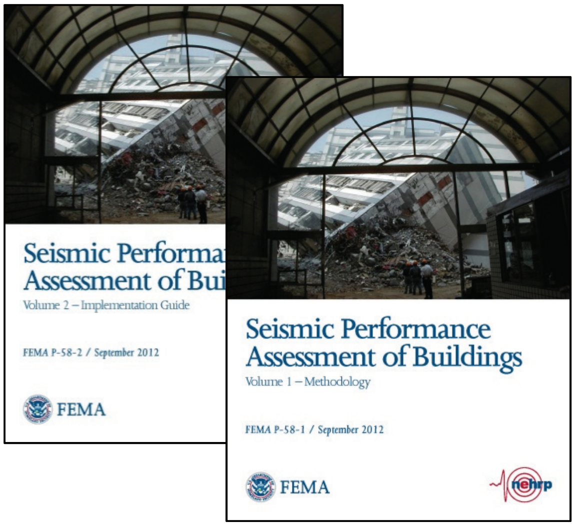


Background Document

FEMA P-58/BD-3.8.6

Fragility Functions for Reinforced Concrete Moment Frames

Prepared by

Laura Lowes and Jingjuan Li
Dept. of Civil & Environmental Engineering
Box 352700
University of Washington
Seattle, Washington 98195

Submitted to

APPLIED TECHNOLOGY COUNCIL
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065
www.ATCouncil.org

Prepared for

FEDERAL EMERGENCY MANAGEMENT AGENCY
U.S. Department of Homeland Security
500 C Street, SW
Washington, D.C. 20472

August 24, 2009

Background Documentation

FEMA P-58 Background Documents are a series of reports documenting the technical background and source information for key aspects of the FEMA P-58 methodology and its implementation. These reports were developed over the course of the 10-year ATC-58/ATC-58-1 Projects funded under FEMA Contracts EMW-2001-RP-0056 and HSFEHQ-06-D-1105.

Background Documents were developed by consultants, serving at various levels within the project hierarchy, reporting the results of: (1) decisions on technical development protocols; (2) focused studies on the development of key aspects of the methodology; (3) documentation of recommended procedures; and (4) collection of available data for the development of structural and nonstructural fragilities. They were initially intended to serve as a record of the technical state-of-knowledge at the time they were produced, and as resources for the development of the eventual project reports. As such, they represent a snapshot in time, and may, or may not, match the technical content, recommended procedures, or data incorporated into the final methodology and its implementation.

This Background Document is intended for the purpose of providing supplemental knowledge to users of the FEMA P-58 methodology. Information contained herein has not been independently verified for accuracy as a stand-alone document, and may have been superseded in its final implementation within the methodology. Users of information in this document assume all liability arising from such use.

Cover illustration – Primary resource documents for the FEMA P-58 *Seismic Performance Assessment of Buildings, Methodology and Implementation* series of products: FEMA P-58-1, *Volume 1 – Methodology*, and FEMA P-58-2, *Volume 2 – Implementation Guide*.

Fragility functions for Reinforced Concrete Moment Frames

Developed by Laura Lowes, Jingjuan Li and
ATC 58 Structural Performance Products Team

24 Aug 2009

1 Introduction

This document summarizes the development of fragility functions for reinforced concrete (RC) frames. Frames that meet ACI Code (ACI Com. 318 2008) requirements for Special Moment Frames (SMF), Intermediate Moment Frames (IMF) and Ordinary Moment Frames (OMF) as well as those that do not comply with these requirements (NCF) are considered. Additionally, frames that meet conditions specified in Chapter 6 of ASCE/SEI Standard 41-06, Supplement 1 (2007) are considered. These fragility functions were developed using i) data from laboratory tests of beam-column frame sub-assemblages and ii) data from laboratory tests of rectangular frame members (columns) in combination with a numerical model of a frame sub-assemblage. Test specimens represent bare frames, and the impact of the concrete floor slab on sub-assemblage response and damage progression is not considered.

