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Fragility Functions for Concrete Moment Frames

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ABSTRACT

The ATC-58 project seeks to develop the next generation of guidelines for performance-based seismic design and evaluation. This includes the development of seismic fragility functions for commonly employed building systems. These fragility functions define the likelihood that the system will exhibit a specific level of damage and thus require a specific type of repair when subjected to earthquake demand as defined by maximum story drift. The research presented here uses data from laboratory tests to develop fragility functions for one type of building system: reinforced concrete frames.

Introduction

Performance-based seismic design and evaluation requires fragility functions to enable engineers to predict the damage state of a structure following an earthquake as well as the economic impact of repairing the structure. Here fragility functions were developed for different categories of reinforced concrete moment frames, where categories were determined using both the ACI Code (ACI Com. 318 2008) and Chapter 6 of Supplement 1 of the ASCE/SEI Standard 41-06 (2007). The fragility functions were developed using data from laboratory tests of beam-column frame sub-assemblages and from laboratory tests of rectangular frame members (columns) in combination with a numerical model of a frame sub-assemblage. Fragility functions define the likelihood that a story of the frame will exhibit a particular damage state, given maximum story drift under earthquake loading. Damage states are defined by the extent and severity of concrete cracking and crushing, yielding and buckling of reinforcing steel, and lateral load resistance. Each damage state is associated with a specific set of repair activities that would be required to restore the frame to its pre-earthquake (essentially undamaged) state.

Experimental Test Specimens

Data from two types of experimental tests were used to develop fragility functions: frame sub-assemblage tests and cantilever column tests. Frame sub-assemblage tests are the preferred tests as the specimens are complete frame sub-assemblages and are subjected to load distributions that are representative of those that could be expected to develop in a frame subjected to earthquake loading. A review of the literature produced 106 frame sub-assemblage

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test specimens from 24 test programs for which damage data were available for use in developing fragility functions. Specimen details are provided in Li and Lowes (2009). The response of these specimens was typically controlled by flexural yielding of beams or columns, joint failure, or a combination of these; thus, these specimens did not exhibit all of the response mechanisms (such as beam or column shear failure).

To extend the data set to include all possible failure mechanisms, data from column tests presented in the PEER Structural Performance Database (Berry et al. 2004) were included in the study. To use the cantilever column data in this study, damage progression in rectangular cantilever column test specimens with low axial loads (less than $0.1A_gf_c$) was assumed to characterize damage progression in the beams of a frame subassemblage while damage progression in specimens with moderate to high axial loads (greater than $0.1A_gf_c$) was considered to characterize damage progression in the columns of a frame subassemblage. Measured load-displacement response of the cantilever column was assumed equal to beam (or column) response in the frame sub-assemblage, and a model of the frame sub-assemblage was used to determine frame story drift from the cantilever column displacement. Since frame columns (or beams) and joints were assumed elastic in the model and since at larger drift levels some inelastic action might be expected in these components, column data were used only for the low to moderate story drifts levels. A review of the PEER Structural Performance Database resulted in 35 test specimens from 19 test programs for which damage data were available for use in developing fragility functions. Specimen details are provided in Li and Lowes (2009).

Frame Sub-assemblage Test Data

Specimens were sub-assemblages from two-dimensional building frames, comprising a segment of a continuous beam extending from mid-span of one frame bay to mid-span of the next, a segment of a continuous column extending from mid-height of one story to mid-height of the next, and the beam-column joint at the intersection of these. Specimens with slabs, eccentric joints, and beams extending in the out-of-plane direction were not included in the study. Typically, test specimens were subjected to lateral loading applied as a shear load at the top of the column and reacted by shear loads at the base of the column and beam ends. If, under earthquake loading, beams and columns develop a point of contra-flexure at mid-span, then this laboratory load distribution is representative of earthquake loading in a real frame. Simulated earthquake load was applied quasi-statically under displacement control, and typically specimens were subjected to three cycles each at increasing maximum drift demands. In some cases, specimens were also subjected to a constant column axial load. Test data were used to develop fragility functions for six categories of frames:

