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The Korea Atomic Energy Research Institute (KAERI) and Brookhaven National Laboratory are conducting a collaborative research project to develop seismic capability evaluation technology for degraded structures and components in nuclear power plants (NPPs). One of the goals of this collaboration endeavor is to develop seismic fragility analysis methods that consider the potential effects of age-related degradation of structures, systems, and components (SSCs). The essential part of this collaboration is aimed at achieving a better understanding of the effects of aging on the performance of SSCs and ultimately on the safety of NPPs.

A recent search of the degradation occurrences of structures and passive components (SPCs) showed that the rate of aging related degradation in NPPs was not significantly large but increasing, as the plants get older. The slow but increasing rate of degradation of SPCs can potentially affect the safety of the older plants and become an important factor in decision making in the current trend of extending the operating license period of the plants (e.g., in the U.S. from 40 years to 60 years, and even potentially to 80 years). The condition and performance of major aged NPP structures such as the containment contributes to the life span of a plant. A frequent misconception of such low degradation rate of SPCs is that such degradation may not pose significant risk to plant safety. However, under low probability high consequence initiating events, such as large earthquakes, SPCs that have slowly degraded over many years could potentially affect plant safety and these effects need to be better understood.

As part of the KAERI-BNL collaboration, a condensate storage tank (CST) was analyzed to estimate its seismic fragility capacities under various postulated degradation scenarios. CSTs were shown to have a significant impact on the seismic core damage frequency of a nuclear power plant. The seismic fragility capacity of the CST was developed for five cases: (1) a baseline analysis where the design condition (undegraded) is assumed, (2) a scenario with degraded stainless steel tank shell, (3) a scenario with degraded anchor bolts, (4) a scenario with anchorage concrete cracking, and (5) a perfect correlation of the above three degradation scenarios. This paper will present the methodology for the time-dependent fragility calculation and discuss the insights drawn from this study.

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ABSTRACT

To achieve a better understanding of the effects of aging on the performance of structures and passive components (SPCs) in nuclear power plants (NPPs), the Korea Atomic Energy Research Institute (KAERI) and Brookhaven National Laboratory (BNL) are collaborating to develop seismic fragility analysis methods that consider age-related degradation of SPCs. The rate of age-related degradation of SPCs was not found to be significantly large, but increasing as the plants get older. The slow but increasing rate of degradation of SPCs can potentially affect the safety of the older plants and become an important factor in decision making in the current trend of extending the operating license period of the plants (e.g., in the U.S. from 40 years to 60 years, and even potentially to 80 years). In this paper, a condensate storage tank (CST) was analyzed to estimate its seismic fragility capacities under various postulated degradation scenarios. This paper will present the methodology for the time-dependent fragility calculation and discuss the insights drawn from this study.

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1. INTRODUCTION

The Korea Atomic Energy Research Institute (KAERI) and Brookhaven National Laboratory (BNL) are conducting a collaborative research project to develop seismic capability evaluation technology for degraded structures and components in nuclear power plants (NPPs). One of the goals of this collaboration endeavor is to develop seismic fragility analysis methods that consider the potential effects of age-related degradation of structures, systems, and components (SSCs). The essential part of this collaboration is aimed at achieving a better understanding of the effects of aging on the performance of SSCs and ultimately on the safety of NPPs.

A recent search of the degradation occurrences of structures and passive components (SPCs) showed that the rate of aging related degradation in NPPs was not significantly large but increasing, as the plants get older (Nie, et al., 2008). The slow but increasing rate of degradation of SPCs can potentially affect the safety of the older plants and become an important factor in decision making in the current trend of extending the operating license period of the plants (e.g., in the U.S. from 40 years to 60 years, and even potentially to 80 years). The condition and performance of major aged NPP structures such as the containment contributes to the life span of a plant. A frequent misconception of such low degradation rate of SPCs is that such degradation may not pose significant risk to plant safety. However, under low probability high consequence initiating events, such as large earthquakes, SPCs that have slowly degraded over many years could potentially affect plant safety and these effects need to be better understood.

Although the age-related degradation of SPCs is fundamentally important to the safety of NPPs, research results that can lead to good prediction of long-term performance of the SPCs are rare (Nie, et al., 2009). Through a recent revisit to references generated in the NRC structural aging (SAG) program (e.g., Naus, et al., 1991, 1996, Oland, et al., 1993, among others), it was confirmed that very limited data were available for long-term environment-dependent material properties at the time of this large-scale research project. One exception is the change in compressive strength of concrete over time, which is well known and is available through public resources. Therefore, a couple of material degradation models used in this study come from a recent extensive search and review for time-dependent material models.