Six damage states were defined to characterize damage progression in RC frames. These damage states are defined by the extent and severity of concrete cracking, concrete crushing, yielding of reinforcing steel, 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.

Ultimately, experimental damage data were collected to enable development of fragility functions for three of these damage states: Damage State 1 – Epoxy Inject Concrete, Damage State 2 – Patch Concrete, and Damage State 3 – Replace Concrete. For some frame categories, the data supported development of a single fragility function for Damage States 1 and 3.

For the purpose of developing consequence functions using the damage states and repair measures presented later in this report, the following frame parameters should be used:

- Story height: 14 feet
- Beam span: 30 feet
- Slab span: 25 feet
- Columns: 24×30 inches, reinforced with 2% longitudinal steel and #5 hoops and two cross-ties at 4 inches on center.
- Beams: 24×27 inches, reinforced with 0.6% longitudinal steel and #5 hoops and one cross-tie at 6 inches on center.
- Beam-column joints: reinforced with #5 hoops and two cross-ties at 4 inches on center.
- Slabs: 8 inches thick, reinforced with #5 bars at 12 inches on center top and bottom.
- Concrete: 5,000 psi concrete
- Rebar: Grade A706

2 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 test for development of fragility functions for RC frames as the test specimens are complete (with the exception that slabs are not included) 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 test specimens from 24 test programs for which damage data were available for use in developing fragility functions. However, 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.

The following sections present the criteria used to identify specimens for this study and the process used to compute story drift given displacement data from a cantilever column test.

2.1 Frame Sub-assemblage Test Data

Figure 1 shows a typical frame sub-assemblage test specimen used in this study. Test 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 two members. To limit dispersion of the data, test specimens with slabs, eccentric beam-column connections, and beams extending in the out-of-plane direction were not included in the study.