1. ***ACI Special Moment Frames (SMF).*** A test specimen was assumed to be representative of a SMF if 1) maximum strength was determined by beam yielding, 2) columns did not yield, and 3) the specimen met ACI Code Chapter 21 provisions for beams and joints and the relative flexural strength of columns. Since specimens did not exhibit column yielding, Code requirements for column detailing did not affect performance and were ignored.
2. ***ASCE Category 1 Frames (ASCE1).*** Using the component categories defined in Chapter 6 of ASCE 41-06, the highest level of performance for an existing concrete frame could be expected from frames that comprise a) "beams controlled by flexure", b) "Condition i columns" that are expected to response in flexure, and c) interior joints with compliant

transverse reinforcement (spaced at less than half of the column depth) and a shear demand-capacity ratio less than 1.2.

3. ***ACI Intermediate Moment Frames (IMF).*** Specimens were classified as IMF if they met the ACI Code Chapter 21 requirements for IMF but not those for SMF.

4. ***ASCE Category 2 Frames (ASCE2).*** Using the component categories defined in Chapter 6 of ASCE 41-06, a moderate level of performance could be expected from an existing concrete frame that comprises a) "beams controlled by flexure", b) "Condition i columns" that are expected to respond in flexure, and c) interior joints that do not have compliant transverse reinforcement and a shear demand-capacity ratio less than 1.2.

5. ***ACI Ordinary Moment Frames Controlled by Beam Yielding or Joint Failure (OMF-BYJF).*** Specimens were classified as OMF if they met the ACI Code Chapter 21 provisions for OMF but not those for IMF or SMF. The response of specimens from the data set meeting these requirements was controlled by beam yielding or joint failure prior to beam yielding.

6. ***ACI Non-Compliant Frames (NCF / ASCE3).*** Specimens were classified as non-compliant if they did not meet the requirements of Chapters 1-20 of the ACI Code. This included frames sub-assemblages in which column longitudinal steel was spliced above the joint, beam longitudinal steel was not continuous through the joint, or the Code-specified minimum volume of transverse reinforcement was not provided. The seismic response of frames included in this category is representative of existing frames that include, using the ASCE 41-06 descriptors, a) "Beams controlled by inadequate development or splicing along the span", b) "Beams controlled by inadequate embedment into beam-column joint", or c) "Column controlled by inadequate development or splicing along the clear height.

Table 1: Design parameters for frame sub-assemblages. With the exception of shear demand-capacity ratio, column axial load ratio, and the ratio of column to beam flexural strength, the ratio of the provided to ACI Code required quantity for SMF is listed.

Category		Beam-Column Joint Design					Beam Design		Column Design				
		Min. dim. (21.5.1.4)	ρ_t (21.4.4.1)	s_t (21.4.4.2)	$V_u/\phi V_n$ (21.5.3.1)	$V_u/\sqrt{f'_c}$ (21.5.3.1)	s_t (21.3.3)	$V_u/\phi V_n$ (21.3.4)	Axial load ratio	$\Sigma M_c/\Sigma M_b$ (21.4.2.2)	ρ_t (21.4.4.1)	s_t (21.4.4.2)	$V_u/\phi V_n$ (21.4.5)
SMF	Min.	1.02	0.98	0.44	0.49	6.29	0.36	0.25	0.00	1.16	0.71	0.53	0.12
	Ave.	1.46	2.10	0.66	0.70	8.86	0.67	0.56	0.13	1.75	1.63	0.80	0.23
	Max.	1.67	2.76	1.05	1.03	13.12	1.00	0.81	0.44	3.83	2.60	1.20	0.56
ASCE1	Min.	0.50	0.12	0.34	0.27	3.49	0.53	0.33	0.00	1.17	0.35	0.50	0.13
	Ave.	1.16	0.64	0.86	0.76	9.74	0.85	0.86	0.11	2.16	0.85	0.83	0.50
	Max.	2.14	2.68	1.69	1.28	16.28	1.26	1.39	0.44	5.85	2.33	1.20	1.23
IMF	Min.	0.33	0.12	0.34	0.20	2.59	0.53	0.33	0.00	0.89	0.35	0.50	0.09
	Ave.	0.99	0.64	0.88	1.09	13.94	0.78	0.91	0.14	1.92	1.09	0.81	0.48
	Max.	2.14	2.83	1.84	2.93	37.39	1.00	1.43	0.48	5.85	3.70	1.20	1.23
ASCE2	Min.	0.61	0.22	0.50	0.20	2.59	0.62	0.62	0.06	1.39	0.52	0.50	0.36
	Ave.	0.86	0.42	0.59	0.97	12.32	0.76	1.05	0.13	1.92	0.77	0.69	0.44
	Max.	1.19	0.56	0.95	1.76	22.41	0.92	1.43	0.20	2.28	1.52	1.08	0.49
OMF-BYJF	Min.	0.42	0.00	0.00	0.47	5.99	0.62	0.22	0.06	0.75	0.24	0.50	0.18
	Ave.	0.86	0.38	0.64	1.55	19.80	1.06	1.51	0.14	1.67	0.99	1.05	0.39
	Max.	1.21	2.59	1.26	3.68	46.87	1.45	3.86	0.23	3.41	3.12	2.02	0.80
NCF / ASCE3	Min.	0.48	0.00	0.00	0.38	4.78	0.78	0.40	0.10	0.58	0.15	0.89	0.13
	Ave.	0.89	0.17	0.39	1.42	18.07	1.22	2.61	0.24	1.41	0.41	2.71	0.37
	Max.	1.44	0.88	2.00	3.46	44.16	1.60	5.97	0.46	3.44	2.13	4.00	1.09