As part of the KAERI-BNL collaboration, a condensate storage tank (CST) was analyzed to estimate its seismic fragility capacities under various postulated degradation scenarios. CSTs were shown to have a significant impact on the seismic core damage frequency of a nuclear power plant, contributing 17.7% to the seismic core damage frequency for a Korean nuclear power plant (Choun, et al., 2008). This reference showed that the CST ranked as the 3rd among all considered components (diesel generator and offsite power ranked the first two) and ranking it the 1st among all SPCs. The seismic fragility capacity of the CST was developed for five cases: (1) a baseline analysis where the design condition (undegraded) is assumed, (2) a scenario with degraded stainless steel tank shell, (3) a scenario with degraded anchor bolts, (4) a scenario with anchorage concrete cracking, and (5) a perfect correlation of the above three degradation scenarios. This paper will present the methodology for the time-dependent fragility calculation and discuss the insights drawn from this study (Nie, et al., 2010).

2. THE CDFM METHOD FOR TANK FRAGILITY ANALYSIS

Two methods, namely the conservative deterministic failure margin (CDFM) method and Fragility Analysis (FA) method were introduced in NUREG/CR-5270 (Kennedy, et al., 1989) to estimate the seismic margins of NPP SSCs. The seismic margin of a component is defined in these methods as the high confidence low probability of failure (HCLPF) capacity. The procedure to obtain the HCLPF capacity of a component requires the estimation of its seismic response as a function of the seismic margin earthquake (SME) and its seismic capacity. The CDFM method conservatively prescribes values for the parameters and requires some level of subjective decisions in formulating the procedures; it produces a deterministic HCLPF capacity. On the other hand, the FA method requires the determination of the median and the associated uncertainties (β_R and β_U), which are under substantial subjective judgment; this method yields an HCLPF capacity as well as the overall randomness β_R and uncertainty β_U. The CDFM method was developed for simplicity based on the FA analysis method, such that the HCLPF capacity can be calculated deterministically without specifying many subjective parameters.

In the CDFM method, a set of deterministic guidelines are specified to prescribe the selection of strength, damping, ductility, load combination, structural model, soil-structural interaction, in-structural response spectra, etc, in the fragility calculation. This method follows the design procedures commonly used by the industry, except that some parameters are chosen differently. It is therefore easy to be implemented and accepted by fragility analysts. The selection of the parameters is somewhat judgmental to account for the margins and uncertainties. The goal of this method is to obtain conservative but somewhat realistic HCLPF capacities.

The CDFM method is used herein in the fragility analysis of the undegraded and degraded CST.

3. FRAGILITY ANALYSIS OF THE UNDEGRADED CST

The CST analyzed in this study is located in the Ulchin nuclear power plant, which is located on the east side of Korea on the coast of the Pacific Ocean. Two CSTs are built close to each other, with a center-to-center separation of 89' (27.13 m). There is an auxiliary building between the two CSTs, with the roof about 13 feet above the tank foundation. Figure 1 shows a photo of the CSTs and the auxiliary building. The shell plate, bottom plate, and the roof plate of the tank are made of SA240-304 stainless steel.

FIGURE 1 THE CONDENSATE STORAGE TANKS

The CST is a flat-bottom cylindrical tank filled with water and under atmospheric pressure. The inner diameter of the tank is 50' (15.24 m) and the height of tank (up to the design water level) is 37'-6" (11.43 m). The thickness of the tank shell is 5/8" (16 mm). The thickness of the bottom plate is about 7 mm.

The CST is heavily anchored to the reinforced concrete foundation through 78 anchor bolts. The anchor bolts have a diameter of 2-1/2" (63.5 mm) and are A36 steel. The length of the anchor bolts is 3'-6" (1.07 m), with an embedment of about 2'-1" (0.64 m). The anchor bolts were post-installed in pre-formed holes in the concrete foundation with non-shrinking grout. The compressive strength of the concrete foundation of the CSTs was specified as 4,500 psi. In a test that was setup for very similar CSTs in another Korean NPP, the actual 7 day and 28 day compressive strengths of the concrete were measured to be 5,419 psi and 7,180 psi, respectively. The actual compressive strength of the non-shrinking grout was reported to be 7,550 psi and 111,000 psi, respectively, at 7 days and 21 days (Lee, et al., 2001).

The tank is founded on a rock site. Therefore, soil-structure interaction (SSI) is not relevant to the subject CST.

A sophisticated procedure to calculate the HCLPF capacity of flat bottom tanks using the CDFM method is introduced in Appendix A of NUREG/CR-5270 (Kennedy, et al., 1989). This procedure involves an extensive set of equations to calculate the seismic responses and seismic margin capacities. The mathematical software Mathcad (2007) was chosen as the computational tool for this study because of: (1) its capability in explicitly expressing mathematical equations, (2) its advanced functions in performing interpolation and root finding without significant programming, (3) its capability in mixing documentation and calculation, and (4) its instant numerical calculation and plot rendering when any parameters are varied. The utilization of this tool saved considerable time that would be used in developing a spreadsheet or in-house code.

