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Fragility Functions for Seismic Performance Assessment of Safety-Related Reinforced Concrete Nuclear Structures

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1 ABSTRACT

Squat (shear-critical) reinforced concrete walls are widely used in nuclear power plants and other safety-related nuclear structures to provide resistance to extreme earthquake loadings. Performance assessment of such structures utilize fragility functions that relate the probability of exceeding one or more damage thresholds to either a ground-motion or response (demand) parameter such as peak ground acceleration, spectral acceleration at a selected period, story drift or component plastic deformation.

Fragility functions are developed for squat reinforced concrete walls with aspect ratio (height-to-length or (h_w / l_w) of 2 or less) by review and statistical evaluation of experimental data in the literature. The experimental data includes tests of three cross-section types: rectangular, barbell and flanged. Per modern practice, a demand parameter is used to construct the curves. Experimental damage data is characterized using damage states and methods of repairs. Documents that provide guidelines for repair of reinforced concrete walls, observations from experimental programs, previous research on retrofit of squat walls and expert opinion are used to identify the most appropriate damage states and their corresponding methods of repair. Damage states are characterized generally by direct indicators of damage such as initiation of cracking, maximum concrete crack width, extent of concrete crushing, sliding shear displacement, and reinforcement yielding, buckling, and fracture. Each of these damage states is linked with one of four methods of repair, namely, cosmetic repair, epoxy injection, partial wall replacement, and wall replacement.

Different families of fragility functions are required for each cross-section type but the data do not support the development of fragility surfaces to accommodate axial force, rebar ratio and aspect ratio as input variables. Story drift is used as the demand parameter. Scopes of repair are provided elsewhere.

2 INTRODUCTION

NUREG-1407 (Chen et al. 1991) provides guidance to nuclear power plant (NPP) utilities on Individual Plant Examination of External Events. NUREG-1407 identified Seismic Margin Assessment (SMA) and Seismic Probabilistic Risk Assessment (SPRA) as acceptable methodologies for the examination of earthquake risk.

SMA seeks to identify critical components and systems in a nuclear power plant (NPP) and determine the high-confidence-low-probability-of-failure capacity of each critical NPP component and plant damage state in terms of ground-motion intensity. The annual frequency of unacceptable performance, such as core melt and release of radiation, can be computed using SPRA, which involves integration of plant fragility data and seismic hazard curves over a wide range of ground-shaking intensity and requires a full consideration of uncertainty in seismic hazard, structural response and properties and capacities of NPP components (Huang et al. 2008, 2009).

NUREG/CR-2300 (USNRC 1983) provides general guidance for performing a SPRA and identifies two acceptable methods: 1) Zion, and 2) the Seismic Safety Margin (SSM). The Zion method was developed and applied first in the Oyster Creek probabilistic risk assessment and later improved and applied in 1981 to estimate seismic risk for the Zion Plant (Pickard, Lowe, and Garrick, Inc., et al. 1981). The method has been widely used for assessment of existing nuclear facilities in the United States. The SSM method (Smith et al. 1981) was developed by the United States Nuclear Regulatory Commission (USNRC) but has not been applied to NPP projects to the knowledge of the authors.

An important difference between the Zion and SSM methods is the parameter used to characterize fragility curves. In the Zion method, the component fragility curves are defined in terms of ground-motion intensity (peak ground acceleration). In the SSM method, fragility curves are defined in terms of response parameters and response (or component demand) is computed using response-history analysis, where the ground motions are scaled using peak ground acceleration. The advanced seismic performance assessment methodology of Huang et al. (2008, 2009) also uses response parameters to characterize fragility curves but the procedure used to scale earthquake ground motions is most different from the SSM method. The use of response- rather than ground-motion-based fragility curves enables the fragility curves to be independent of seismic hazard and closely related to component capacity.