Table 1 provides statistics for design parameters that could be expected to significantly affect seismic performance for the test specimens used in this study. In Table 1, ρ_t is the transverse steel ratio, s_t is the spacing for transverse steel, M_c is the column nominal flexural strength, M_b is the beam nominal flexural strength, V_u is the component shear demand, V_n is the component shear strength and ϕ is the strength reduction factor, which is different for different components. Quantities are defined using ACI Code definitions.

UW-PEER Column Test Data

To extend the frame subassemblage data set and include data for all possible response mechanisms, the frame sub-assemblage data set was extended by adding data for square cantilever column tests from the PEER Structural Performance Database. The following explains the process used to estimate story drift from measured cantilever column displacement for the case of a column with low axial load (less than $0.1A_gf_c$), which was assumed to represent a beam in a frame. A similar process was used for column specimens with moderate to high axial loads (greater than $0.1A_gf_c$), which were assumed to represent columns in a frame.

The PEER Structural Performance database provides shear load versus lateral deflection data for a cantilever column. The model shown in Fig. 1 was used to estimate story drift from measured column drift using the following assumptions:

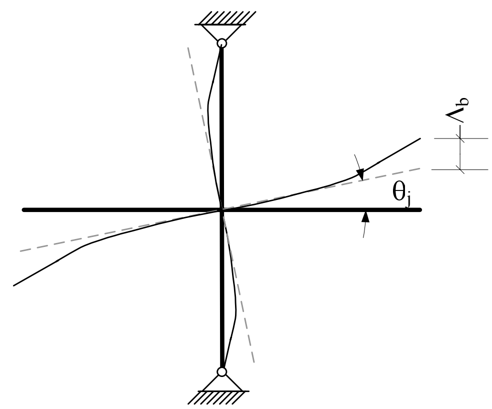

Figure 1: Frame deformation due to elastic bending of the column and deformation of the beam.

1. The measured load-displacement response of the column is equal to the response of each beam segments in the frame sub-assemblage.
2. Story drift is equal to the computed elastic deformation of the column and joint plus the measured deformation attributed to the beam.
3. The ratio of column height to beam length is 0.6. This is the average ratio for the frame sub-assemblages in the data set.
4. Column and beam out-of-plane widths are the same.
5. Column depth is 20 times the diameter of beam longitudinal reinforcement, such that ACI Code requirements for joints in SMF are satisfied.
6. Assuming the ratio of column to beam flexural strength exceeds 1.2 and following the recommendations of ASCE/SEI 41 (2007), the column is assumed rigid within the joint and the beam is assumed flexible.[3]
7. The flexural stiffness of the column in the frame sub-assemblage model is computed from the measured flexural stiffness of the cantilever column at yield.