The calculation of the HCLPF capacity using the CDFM method follows mostly the recommendations in NUREG/CR-5270, supplemented with BNL 52361 (Bandyopadhyay, et al., 1995), ASCE 4-98 (2000), NASA SP-8007 (1968), and other references.

The detail of the analysis can be found in Reference 7. A summary of the analysis and the results will be presented in the following.

The design basis earthquake (DBE) used for the design of the subject CST was based on NRC Regulatory Guide (RG) 1.60 (1973) design spectrum anchored to a PGA level of 0.20 g. Therefore, the NRC RG 1.60 spectrum shapes for the horizontal ground motion and the vertical ground motion were used for the HCLPF capacity evaluation. The RG 1.60 spectrum shapes were implemented in Mathcad using its interpolation function to automatically determine the spectral acceleration for any given frequency. In addition, the initial SME estimate is set to 1.67×0.2 g = 0.334 g, in which the factor 1.67 comes from the SRM/SECY 93-087 (1993) requirement that the HCLPF capacity shall be greater than or equal to 1.67 times the safe shutdown earthquake (SSE) in a margin assessment of seismic events. After several iterations by trial-and-error, the SME capacity converges to 0.426g, which is governed by the sliding mode. It is important to emphasize that the estimated HCLPF SME capacity is conditioned on the RG 1.60 response spectra anchored to 0.426 g. At this capacity, the calculated SME's based on the overturning moment and the fluid pressure response modes were 1.1 g and 2.1 g, respectively. Other failure modes, such as slosh height for roof damage and interaction of tank-auxiliary building, were assessed and were not determined to be governing.

This HCLPF SME capacity estimate is very close to the value reported by Choun, et al (2008), which is 0.41 g and also sliding mode governs. This good agreement validates the accuracy of the calculation implemented in Mathcad and provides confidence in the results of the fragility analyses of the degraded CST because the calculation procedures for these analyses were derived from the undegraded case.

Uncertainties β_R and β_U are required to develop the fragility curve of the CST. Since the CDFM method relies on deterministic but conservative parameters and only yields the HCLPF capacity, the uncertainties are not available in this analysis. As commonly understood, the uncertainties are very much subjective; therefore their determination depends on a significant level of expertise. In this study, a full examination of the uncertainties associated with the CST was not performed. Instead, the uncertainties in various parameters, especially the resultant uncertainties associated with the median fragility of the example tank in NUREG/CR-5270, were used directly, because these two tanks are similar in size and materials. As reported in Appendix A of NUREG/CR-5270 in the FA method, the aleatory uncertainty β_R and the epistemic uncertainty β_U were 0.20 and 0.27, respectively. These uncertainty values are almost identical to those reported by Choun, et al (2008), where the only difference is that the aleatory uncertainty was 0.21. The composite uncertainty β_C can be calculated as 0.34.

Based on the HCLPF capacity and the uncertainties, the median fragility capacity can be estimated to be 0.923 g. Figure 2 shows the mean fragility curve and the median, 5% percentile, and 95% percentile fragility curves.

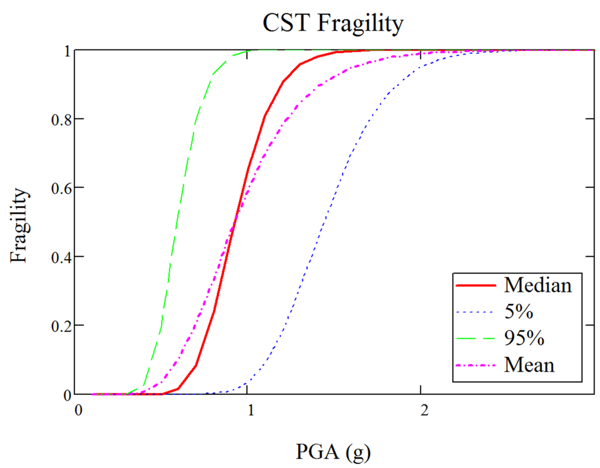

FIGURE 2 FRAGILITY CURVES OF THE CST

4. FRAGILITY ANALYSES OF DEGRADED CST

A series of time-dependent fragility analyses of the CST was performed by incorporating selected age-related material degradation models into the fragility analysis. Three separate degradation scenarios and one combined degradation scenario were considered: (A) degraded stainless tank shell, (B) degraded anchor bolts, (C) anchorage concrete cracking, and (D) a perfect correlation of the three degradation scenarios.