This paper presents fragility curves for squat reinforced concrete walls, which are important structural components in NPPs and other safety-related nuclear structures. A squat wall is defined herein as a wall with a height-to-length or (h_w / l_w) of 2 or less. Three different wall geometries are considered: rectangular, barbell, and flanged. Damage states are characterized using qualitative descriptors such as residual concrete crack width (a critical parameter for gas leakage, see Wang and Hutchinson 2005), extent of concrete crushing, sliding shear displacement, and reinforcement yielding and buckling, but must be presented by measures (or scopes) of repair to compute estimates of facility downtime and the annual probability of unacceptable performance. Information from experiments on squat walls reported in the literature is used to generate damage data as a function of story drift, which is an efficient demand parameter for squat walls. The lognormal probability distribution is used to present the damage data. Goodness-of-fit tests are performed to evaluate the utility of the distribution. Much additional information is available in Gulec (2009) and Gulec et al. (2009).

3 SUMMARY OF EXPERIMENTAL DATA

A significant number of squat reinforced concrete walls have been tested in the past 60 years. Gulec (2009) reviewed and catalogued information on 434 squat wall tests. However, most of these experimental programs focused on the maximum strength and initial (elastic) stiffness of the walls and limited attention was paid to progression of damage with increasing lateral displacement (drift). Information that is critical to the development of damage states and fragility curves, such as residual crack width, maximum reinforcement strain and buckling, extent of concrete crushing, and photographs showing damage as a function of drift, was generally not reported. Only 111 of the 434 test specimens could be used to develop fragility functions.

Experimental data from the 111 squat walls provided the damage data presented below. The data for these 111 walls were acquired from Pilakoutas (1991), Greifenhagen et al. (2005), Lefas et al. (1990), Maier and Thürlimann (1985), Wiradinata (1985), Pilette (1987), Mohammadi-Doostdar (1994), Salonikios et al. (1999), Synge (1980), Massone (2006), Xie and Xiao (2000), Hidalgo et al. (2002), and Lopes and Elnashai (1991). Twenty-eight walls had flanges, 32 walls had barbells, and 51 walls had rectangular cross-sections. The web thickness ranged between 1.77 in. and 6 in. The aspect ratios ranged between 0.21 and 2.0. The maximum aspect ratio in the barbell and flanged wall datasets are 0.63 and 1.02, respectively. Sixty-five walls were tested with applied axial load ranging between 0.022 Af_c' and 0.273 Af_c', where A is total wall area and f_c' is concrete compressive strength. Concrete compressive strength, based on standard cylinder tests, varied between 2370 and 8463 psi. Eight walls did not have vertical web reinforcement, 6 walls did not have horizontal web reinforcement and 3 walls did not have any web reinforcement. Eleven of the 111 test specimens did not comply with the minimum reinforcement requirements ($\rho_h = \rho_v = 0.25\%$) of ACI 318-05 (ACI 2006a) and 349-06 (ACI 2006b) for *Special Structural Walls and Concrete Beams* (Chapter 21.9). The maximum observed horizontal and vertical reinforcement ratios were 2.80%. Eighty-nine of the 111 test specimens were tested using cyclic loading, 21 were tested using monotonic loading and 1 was tested using an earthquake simulator. Information on the 111 squat walls used to generate fragility functions is presented in Gulec (2009) and Gulec et al. (2009).

4 DAMAGE STATES AND METHODS OF REPAIR

Damage states (DS) define threshold levels of damage sustained by structural components under earthquake loading. The family of damage states listed in Table 1 was assembled following analysis of test data and review of the literature. Damage states are characterized generally by direct indicators of damage such as initiation of cracking, residual concrete crack width, extent of concrete crushing, sliding shear displacement,

and reinforcement yielding, buckling, and fracture. Each of these damage states is linked with one of four methods of repair in the table.

Documents that provide guidelines for repair of reinforced concrete walls (e.g., FEMA 306 [ATC 1998a] and FEMA 308 [ATC 1998b]), repair of concrete (e.g., ACI 546R-04 [ACI 2004)]), observations from experimental programs, previous research on retrofit of squat walls, and expert opinion are used to identify the most appropriate damage states and their corresponding methods of repair. The following subsections present information on each Method of Repair (MoR) and the corresponding damage states. Information on the scope of each repair (e.g., length of crack to be epoxy injected can be found in Gulec (2009) and Gulec et al. (2009).