Column test data were used to develop fragility functions for six categories of frames:

[3] For columns from the PEER database with moderate to high axial loads, which are assumed to represent columns in a frame, beams are assumed rigid in the joint and columns are assumed flexible within the joint.

1. ***ACI Special Moment Frames (SMF).*** Specimens were columns with low axial loads (less than $0.1A_gf_c$) meeting the ACI Code requirements for beams in SMF.
2. ***ACI Intermediate Moment Frames (IMF).*** Specimens were columns with low axial loads (less than $0.1A_gf_c$) meeting the ACI Code requirements for beams in IMF but not SMF.
3. ***ACI Ordinary Moment Frames Controlled by Flexure-Shear or Shear Response of Beams (OMF-BYS / ASCE4).*** Frames meeting the ACI Code requirements for OMF could be expected to exhibit a number of different response mechanisms under lateral loading including flexure-shear or shear response of beams or columns. However, no frame sub-assemblage test specimens were found in the literature for which response was controlled by shear action in beams or columns. Thus, this data set was supplemented with data from the PEER Structural Performance Database. Cantilever columns with low axial loads, meeting the ACI Code requirements for beams in OMF and exhibiting flexure-shear or shear response were included in this category. The seismic response of frames included in this category is representative of an existing frame that includes, using the ASCE 41-06 descriptors, a) "beams controlled by shear (condition ii in Table 6-7) or "beam controlled by flexure" with non-compliant transverse reinforcement and high shear demand" (condition i in Table 6-7), b) "Condition i columns (Table 6-8)" that have shear strength significantly in excess of plastic shear demand, transverse reinforcement that meets modern detailing requirement and could be expected to response in flexure, and c) interior joints with compliant transverse reinforcement (spaced at less than half of the column depth) and a shear demand-capacity ratio less than 1.2. Thus, specimens in this category are identified also as ASCE4.
4. ***ACI Ordinary Moment Frames Controlled by Flexure-Shear or Shear Response of Columns with Moderate Axial Loads (OMF-CYSM / ASCE5).*** The frame subassemblage data set was supplemented with data from the PEER Structural Performance Database for columns with moderate axial load ratios (axial load ratios between $0.1A_gf_c$ and $0.6A_gf_c$) and exhibiting flexure-shear or shear response. This category was established because evaluation of the damage data indicated that column axial load was the most critical design parameter in determining damage progression. The seismic response of frames included in this category is expected to be representative of an existing frame that includes, using the ASCE 41-07 Supplement 1 descriptors, a) "beams controlled by flexure (condition i in Table 6-7) b) "Condition i or ii columns (Table 6-8)" with axial loads less than $0.6A_gf_c$, and c) interior joints with compliant transverse reinforcement (spaced at less than half of the column depth) and a shear demand-capacity ratio less than 1.2. Thus, specimens in this category are identified also as ASCE5.
5. ***ACI Ordinary Moment Frames Controlled by Flexure-Shear or Shear Response of Columns with High Axial Loads (OMF-CYSH / ASCE6).*** The frame subassemblage data set was supplemented with data from the PEER Structural Performance Database for columns with high axial load ratios (axial load ratios greater than $0.6A_gf_c$) and exhibiting flexure-shear or shear response. The seismic response of frames included in this category is representative of an existing frame that includes, using the ASCE 41-07 Supplement 1 descriptors, a) "beams controlled by flexure (condition i in Table 6-7) b) "Condition i or ii columns (Table 6-8)" with axial loads greater than $0.6A_gf_c$, and c) interior joints with compliant transverse reinforcement (spaced at less than half of the column depth) and a shear demand-capacity ratio less than 1.2. Thus, specimens in this category are identified also as ASCE6.

6. ***ACI Non-compliant Frames (NCF / ASCE3).*** The frame subassemblage data set was supplemented with data from the PEER Column Database for specimens that did not meet the requirements of Chapters 1-20 of the ACI Code and for which longitudinal steel was spliced at the point of maximum moment demand. These specimens had axial load ratios less than 0.1Agfc and, thus, were considered to represent damage progression for beams in a frame. The seismic response of frames included in this category is representative of existing frames that include, using the ASCE 41-07 Supplement 1 descriptors, a) "Beams controlled by inadequate development or splicing along the span" (category iii beams in Table 6-7). Thus, specimens in this category are identified also as ASCE 3.