4.1 Fragility Analysis for (A) Degraded Tank Shell

The material degradation model for the stainless steel tank shell is the mechanochemical model for stress corrosion cracking (SCC) (Saito and Kuniya, 2001). Austenitic stainless steel (especially type 304) is widely used in light water reactors (LWRs) and in particular for the subject CST. The structural integrity of the involved components due to inter-granular stress corrosion cracking (IGSCC) is often a concern in NPPs. The derivation of this model was lengthy, highly theoretical, and cannot be easily summarized in this paper. Interested readers are recommended to refer back to the original reference. Fortunately, based on the theoretical development, a relatively simple numerical model was also developed for type 304 stainless steel in 288 °C water, using only four parameters. This model was further simplified to a constant crack rate of 0.0075 in/year (0.19 mm/year) by specifying appropriate values for the four parameters, with considerations to alleviate the effect of the high temperature that biases from the actual temperature of the CST. The effect of the SCC cracks developed based on this model was assumed to be similar to loss of material for simplicity. The smaller thickness due to loss of material is assumed to occur at local regions at the base of the tank shell, and therefore only the capacity calculation but not the frequency and the response calculation will be changed.

The direct impact of the degraded tank shell is on the compressive buckling capacity and the fluid hold down capacity, but obviously not on the bolt hold down capacity. All three major resultant capacities: the overturning moment capacity, sliding capacity, and the fluid pressure capacity are affected.

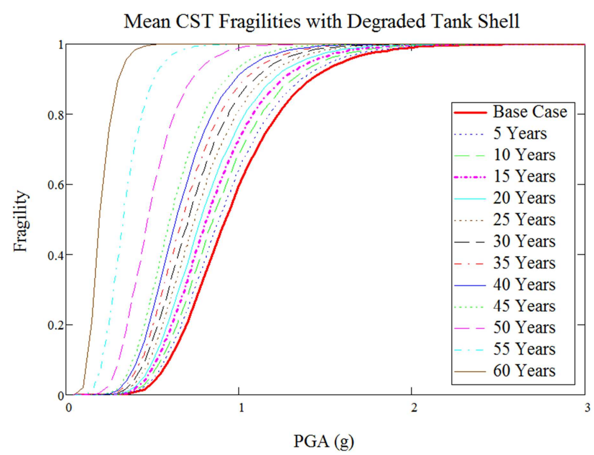

FIGURE 3 MEAN FRAGILITY CAPACITY CURVES (DEGRADED TANK SHELL)

Figure 3 shows the mean fragility capacity of the CST with degraded tank shell for a series of years, from 0 up to 60 years, after which the fragility calculation was not mathematically achievable. These mean fragility curves were calculated using unchanged uncertainties, i.e., β_R = 0.2 and β_U = 0.27, the same as utilized for the base case. In reality, since the degradation process is highly random and uncertain, both the epistemic and aleatory uncertainties should vary with time. Since the objective of this study is for demonstration purposes, the effect of the degradation on the uncertainties is not considered. In Figure 3, it is obvious that the spacing of the fragility curves suddenly increases significantly after 45 years, when the governing failure mode shifted from the sliding failure to the overturning moment failure.

It is easier to see the transition of failure mode by the relation of the HCLPF fragility and time. Figure 4 shows in solid lines the HCLPF fragility of the CST as a function of time. It also includes the corresponding overturning moment capacities, sliding capacities, and the fluid pressure capacities, in dotted, dashed, and dash-dot lines, respectively. The fragility capacity is taken as the minimum of these three capacities. It is obvious that the tank shell degradation (wall thinning) has the most significant impact on fluid pressure capacity and the least impact on sliding capacity. The fragility capacity is clearly dominated by the sliding mode until slightly after 45 years, and then by the overturning mode. Although the fluid pressure mode does not dominate the fragility capacity up to 60 years, it would be dominant shortly after 60 years had the calculation continued.

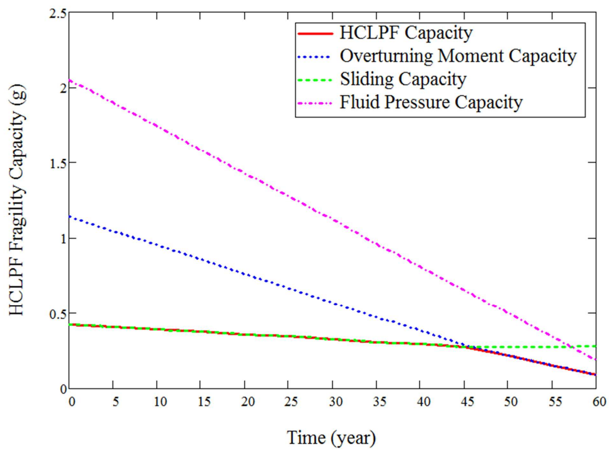

FIGURE 4 HCLPF CAPACITY OVER TIME (DEGRADED TANK SHELL)