4.1 MoR-1, Cosmetic Repair

Damage states DS1.1 through DS1.4 of Table 1 are associated with cosmetic repair. These damage states represent the initiation of cracking in concrete and propagation of these cracks under earthquake loading. For cosmetic repair, crack widths are small (less than 0.02 in. as defined in DS1.4) and structural repair to restore strength and stiffness is unnecessary. Repair of the surface finishes may be required to restore the aesthetic appearance, maintain fire resistance, and prevent water infiltration (ATC 1998b). Cosmetic repair has no impact on structural performance. Examples of damage states 1.2 and 1.3, each of which is linked to MoR-1, are presented in Figure 1 and Figure 2.

4.2 MoR-2, Epoxy Injection

Damage states DS2.1, DS2.2, DS2.3, DS2.4a, DS2.5a, DS2.4b, and DS2.5b of Table 1 are associated with epoxy-injection repair. Epoxy injection is widely used to restore the stiffness and strength of cracked concrete components. Crack width (DS2.4a, DS2.5a, DS2.4b, and DS2.5b) and reinforcement yielding (DS2.1, DS2.2, and DS2.3) are considered two indicators of the need for epoxy injection repair. Examples of damage states DS2.1, DS2.3, and DS2.4a, each of which is linked to MoR-2, are presented in Figure 3 and Figure 4. There is no consensus on the minimum residual crack width for epoxy injection and so two options are presented herein: MoR-2a and MoR-2b.

4.3 MoR-3, Partial Wall Replacement

Damage states DS3.1 through DS3.4 of Table 1 are associated with removal and replacement of damaged concrete. Localized crushing at boundary elements and in the wall web will require replacement of damaged concrete. Vertical splitting cracks in boundary elements (DS3.2) and buckling of boundary element vertical reinforcement (DS3.3) are also linked to this MoR. Photographs of test specimens and observations made by the cited authors were used to identify data for damage states DS3.1, DS3.2, and DS3.3. Damage state DS3.4 assumes that a wall with flexural crack widths greater than 0.12 in. (3 mm) cannot be repaired to its pre-earthquake condition using epoxy injection and that damaged concrete and rebar must be replaced. Examples of damage states DS3.1 and DS3.4 are presented in Figure 5 and Figure 6, respectively.

4.4 MoR-4, Wall Replacement

Damage states DS4.1 through DS4.5 of Table 1 are associated with complete replacement of the wall panel. Sliding at the interface between the wall web and the foundation (DS4.1) will require wall panel replacement, especially if the residual displacement is significant. Squat walls with low vertical reinforcement ratio and low axial load are susceptible to sliding shear. A sample cracking pattern for damage state DS4.1 is presented in Figure 7. Substantial corner-to-corner diagonal cracks (DS 4.2) which are indicators of diagonal tension failures, trigger a rapid loss of strength and stiffness. Such walls cannot be repaired using epoxy injection because the crack widths are too great and partial wall replacement is not feasible because of the orientation and length of the cracks. Squat walls with light horizontal reinforcement are susceptible to this type of damage. Figure 8 presents a cracking pattern for damage state DS4.2.