Table 2 provides statistics for design parameters that could be expected to significantly affect the performance of the column test specimens. In Table 2, axial load ratio is defined as the applied axial load divided by $A_g f_c$, s_t is the provided spacing for transverse steel divided by that required by the ACI Code for SMF), V_u is the component shear demand, V_n is the component shear strength, and ϕ is the strength reduction factor for shear.

Table 2: Design parameters for column test specimens.

Category		Column Design				Category		Column Design			
		Axial load ratio	$\rho_{l\,(\%)}$	s_t (21.4.4.1)	$V_u/\phi V_n$ (21.4.5)			Axial load ratio	$\rho_{l\,(\%)}$	s_t (21.4.4.1)	$V_u/\phi V_n$ (21.4.5)
ASCE5	Min.	0.10	0.016	1.10	0.50	SMF	Min.	0.0	0.013	0.57	0.22
	Ave.	0.16	0.025	1.92	1.85		Ave.	0.1	0.021	0.82	0.65
	Max.	0.45	0.031	3.23	5.16		Max.	0.1	0.036	0.96	0.96
ASCE6	Min.	0.61	0.018	1.12	0.29	IMF	Min.	0.0	0.016	0.76	1.84
	Ave.	0.75	0.022	1.94	0.94		Ave.	0.0	0.016	0.76	1.90
	Max.	0.90	0.025	4.78	2.93		Max.	0.0	0.016	0.76	1.94
ASCE3	Min.	0.07	0.019	4.50	1.41	ASCE4	Min.	0.0	0.014	0.88	0.32
	Ave.	0.08	0.025	4.52	1.71		Ave.	0.1	0.021	1.69	1.24
	Max.	0.09	0.030	4.54	2.01		Max.	0.3	0.032	4.54	4.97

Damage States

Table 3 lists the six repair-specific damage states (DS) used in the current study. These were developed using recommendations for the repair of damaged concrete frames (ATC 1998, ACI 546R 1996), previous investigation of the repair of earthquake damaged frame components (e.g, Jara et al. 1989, Karayannis 1998, Filiatrault 1996, Tasai 1992), and previously proposed fragility functions for concrete components (Pagni and Lowes 2006, Brown and Lowes 2007).

Ultimately, it was decided to develop fragility functions for DS 1-3. DS C was considered to be too limited to require structural repair and, repair activities following the Northridge and Loma Prieta earthquake suggested that structures exhibiting DS 0 would not be immediately repaired. For DS 4, the additional cost associated with replacing reinforcing steel was considered to be insignificant in comparison to the cost of replacing concrete. Thus, fragilities for DS 4 were not considered necessary to assess the economic impact of severe damage.

Table 3: Damage states and repair activities

Damage State	Damage Characteristic	Repair Activity
C	Damage to finishes: i) initial hairline cracking, ii) crack widths less than 0.02 in. (0.5mm).	Cosmetic Repair
0	Concrete Cracking: i) joint transverse or beam or column longitudinal reinforcement yields, ii) maximum crack widths exceeds 0.02 in. (0.5 mm).	Epoxy Inject Concrete
1	Concrete Cracking: i) maximum crack widths exceed 0.05 in. (1.3 mm).	Epoxy Inject Concrete
2	Concrete Spalling: i) initiation of beam bar slippage in the joint, ii) spalling of at least 10% of the joint surface concrete, iii) initial spalling of beam or column cover concrete.	Patch Concrete
3	Concrete Crushing: i) lateral strength begins to deteriorate, ii) spalling of more than 30% of the joint surface area, iii) concrete spalling in beams or columns extends over 10% of member depth, iv) concrete spalling exposes longitudinal reinforcement.	Replace Concrete
4	Steel yielding, buckling and fracture: i) failure due to buckling of beam or column longitudinal steel or loss of beam steel anchorage within the joint.	Replace Steel