4.2 Fragility Analysis for (B) Degraded Anchor Bolts

Unlike the stainless steel tank shell of the CST, the anchor bolts made of A36 are prone to corrosion because of the salty moisture in a location close to the ocean. The power model for steel corrosion used by Mori (2005) in a study of reliability-based service life prediction was chosen for modeling the degradation of the anchor bolts. The power model can be used for modeling of both concrete cracking/reinforcement corrosion and corrosion of carbon and low alloy steel. Since the Ulchin NPP is located on the coast, a marine environment was assumed in identifying the parameters for this power model. The resultant power model is expressed by,

$$X(t) = 70.6t^{0.79}, \tag{1}$$

where t is the elapsed time in years and $X(t)$ is the level of attack in μm. A reduction of bolt diameter was assumed uniformly for all anchor bolts, as given by,

$$D_{bolt\_degraded} = D_0 - 2X(t). \tag{2}$$

The direct impact of degraded anchor bolts is simply on the bolt hold down capacity, and consequently on the overturning moment capacity and the sliding capacity. The degradation of anchor bolts does not affect the compressive buckling capacity, the fluid hold down capacity, and the fluid pressure capacity.

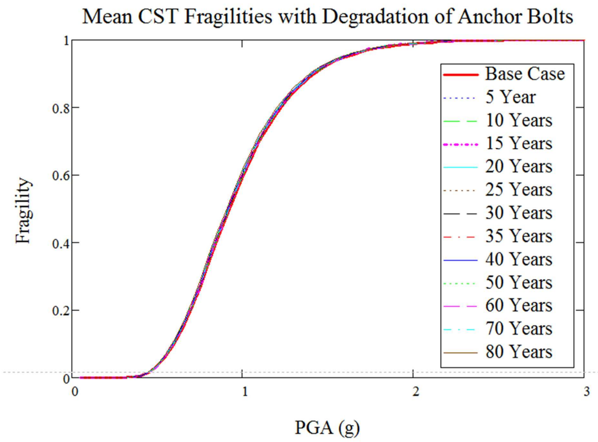

FIGURE 5 MEAN FRAGILITY CAPACITY CURVES (DEGRADED ANCHOR BOLTS)

Figure 5 shows the mean fragility capacity of the CST with corroded anchor bolts for a series of years, from 0 up to 80 years. In a practical sense, it is obvious that the mean fragility is virtually unchanged for a period of 80 years. Even with a degradation level equivalent to half of the bolt diameter (approximate 950 years using the current power model), the HCLPF SME capacity was found to be still as high as 0.34 g, compared to 0.426 g in the undegraded case. Sliding capacity dominates the HCLPF capacity for the same period. This high level of HCLPF capacity and the high reliability of the CST are believed to be attributed to the large number of bolts (78 in total).

Similarly to degradation case (A), Figure 6 shows in solid lines the HCLPF fragility of the CST as a function of time, as well as the corresponding overturning moment capacities, sliding capacities, and the fluid pressure capacities, in dotted, dashed, and dash-dot lines, respectively. From this figure, it is obvious that the anchor bolt corrosion has no or minimal impact on all three major capacities, with slightly noticeable effect on the overturning moment capacity. It is clear that the sliding capacity dominates.

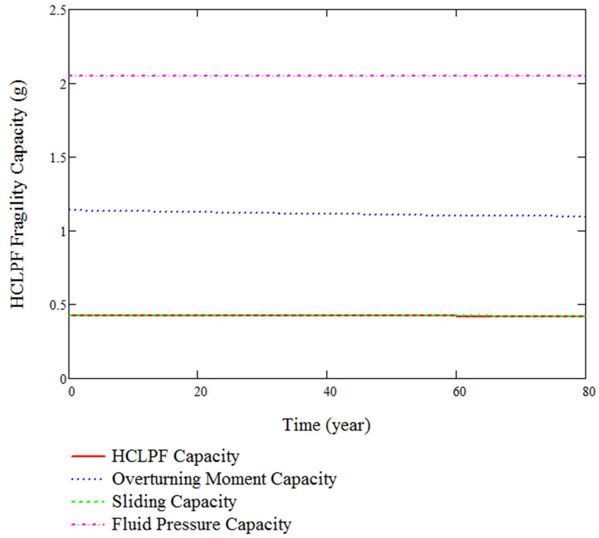

FIGURE 6 HCLPF CAPACITY OVER TIME (DEGRADED ANCHOR BOLTS)