Table 1. Damage states and corresponding methods of repairs

ID	Damage States	Method of Repair (MoR)
DS1.1	Initiation of cracking	Cosmetic repair (MoR-1)
DS1.2	Initiation of flexural cracking	
DS1.3	Initiation of shear cracking	
DS1.4	Maximum measured crack widths less than 0.02 in. (0.5 mm)	
DS2.1	Initiation of yielding in horizontal web reinforcement	Epoxy injection (MoR-2)
DS2.2	Initiation of yielding in vertical web reinforcement	
DS2.3	Initiation of yielding in vertical boundary element reinforcement	
DS2.4a	Maximum measured shear crack widths larger than 0.02 in (0.5 mm) but less than 0.12 in. (3 mm)	
DS2.5a	Maximum measured flexural crack widths larger than 0.02 in (0.5 mm) but less than 0.12 in. (3 mm)	
DS2.4b	Maximum measured shear crack widths larger than 0.04 in (1.0 mm) but less than 0.12 in. (3 mm)	
DS2.5b	Maximum measured flexural crack widths larger than 0.04 in (1.0 mm) but less than 0.12 in. (3 mm)	
DS3.1	Concrete crushing at the compression toes / initiation of crushing in the wall web	Partial wall replacement (MoR-3)
DS3.2	Vertical cracking in the toe regions of the web	
DS3.3	Buckling of boundary element vertical reinforcement	
DS3.4	Flexural crack widths exceeding 0.12 in. (3 mm)	
DS4.1	Initiation of sliding	Wall replacement (MoR-4)
DS4.2	Wide diagonal cracks	
DS4.3	Widespread crushing of concrete	
DS4.4	Reinforcement fracture	
DS4.5	Shear crack widths exceeding 0.12 in (3 mm)	

Walls with heavy reinforcement and/or high axial loads usually fail in diagonal compression (crushing of compression struts) as shown in Figure 9. The damage is so widespread that partial wall replacement is not feasible and the entire wall panel must be replaced (DS4.3). Reinforcement fracture (DS4.4) is rarely seen in tests of squat walls. Reinforcement fractured in only 3 of the 111 walls. Damage state DS4.4 is linked to wall replacement because reinforcement fracture followed other significant damage in all three cases. Widespread damage at the interface between the wall web and foundation is evident in the figure. Damage State DS4.5 assumes that wall panels with maximum shear crack widths larger than 0.12 in (3 mm) must be replaced. FEMA 306 notes that when the width of shear cracks in a wall exceed 1/8 in. (3.2 mm), the damage is considered *heavy* and restoration of the pre-earthquake strength and stiffness requires replacement of the wall. This damage state is similar to damage state DS4.2 in the sense that it is also closely related to diagonal tension failure observed in squat walls. The available images of damage together with the author's descriptions of damage were used wherever possible to identify the drift associated with wall replacement.

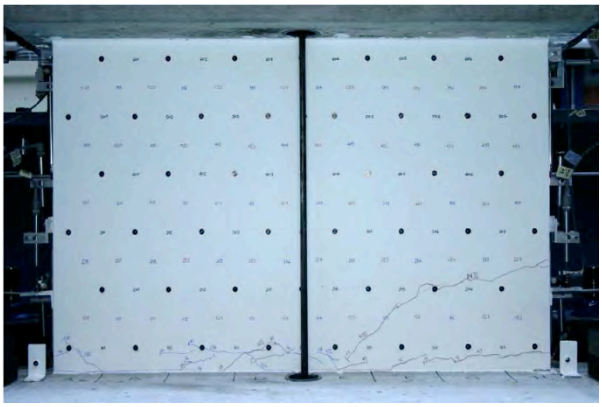


Figure 1. Initial flexural cracks on wall M4 tested by Greifenhagen et al. (2005)

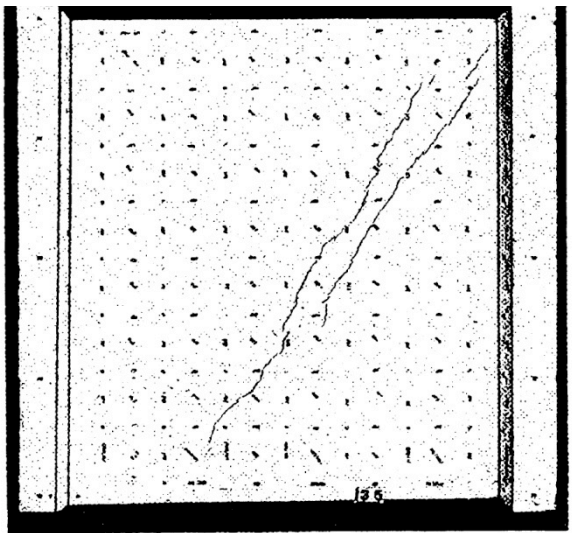


Figure 2. Initial shear cracks on wall S2 tested by Maier and Thürlimann (1985)