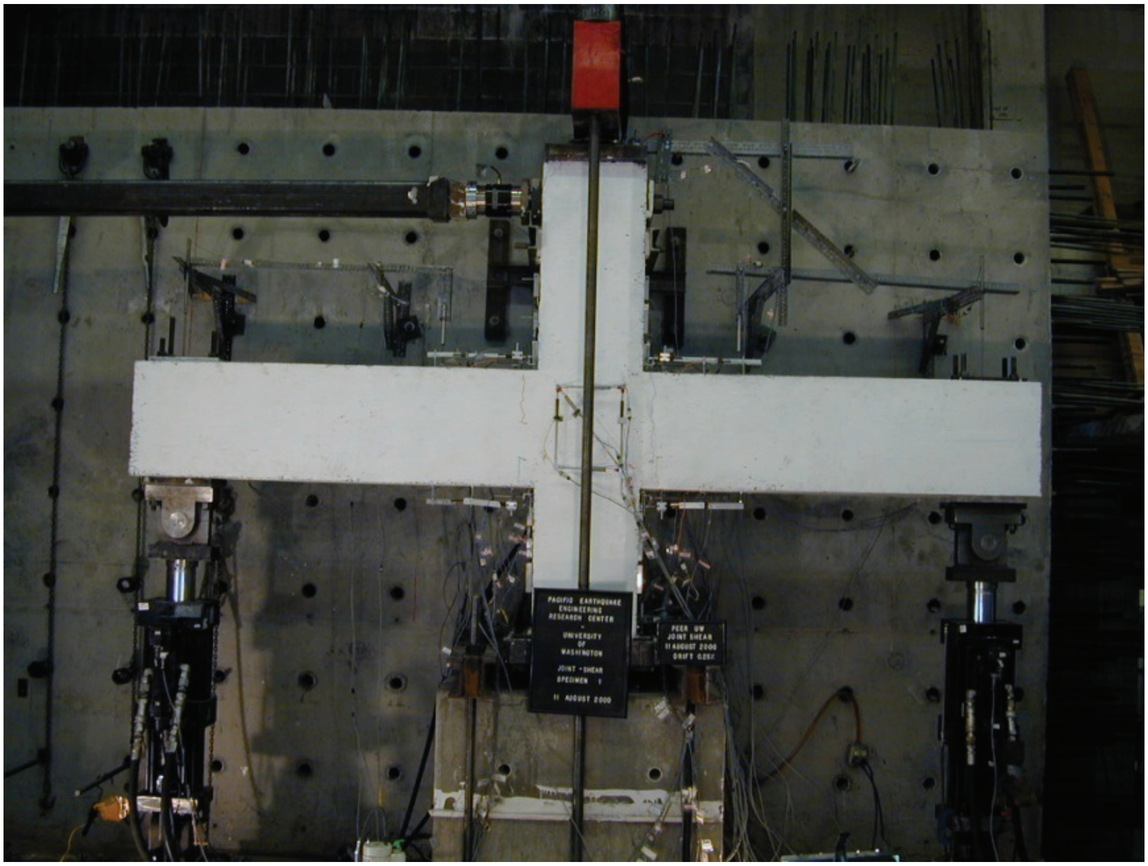


Figure 1: Frame sub-assemblage test specimen with lateral load applied as beam shears and column axial load applied (Walker 2001).

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. In some cases, shear loads were applied in opposite directions at the beam ends and reacted by shear loads at the top and bottom of the column (Figure 1). This results in a similar load distribution. Simulated earthquake load was applied pseudo-statically under displacement control, and typically specimens were subjected to three cycles each at increasing maximum story drift demands. In some cases, specimens were also subjected to a constant column axial load, which was intended to represent the gravity load in the column.

As part of this study, data from frame subassemblage tests were used to develop families of fragility functions for six categories of frames. These categories include the four frame categories identified in the ACI 318 Code as well as two categories developed for this study from the frame *component* categories identified in ASCE/SEI Standard 41-06 Supplement 1 for evaluation of existing concrete frames. These component categories are listed in Chapter 6 in Tables 6-7 (beams), Table 6-8 (columns) and Table 6-9 (joints) and are used for definition of nonlinear model parameters and acceptance criteria. These six frame categories are defined as follows:

1. **ACI Special Moment Frames (SMF).** The ACI Code identifies frames designed to maintain strength and integrity under strong ground shaking that produces multiple inelastic response cycles as Special Moment Frames (SMF). To achieve this objective, the ACI Code requires that SMF, in addition to meeting the provisions for frame design specified in Chapters 1–20 of the Code, meet the additional provisions in Sections 21.1.2-21.1.8, and 21.5-21.7. These provisions refer to general structural design and detailing (21.1.2-21.1.8), beam design (21.5), column design (21.6), and beam-column joint design (21.7). For the current study, 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 subassemblage met all of the additional Chapter 21 provisions except those specified in Sections 21.1.4, 21.1.5 and 21.6.4-21.6.5. The provisions in Sections 21.1.4 and 21.1.5 address concrete compressive strength and reinforcing steel properties. Researchers typically provide actual concrete and reinforcing steel strengths rather than concrete design strength or steel yield strength and ASTM specifications. Thus, actual strengths were used in all calculations and the provisions of Sections 21.1.4 and 21.1.5 were not considered. The provisions in Sections 21.6.4-21.6.5 address detailing of column transverse reinforcement and are intended to ensure that if columns yield in flexure, they exhibit significant ductility, do not lose axial load carrying capacity, and have adequate shear strength. Since SMF columns did not yield, compliance with these Code requirements would not be expected to affect performance and was ignored.

   It is expected that the seismic response of frames meeting to above criteria is controlled by flexural yielding of beams. For the current study, 13 frame sub-assemblage test specimens from 6 test programs were found that met the criteria for SMF For all of these sub-assemblages, response was controlled by flexural yielding of beams and most (80%) specimens did not exhibit strength loss in excess of 20% of maximum strength during testing.