4.3 Fragility Analysis for (C) Cracked Anchorage Concrete

Data regarding the crack width and depth of reinforced concrete were recorded in four Korean NPPs over a period of about 25 years. These data were used to develop a linear regression model:

$$W(t) = 0.0078\,t, \tag{3}$$

in which $W(t)$ is the crack width (mm) and time t is in years. It should be pointed out that the measured crack widths have significant variation and the linear regression model does not necessarily represent the true underlying relationship, as shown in Figure 7. The use of this curve in this study is for the purpose of demonstration; the applicability of this model in practice should be investigated with careful scrutiny.

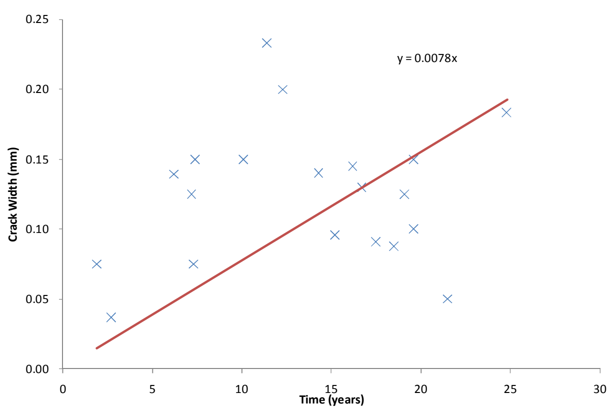

FIGURE 7 CRACK WIDTH MODEL BASED ON MEASUREMENTS IN KOREAN NPPS

The crack width model was mapped to the anchorage strength using test data for anchor strength with cracked concrete reported in NUREG/CR-5434 (Klingner, et al., 1998). The grouted anchors reported in Reference 18 had a diameter of ¾", an embedment of 4", and an effective embedment of 4", which are much smaller than those of the anchor bolts for the subject CST. Since these large differences in scale, the anchor strength test data were used as scaling factors. As a result, the tensile capacity of the anchorage for a crack width of *w* mm can be estimated based on the following linear interpolation/extrapolation:

$$T = 200 + \frac{w}{0.3}(54.4 - 200)\ \text{kips}. \tag{4}$$

The impact of the cracked concrete is directly on the bolt hold-down capacity but not the tank shell buckling capacity and the fluid pressure capacity; the overturning moment capacity and the sliding capacity are affected consequently.

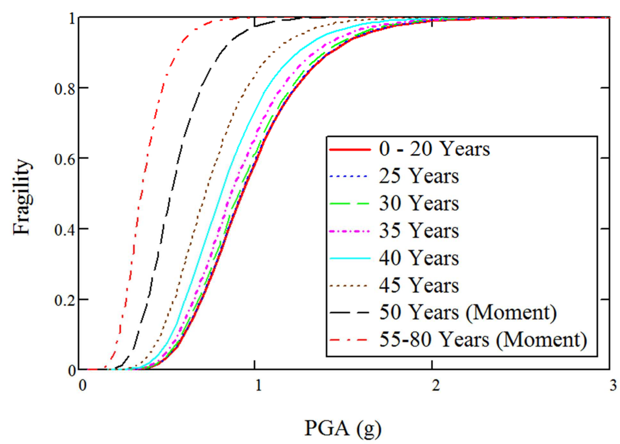

FIGURE 8 MEAN FRAGILITY CAPACITY CURVES (CONCRETE CRACKING)

Figure 8 shows the mean fragility capacity of the CST with anchorage concrete cracking for a series of years, from 0 up to 80 years. The mean fragility does not change in the first 20 years and in the last 25 years, with an increasing rate of fragility capacity deterioration for the years in the middle. The governing failure changed from the sliding mode to overturning moment mode at 50 years.

The deterioration of fragility capacities as a function of time can be easily observed in the HCLPF fragility, as shown in Figure 9. There are 4 regions in this figure: (1) during 0-20 years, with a maximum crack width of 0.156 mm, the fragility capacities were unchanged because of the large number of bolts that have no or moderate reduction in their bolt hold-down capacity; (2) between 20 to about 48 years, the fragility capacity was dominated by the sliding mode; (3) before 55 years, the fragility capacity was dominated by the overturning moment mode and the reduction in the bolt hold-down capacity affects the overturning moment capacity; and (4) after 55 years, the fragility capacity continue to be dominated by the overturning moment capacity, the bolts in tension appeared to have been pulled out, and the CST effectively becomes an unanchored tank. The overturning moment capacity starts to be affected dramatically by the bolt hold-down capacity after 20 years until the bolts reach a zero capacity around 55 years. The bolt hold-down capacity does not have as great an impact on the sliding capacity as on the overturning moment capacity, and it does not have any impact on the fluid pressure capacity as expected. It should be pointed out that these findings are based on the selected material degradation model and the assumption of no periodic inspection/maintenance.