Table 4 lists means, medians and coefficients of variation for the onset of DS 1-3 for frame subassembalge and cantilever column specimens in each of the previously defined frame categories. Data are provided for DS 0, despite the fact that no fragility functions were developed for this damage state, as these data were considered helpful in assessing appropriate fragility function parameters for DS 1 for which the data set was extremely small. While researchers routinely identify the drift at which measurable cracking initiates (DS 0), they rarely identify the drift at which moderate cracking initiates (DS 1). For damage prediction using frame sub-assemblage versus cantilever column data:

1. For DS 0-2, associated with median drifts less than approximately 3%, data from frame sub-assemblage tests and cantilever column tests indicate approximately the same relationship between story drift and damage.
2. For DS 3 and SMF, cantilever column test data suggest that damage initiates at much smaller drift levels than do frame sub-assemblage data (median story drift at the onset of damage of 3.5% versus 5.3%). This is attributed to the fact that at large drift levels, significant inelastic action occurs within the joint, which is not captured in the numerical model used to estimate story drift from cantilever column drift.

With respect to damage progression for different categories of frames:

1. For DS 0-1, the median drift at which the damage state initiates in all categories of frames is approximately the same. A notable exception is DS 0 for ASCE2 frames, for which the median drift is much larger. This discrepancy will be investigated further, given that the larger size of the data set and the moderate c.o.v., do not provide an simple explanation. Another notable exception is DS 0 for the OMF-CYSH / ASCE6 frames, for which the median drift is significantly smaller than for other frame categories. This is attributed to i) the high coefficient of variation on the data suggesting significant uncertainty in the data as well as the extremely small drift associated with failure of this category of frame.
2. For DS 2, the median drift at which the damage state initiates is largest for SMF systems and becomes progressively smaller as frame design and detailing requirements become less

onerous. The one exception to this is the NCF / ASCE3 frames, which have a median drift at the onset of DS 3 that is approximately the same as that for SMF. This exception is attributed to the small size of the NCF / ASCE3 data set and the relatively large c.o.v. for the data.

3. For DS 3, the median drift at which the damage state initiates is largest for SMF systems and becomes progressively smaller as frame design and detailing requirements become less onerous. In particular, the median drift associated with onset of DS 3 is 0.74% for frames included in the OMF-CYSH / ASCE6 category, which have high column axial loads and exhibit flexure-shear or shear failure of columns under lateral loading.

Table 4: Damage State Statistics for Different Data Sets

		Frame Sub-assemblage Data						Cantilever Column Data						Combined Data		
		SMF	ASCE 1	IMF	ASCE 2	OMF-BYJS	NCF / ASCE3	SMF	IMF	OMF-BYS / ASCE4	OMF-CYSM / ASCE5	OMF-CYSH / ASCE6	NCF / ASCE3	SMF	IMF	NCF / ASCE3
Damage State C	no. of specimens	6	18	22	10	9	17			1	4	10	1			18
	median drift (%)	0.72	0.57	0.54	0.25	0.40	0.75			0.19	0.56	0.15	0.59			0.73
	coeff. of var. (%)	0.59	0.68	0.79	0.12	0.42	0.74				0.04	0.74				0.72
Damage State 0	no. of specimens	13	36	41	20	18	5	27	6	2	5	10	1	40	47	6
	median drift (%)	0.80	1.09	1.16	1.85	1.00	1.00	1.04	1.25	1.15	1.42	0.19	0.59	1.00	1.20	0.79
	coeff. of var. (%)	0.49	0.60	0.49	0.47	0.85	0.37	0.39	0.15	0.11	0.03	1.53		0.43	0.47	0.37
Damage State 1	no. of specimens		2	3	3	2	1				1					
	median drift (%)		2.10	2.00	2.00	1.75	2.00				3.02					
	coeff. of var. (%)		0.07	0.06	0.05	0.20										
Damage State 2	no. of specimens	7	24	26	8	7	4	7		1	3	2		14		
	median drift (%)	2.77	2.36	2.36	1.78	2.00	2.75	3.11		2.11	1.59	1.37		2.83		
	coeff. of var. (%)	0.28	0.32	0.35	0.24	0.22	0.45	0.30			0.21	0.72		0.27		
Damage State 3	no. of specimens	4	18	30	22	16	17	5		1	4	10	2	9		19
	median drift (%)	5.25	4.24	3.63	3.00	3.06	2.00	3.54		2.61	2.44	0.71	1.66	4.08		2.00
	coeff. of var. (%)	0.14	0.20	0.31	0.20	0.27	0.42	0.17			0.07	0.77	0.37	0.25		0.43