2. **ASCE Category 1 Frames (ASCE1).** Using the component categories defined in Chapter 6 of ASCE/SEI Standard 41-06, the highest level of performance for an existing concrete frame could be expected from frames that comprise a) “beams controlled by flexure” (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. As these frame components do not meet the more stringent requirements defined by the ACI Code for SMF, they could be expected to see onset of a particular damage state at a lower median drifts. However, as the ASCE1 frame subassemblages do meet joint shear demand-capacity limits, they could be expected to see onset of a particular damage state at a higher drift level than IMF subassemblages that are not required to meet joint shear limits.

   Frames meeting the above criteria are expected to exhibit flexural yielding of beams and/or columns under earthquake loading; however, these frames could also be expected to exhibit limited ductility due to joint damage. For the current study, 38 test specimens from 15 test programs were found that met the above criteria. For these sub-assemblages, response was controlled by beam yielding and most specimens did not exhibit strength loss in excess of 20% of maximum strength during testing to significant story drift demands.

3. **ACI Intermediate Moment Frames (IMF).** The ACI Code identifies frames designed to achieve intermediate seismic performance or for construction in regions of intermediate seismic hazard as Intermediate Moment Frames (IMF). The ACI Code requires that IMF meet, in addition to the provision for frame design specified in Chapters 1–20, provisions in Sections 21.1.2-21.1.8 and 21.3. For the current study, specimens were classified as IMF if they a) met the Section 21.1 (general structural design and detailing) provisions as required for SMF and b) met provisions in Section 21.3 (definition of seismic shear and detailing of beams and column to provide moderate ductility) but not those required for SMF in Sections 21.5-21.7.

   Frames meeting the above criteria could be expected to exhibit flexural yielding of beams and/or columns under earthquake loading; these frames could also be expected to exhibit limited ductility due to joint damage or flexure-shear response in yielding beams and/or columns. For the current study, 61 test specimens from 16 test programs were found that met the criteria for IMF. In general, these specimens were classified as IMF due to inadequate joint design. Thus, response of these sub-assemblages, was controlled by beam yielding (70%) or joint failure prior to beam yielding (30%). For most IMF (60%) specimens, strength loss exceeded 20% of maximum strength during testing to significant story drift demands

4. **ASCE Category 2 Frames (ASCE2).** Using the component categories defined in Chapter 6 of ASCE/SEI Standard 41-06, a moderate level of performance could be expected from an existing concrete frame that comprises a) "beams controlled by flexure" (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 respond in flexure, and c) interior joints that do not have compliant transverse reinforcement and a shear demand-capacity ratio less than 1.2. The ASCE2 frame requirements are similar to the ACI IMF requirements. Thus, both frame categories could be expected to exhibit a similar pattern of damage progression and have similar families of fragility functions.

   Frames meeting the above criteria are expected to exhibit flexural yielding of beams and/or columns under earthquake loading, with limited ductility due to joint damage. For the current study, five (5) test specimens from three (3) test programs were found that met the above criteria. For these sub-assemblages, response was controlled by joint failure prior to beam yielding and strength loss exceed 20% of maximum strength during testing to significant story drift demands.

5. **ACI Ordinary Moment Frames Controlled by Beam Yielding or Joint Failure (OMF-BYJF).** The ACI Code identifies frames designed for construction in regions of low seismic hazard (ASCE 7 Seismic Design Category B) as Ordinary Moment Frames (OMF). The ACI Code requires that OMF meet, in addition to the provision for frame design specified in Chapters 1-20, provisions in Section 21.1.2-21.1.8 and 21.2. For the

current study, specimens were classified as OMF if they a) met the Section 21.1 (general structural design and detailing) provisions as required for SMF and b) the Section 21.2 requirements.

