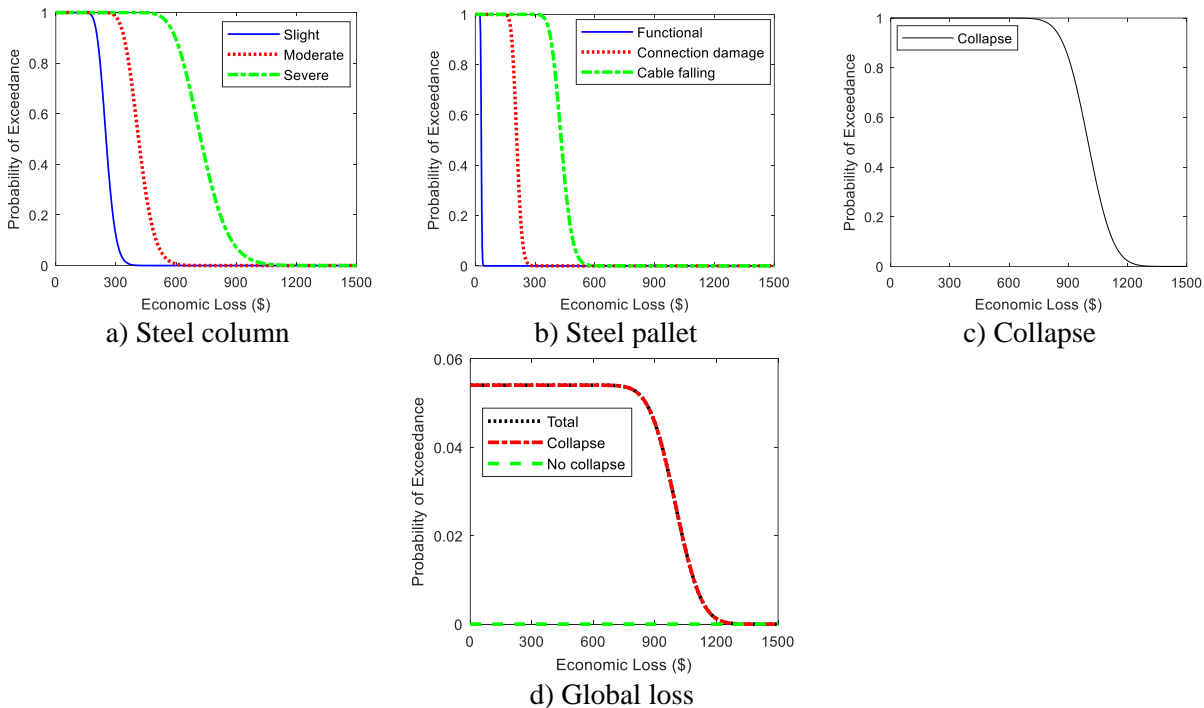


Figure 10. Loss curves

4. CONCLUSIONS

Seismic fragility analysis of the NPPs has been studied by several researchers (Kennedy and Ravindra, 1984; Pisharady & Basu, 2010; Zentner, 2010; Wang et al., 2018), while the seismic performance of the cable tray system in NPPs is rarely investigated in the literature. PEER PBEE framework was applied to the cable tray system where hazard analysis, ground motion selection, structural analysis, and loss analysis were performed, some conclusions are addressed below.

- 1) To carry out PBEE of a cable tray system, hazard curve and spectra were defined for two hazard levels of the ground motions, i.e. OBE, and SSE Accordingly, two sets of spectral compatible ground motions were selected for dynamic analysis.
- 2) A finite element model of the cable ray was developed for THA under the excitation of the selected ground motions. The corresponding *EDPs* of peak drift ratio for the columns, sliding displacement for the steel pallets, and the stress level of the column heels were probabilistically investigated.
- 3) Two damage groups, namely the steel columns, and steel pallets were analysed based on the results of the shaking table tests (Huang et al., 2017). Median and COV of the *EDPs* were computed based on the structural analysis results, then fragility curves were generated using lognormal distributions.
- 4) The POE of economic losses of the high damage states was larger than that of lower ones. Damage of the steel columns led to fatal damage of the global structure where the whole economic loss dominated by collapse. A structural engineer can use the total loss curve to make a decision on the structural design of the cable tray system benefiting stakeholders from the PBEE methodology.
- 5) PBEE analysis of the cable tray in NPP was introduced and the resulting floor motions from the time history analysis of the NPP structure should be used as the input motion to the cable tray.

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