Fragility Functions

To develop fragility functions for use in practice, first the data in Table 4 were reviewed to determine if unique suites of fragility functions were required for all of the identified frame categories. Given the large number of frame categories and that frame categories were established using ACI Code and ASCE 41-06 criteria, it was expected that there would be some overlap in fragility functions. Also, it was expected that the relatively small size of some of the data sets would not justify development of a unique suite of fragilities. Consideration of observed specimen response and review of the above data suggested the groupings listed in Table 5, which includes the same fragility functions for frames that exhibit flexure-shear or shear response of beams or columns with moderate axial loads (ASCE4 and ASCE5).

For each damage state, *dm*, drift data were assumed to be lognormally distributed: $P[DM \ge dm | DP = x] = \Phi\left(\frac{\ln\left(x/\theta\right)}{\beta}\right)$, where DM is the damage state of the frame subassemblage, x is the maximum story drift experienced by the frame subassemblage during the earthquake, and Φ is the normal cumulative distribution function. Given the relatively small size of some of the data sets, the Method of Maximum Likelihood was used initially to determine θ and β to provide a best fit to the data. These computed θ and β values were approximately, but not exactly, equal to the median and c.o.v. for each data set. Ultimately, θ and β were adjusted to facilitate application in practice, and fragility function parameters recommended for design and evaluation are listed in Table 5. Adjustments included rounding θ values to the nearest 0.25%, increasing β to account for uncertainty associated with actual building conditions and lack of data following the recommendations provided in Appendix F of the ATC-58 "Guidelines for Seismic Performance Assessment of Buildings", and assigning θ and β values for DS 1 such that θ values were consistent with the few data points available for this damage state and fell between median drifts associated with DS 0 and DS 2 and β values reflected the uncertainty associated with the chosen θ values. Fig. 2 shows, for selected frame categories, empirical fragilities and smoothed fragilities that employ the θ and β values listed in Table 5.

Table 5: Fragility function parameters recommended for design and evaluation

	Damage State 1		Damage State 2		Damage State 3	
	θ	β	θ	β	θ	β
SMF	2.00	0.40	2.75	0.30	5.00	0.30
ASCE 1	2.00	0.40	2.50	0.30	4.00	0.30
IMF	2.00	0.40	2.50	0.30	3.50	0.40
ASCE 2 / OMF-BYJS	1.75	0.40	2.25	0.30	3.25	0.40
ASCE4 / ASCE 5	1.50	0.40	2.00	0.30	2.50	0.40
NCF / ASCE3	1.50	0.40			2.00	0.40
ASCE 6	0.25	0.40			0.50	0.50

Summary and Conclusions

Fragility functions were developed to predict damage progression as a function of story drift for seven categories of concrete moment frames. Frames were categorized using ACI Code requirements as well as component categories defined in ASCE/SEI Standard 41-06, Supplement 1. Three damage states were considered. These damage states are defined by the extent of concrete cracking, spalling and crushing as well as loss of lateral strength. Each damage state is associated with a specific set of repair activities and may be used as a basis for estimating repair costs and downtime. Data from laboratory tests of frame subassemblages and column tests were used as a basis for defining fragility function parameters.

The results of this study show that for most frames, detailing and design requirements have limited impact on the drift at which significant cracking and spalling occur. Specifically, frame categories SMF through NCF develop damage requiring epoxy injection at drifts ranging from 2.0% to 1.5% and develop damage requiring patching of concrete at drifts ranging from 2.75% to 2.0%. However, detailing and design requirements have a significant impact on the drift at which replacement of concrete (and possibly reinforcing steel) is required, with the median drift at onset of this damage state ranging from 5.0% for SMF to 2.0% for NCF. Additionally, the results show that frames with columns with moderate to heavy axial loads and inadequate transverse reinforcement are damaged at extremely small drift demands.