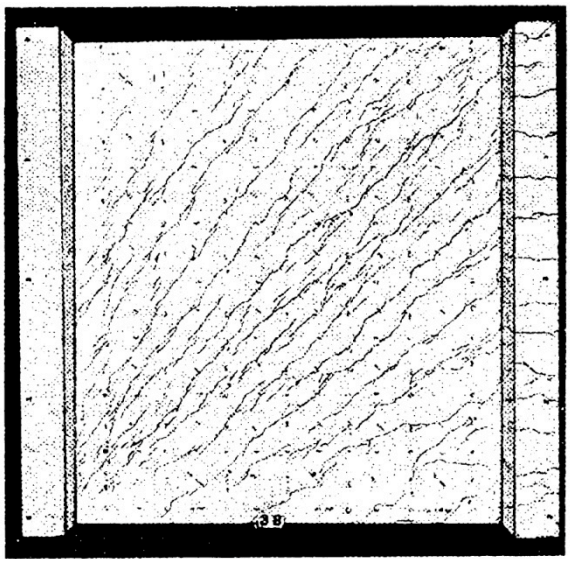


Figure 3. Cracking pattern for wall S3 tested by Maier and Thürlimann (1985) at first yielding of horizontal web reinforcement

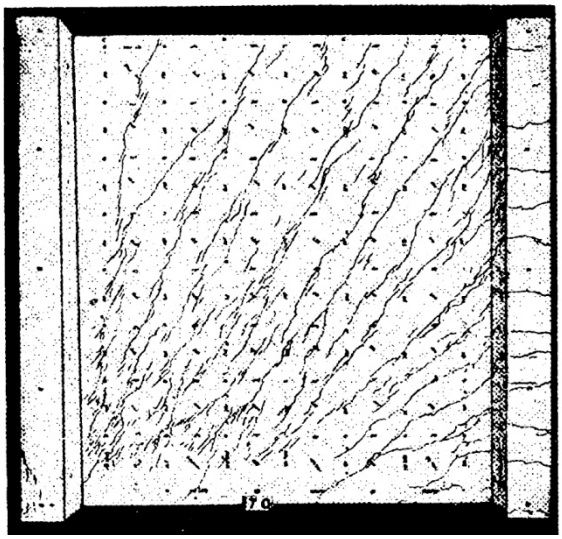


Figure 4. Cracking pattern for wall S2 tested by Maier and Thürlimann (1985) at first yielding of flange vertical reinforcement

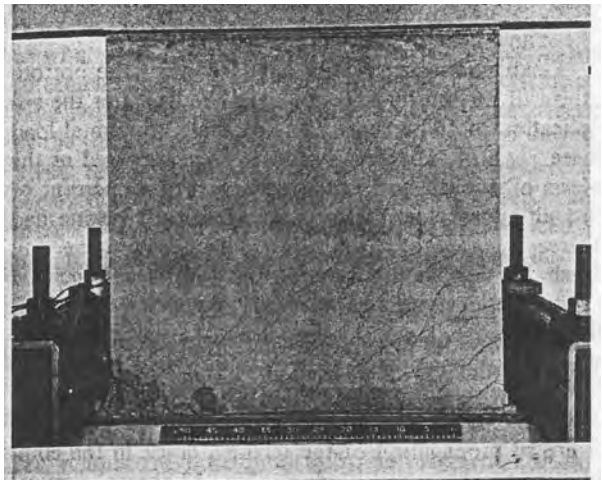


Figure 5. Cracking pattern for wall SW11 tested by Lefas et al. (1990) at compression zone failure

Figure 6. Cracking pattern for wall M2 tested by Greifenhagen et al. (2005) at a maximum flexural crack width of 3.0 mm