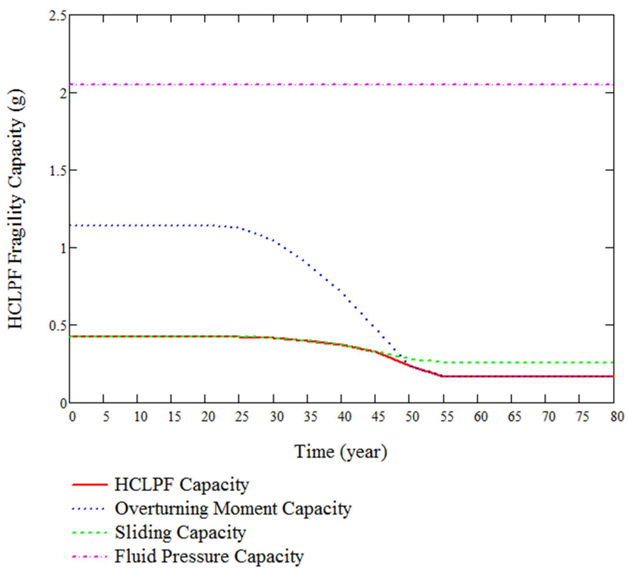

FIGURE 9 HCLPF CAPACITY OVER TIME (CONCRETE CRACKING)

It is cautioned that the above observation is based on a greatly simplified conversion from the NUREG/CR-5434 test results (Klingner, et al., 1998) to the large size anchor bolts, in which many uncertain factors were not considered, for example, how the crack depth in conjunction with the crack width affect the bolt hold-down capacity. The surface crack may not always be a good indicator of the crack depth.

4.4 Fragility Analysis for (D) Multiple Degradations

Degradation cases A, B, and C were combined together to investigate the effect of multiple degradations on the seismic fragility capacities. The three degradation cases are assumed to be perfectly correlated, i.e., the severity of each of the degradation cases is a deterministic function of the common time variable.

Figure 10 shows the median fragility curves for the CST with combined degradations up to 65 years. The fragility curves before the end of 45 years show equal and fine spacing between them, indicating a steady but slow degradation process. Between 45 years and 55 years, a sudden increase of the degradation severity is shown by the large spacing between the corresponding fragility curves. The very small spacing between 55 and 60 years suggest a very small drop in the fragility capacity, followed by a slightly increased drop in fragility capacity. The fragility capacity diminishes at 65 years, after which the fragility calculation in Mathcad could not reach a plausible solution.

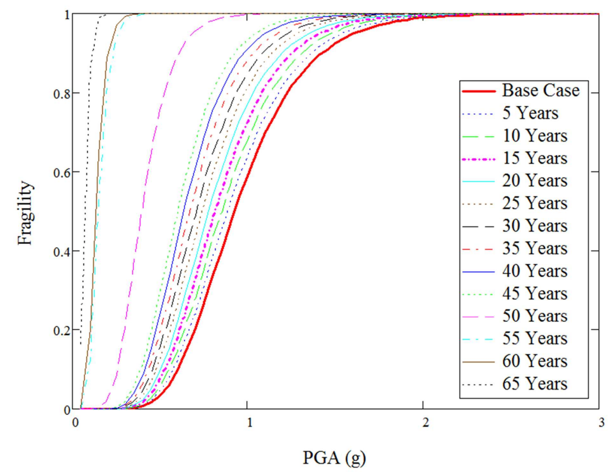

FIGURE 10 MEAN FRAGILITY CAPACITY CURVES (MULTIPLE DEGRADATIONS)

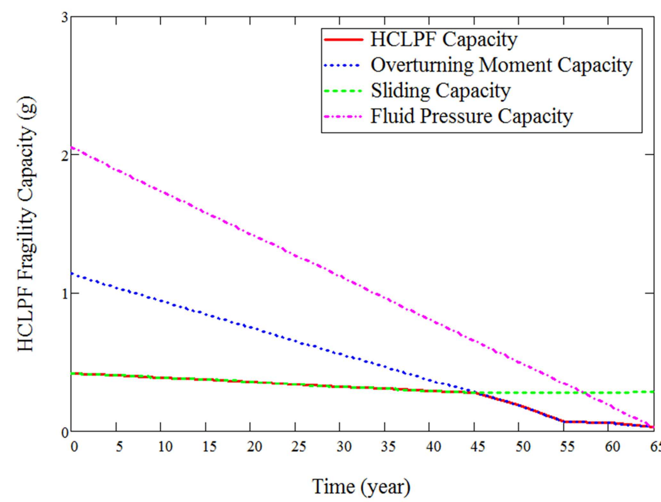

FIGURE 11 HCLPF CAPACITY OVER TIME (MULTIPLE DEGRADATIONS)