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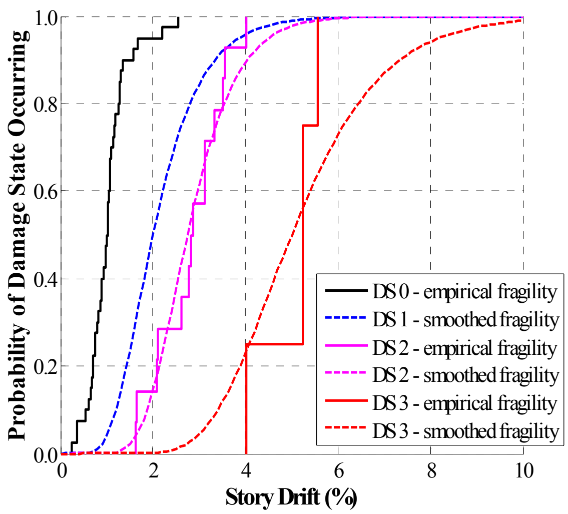
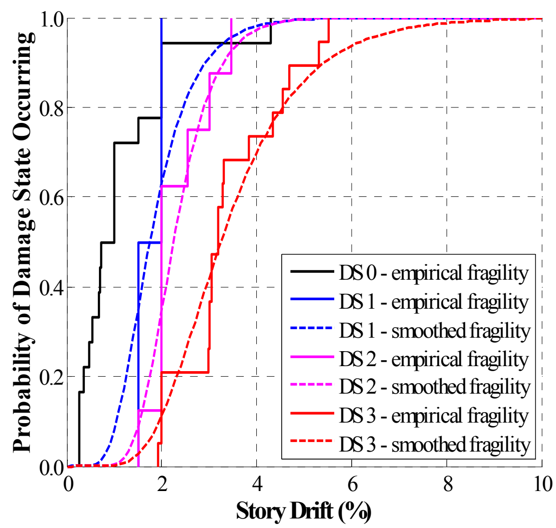
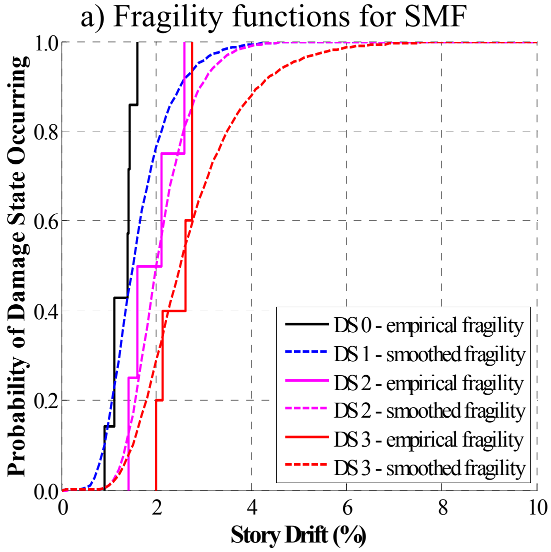


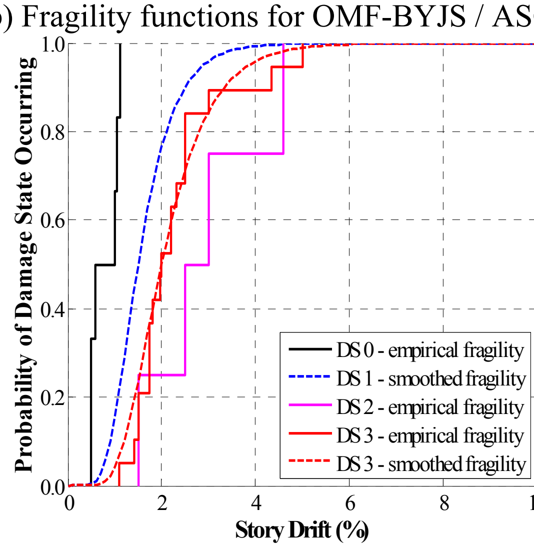
a) Fragility functions for SMF



b) Fragility functions for OMF-BYJS / ASCE 2



c) Fragility functions for ASCE4/5



d) Fragility functions for NCF/ASCE 3

Figure 2: Fragility functions for concrete frames