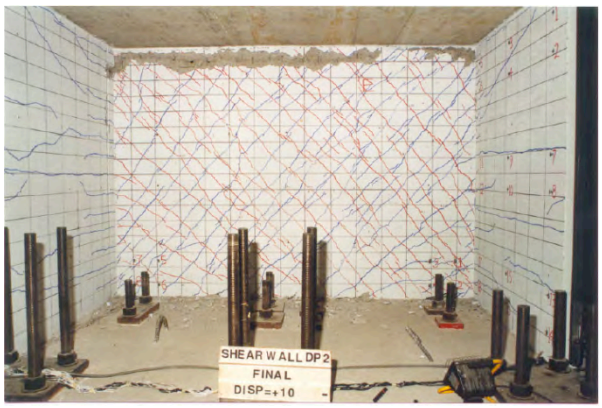


Figure 7. Cracking pattern for wall DP2 tested by Palermo and Vecchio (2002) at a sliding failure between the wall web and top slab

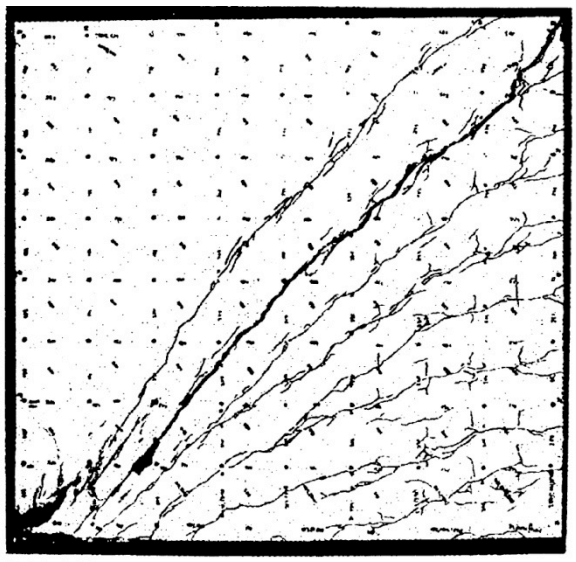


Figure 8. Cracking pattern for wall S9 tested by Maier and Thürlimann (1985) at diagonal tension failure

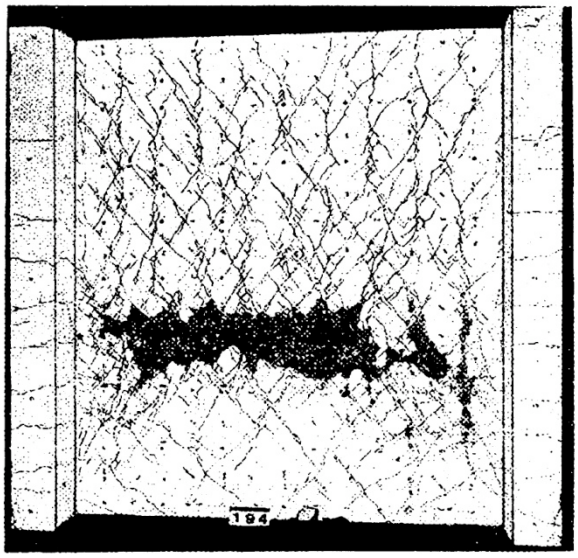


Figure 9. Cracking pattern for wall S7 tested by Maier and Thürlimann (1985) at crushing of diagonal compression struts

5 FRAGILITY FUNCTIONS

5.1 Introduction

Herein, fragility functions are developed to characterize the probability that a specific MoR will be required as a function of racking (shear) drift using the lognormal distribution. The lognormal distribution is a one-sided probability distribution of a random variable whose logarithm is normally distributed. This distribution is widely used for fragility studies because the demand parameter (drift or acceleration) must be positive and its relationship with the normal or Gaussian distribution.

The method of maximum likelihood is used to fit the data. The method of maximum likelihood is the most widely used rule for finding point estimations of distribution parameters for sample data and is used below to estimate the parameters for the lognormal distribution. This method makes use of the sample likelihood function. To quantify the utility of the distributions, Lilliefors goodness-of-fit tests are performed on each fit. Data for all damage states is used to create the fragility functions, resulting in multiple data points per specimen for a given MoR (Gulec et al. 2009).