The trend of the fragility capacity change can be better characterized by the HCLPF fragility capacity, as shown in Figure 11. Before the end of 45 years, the fragility capacity is dominated by the slow deterioration of the sliding capacity. Between 45 years and 55 years, the dominating failure mode switches to the overturning moment mode and the resultant deterioration rate in the fragility becomes higher. Between 55 and 60, the fragility capacity is still dominated by the overturning moment capacity, which levels to a small constant because the CST effectively is unanchored tank as previously shown in the degradation case C. At the end of 65 years, the overturning moment capacity and the fluid pressure capacity are very close, with the latter dominating the fragility capacity. This is the only occasion among all degradation scenarios that the fluid pressure capacity dominates the fragility calculation.

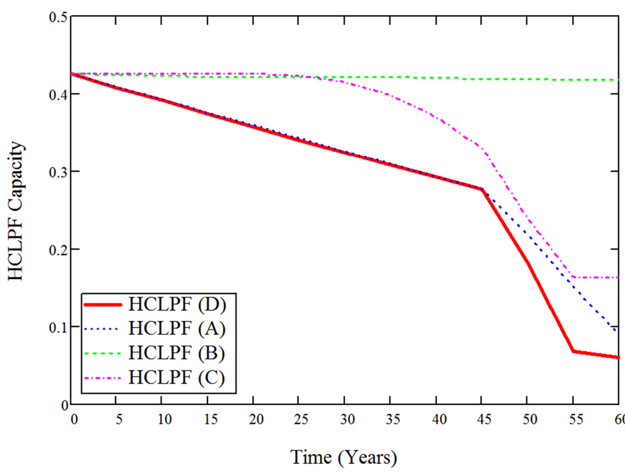

FIGURE 12 COMPARISON OF HCLPF CAPACITIES AMONG ALL DEGRADATION SCENARIOS

Figure 12 compares the HCLPF capacities among all 4 degradation cases, with the solid line for the combined degradation case, the dotted line for the degraded tank shell case, the dashed line for the degraded anchor bolt case, and the dash-dotted line for the cracked anchorage concrete case. It is interesting to note that before 45 years, the HCLPF fragility for the combined degradation case is the same as that for the degradation of the tank shell, indicating the degradation of anchor bolts and the anchorage concrete cracking have no effect on the fragility. After 45 years, it appears all three degradation scenarios contribute to the HCLPF fragility for the combined degradation case. Figure 12 also shows that the corrosion model for the anchor bolts, although appearing to be for the severest environment (marine) case, does not incur a significant amount of loss of cross section and the corresponding deterioration in fragility capacity is minimal.

5. SUMMARY

The conservative deterministic failure margin (CDFM) method was utilized for seismic fragility analysis of the undegraded case and was modified to accommodate the degraded cases. A total of five seismic fragility analysis cases have been described: (1) the undegraded case, (2) degraded stainless tank shell, (3) degraded anchor bolts, (4) anchorage concrete cracking, and (5) a perfect correlation of the three degradation scenarios.

It is found that the HCLPF capacity can deteriorate to the SRM/SECY 93-087 level (0.334 g) in about 25 years, based on only the tank shell degradation or multiple degradations. This finding was based on the mechanochemical model for stress corrosion cracking in the stainless tank shell. Degradation of anchor bolts is not a significant factor in the HCLPF capacity of the CST. Cracking in the reinforced concrete foundation may reduce the HCPLF capacity to the 0.334 g level in about 45 years. However, all these findings depend on the selected material degradation models and assumed degradation rates; these results are presented to demonstrate the process. They do not take into account the inspection/maintenance programs that are normally established in nuclear power plants.

It appeared that the large number of anchor bolts did provide a substantial level of conservatism to the CST design so that the HCLPF capacity deterioration over time can be less of a concern.

It is recognized in this study that the most critical factor for a high quality time-dependent fragility analysis is the identification of accurate and reliability material degradation models. Recorded degradation data in NPPs are not available in a consistent and coherent fashion for use in development of degradation models, but extremely important for performing fragility analysis. Long-term measurement/monitoring of the performance of safety significant SPCs remains a high priority for the future research/operation. As more recorded degradation data in NPPs are obtained in the future, the existing material degradation models and fragility analyses can be updated to improve their accuracy and thereby ensure the continued safe operation of NPPs.

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