5.2 Fragility Curve Parameters

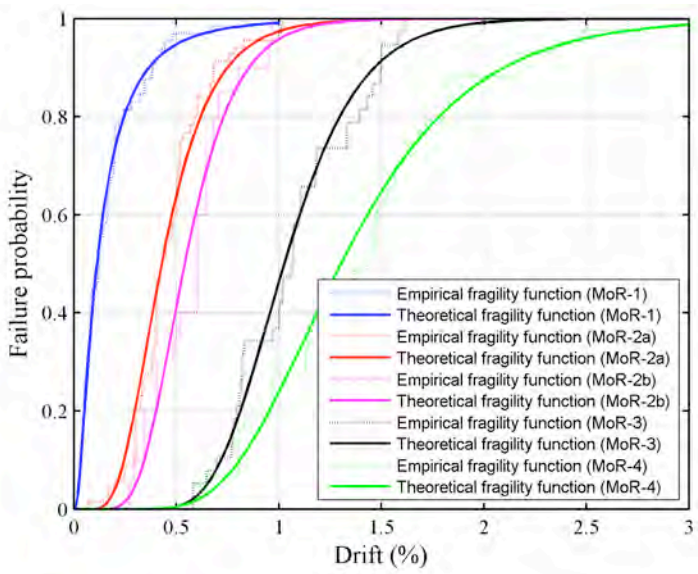
The distribution parameters estimated using the *method of maximum likelihood* and the corresponding results for the Lilliefors goodness-of-fit test are presented in Table 2 for rectangular, barbell and flanged walls. In Table 2, θ is the median and β is the logarithmic standard deviation for the lognormal distribution, and D is the test statistic for the Lilliefors goodness of fit test. The D statistic for the Lilliefors test is a measure of the deviation between the reported data and the hypothesized CDF. The null hypothesis, H_0, for the test is that the population X comes from the hypothesized probability distribution (A = Accept, R = Reject). The null hypothesis is accepted if D is less than or equal to D_{crit} at a specified significance level. The goodness-of-fit tests of Table 2 are conducted at the 5% significance level. The *p* value of the test is also included in the table.

Table 2. Lognormal distribution parameters and Lilliefors goodness-of-fit test results

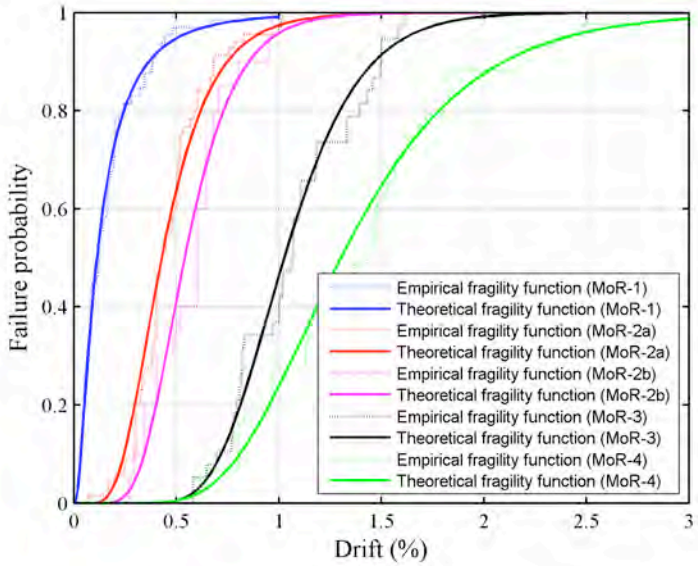
Wall Geometry	**MoR**	θ	β	**Lilliefors Test Results**			
				D	H_0	D_{crit}	p
Rectangular	**1**	0.11	0.92	0.067	A	0.110	0.500
	2a	0.43	0.43	0.113	R	0.107	0.028
	2b	0.54	0.36	0.210	R	0.192	0.020
	3	1.03	0.28	0.116	A	0.142	0.214
	4	1.30	0.37	0.106	A	0.134	0.247
Barbell	**1**	0.04	0.47	0.096	A	0.132	0.381
	2a	0.38	0.50	0.090	A	0.146	0.500
	3	0.32	0.45	0.153	A	0.219	0.438
	4	0.87	0.17	0.326	R	0.154	0.001
Flanged	**1**	0.07	1.03	0.126	A	0.128	0.058
	2a	0.48	0.68	0.080	A	0.121	0.500
	2b	0.71	0.34	0.230	A	0.304	0.322
	3	0.75	0.32	0.107	A	0.173	0.500
	4	1.32	0.44	0.084	A	0.197	0.500

The lognormal distribution fails the Lilliefors goodness-of-fit test for MoR-2a and MoR-2b for the rectangular walls and for MoR-4 for the barbell walls. No outlier analysis was undertaken before processing the data.

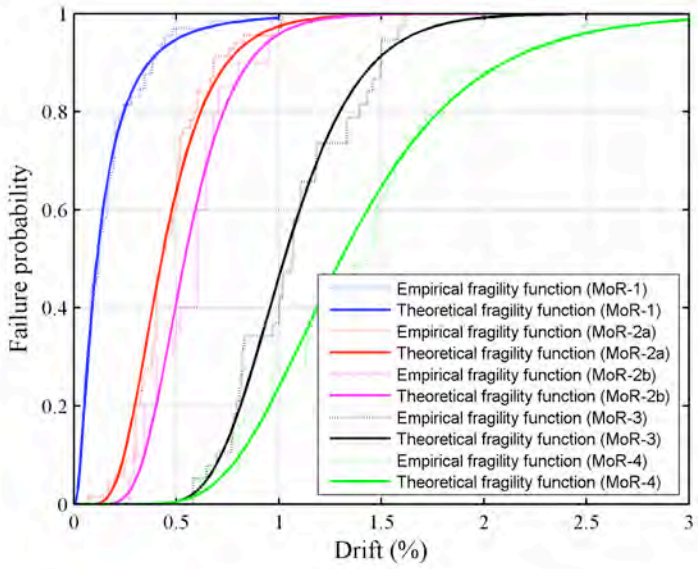
The functions developed using lognormal distribution for each wall geometry and method of repair can be compared using the medians (θ) and logarithmic standard deviations (β) presented in the gray shaded columns of Table 2. The standard deviations for MoR-1 are high. The standard deviations for other methods of repair are reasonable. ATC-58 (ATC 2009) qualifies the fragility functions that pass the Lilliefors goodness-of-fit test and yield logarithmic standard deviations (β) of less than 0.6 as *high quality*. One interesting observation is that for barbell walls, MoR-3 precedes MoR-2a, that is, the median drift associated for MoR-3 is less than that of MoR-2a. This anomaly is attributed to the characteristics of the barbell wall data used herein, namely, low-aspect ratio and heavy web reinforcement in most cases. Figure 10 presents the empirical and theoretical fragility functions for rectangular, barbell and flanged walls. MoR-3 for barbell walls is also included in Figure 10b for completeness.



a. Rectangular walls



b. Barbell walls



c. Flanged walls

Figure 10. Fragility functions for squat reinforced concrete walls

6 SUMMARY AND CONCLUSIONS

Fragility curves, damage states, and method of repair have been developed for squat walls with rectangular, barbell, and flanged cross-sections. Four methods of repair, namely, cosmetic repair, epoxy injection, partial wall replacement, and wall replacement, are appropriate for performance-based evaluation of squat reinforced concrete walls. Each method of repair is associated with a damage state and experimental data are used to develop fragility functions for each method of repair using lognormal distribution. The scope of the repair effort for each method of repair is described in Gulec (2009) and Gulec et al. (2009).

The fragility functions can be used with the Seismic Margin Method and the methodology of Huang et al. to assess the seismic performance of nuclear power plants and other safety-related nuclear structures.

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