The seismic response of frames meeting the ACI Code criteria for OMF could be controlled by any number of different mechanisms including flexural yielding of beams or columns or joint failure prior to flexural yielding; additionally, frames could be expected to exhibit reduced ductility due to joint damage following beam or column yielding or flexure-shear response of beams or columns. For the current study, 18 test specimens from seven (7) test programs were found that met the above criteria. For these sub-assemblages, response was controlled by beam yielding (60%) or joint failure prior to beam yielding (40%). For most (94%) specimens, strength loss exceeded 20% of maximum strength during testing to significant story drift demands. Thus, specimens in this category are identified as OMF-BYJF to indicate that response was controlled by beam yielding (BY) in combination with joint failure (JF).

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. For the specimens employed as part of this study this included frames subassemblages in which column longitudinal steel was spliced above the joint, beam top and bottom longitudinal reinforcement was not continuous through the joint, or the Code-specified minimum volume of transverse reinforcement was not provided.

   It is expected that the seismic response of non-compliant frames will be determined by failure of one or more "non-compliant" details. For the current study, 19 test specimens from five (5) test programs were found that met the above criteria. For these sub-assemblages, response was controlled by beam yielding (26%), joint failure prior to beam yielding (26%), or column yielding followed by joint failure (47%). For most (73%) specimens, strength loss exceeded 20% of maximum strength during testing to significant story drift demands.

   The seismic response of frames included in this category is expected to be 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), b) "Beams controlled by inadequate embedment into beam-column joint (category iv beams in Table 6-7), or c) "Column controlled by inadequate development or splicing along the clear height (Condition iv columns in Table 6-8**)**. Thus, specimens in this category are identified also as ASCE 3.

Table 1 provides statistics for design parameters that could be expected to significantly affect the seismic performance of the frame sub-assemblage test specimens used in this study. Data for individual specimens are provided in Appendix A. In Table 1, design parameters are defined on the basis of the ratio of the provided to required quantity, where the required quantity is that required by the ACI Code for SMF. References to specific sections of the ACI Code are provided. 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.

The fragility functions developed as part of this study are appropriate for use with specimens that fall within the ranges of the dataset presented in Table 1 and Table 11 in Appendix A.

Table 1: Design parameters for frame sub-assemblages used in the current study. 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 required quantity is listed.

Category	No. Spec's		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	13	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
		c.o.v.	0.16	0.28	0.33	0.26	0.26	0.32	0.30	0.94	0.40	0.39	0.26	0.51
ASCE1	39	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
		c.o.v.	0.35	0.83	0.38	0.35	0.35	0.21	0.31	0.82	0.43	0.65	0.23	0.55
IMF	56	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
		c.o.v.	0.42	0.74	0.47	0.49	0.49	0.15	0.29	0.81	0.45	0.77	0.28	0.53
ASCE2	5	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
		c.o.v.	0.22	0.32	0.30	0.56	0.56	0.15	0.26	0.43	0.18	0.49	0.40	0.12
OMF-BYJF	18	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
		c.o.v.	0.33	1.55	0.79	0.75	0.75	0.28	0.84	0.39	0.37	0.88	0.52	0.48
NCF / ASCE3	19	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
		c.o.v.	0.37	1.68	1.48	0.51	0.51	0.27	0.79	0.57	0.56	1.14	0.48	0.79

2.2 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 from cantilever column tests. Specifically, damage-deformation data for square columns were from the PEER Structural Performance Database were used. Damage progression in 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 sub-assemblage 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 sub-assemblage.

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 0.1Agfc), 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 0.1Agfc), 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 such as that idealized in Figure 2. The model shown in Figure 3 and 4 was used to estimate story drift from measured column displacement, assuming that column displacement represents beam displacement in the frame. The following assumptions were made in defining and applying the model:

1. The measured load-displacement response of the cantilever column is equal to the load-displacement response of each of the beam segments in the frame sub-assemblage.
2. Story drift of the frame sub-assemblage 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 out-of-plane width is same as beam out-of-plane width.
5. Column depth is 20 times the diameter of beam longitudinal reinforcement, such that Section 21.7 of the ACI Code, which addresses design of beam-column joints in SMF, is satisfied.
6. Assuming the ratio of column to beam flexural strength exceeds 1.2 and following the recommendations of ASCE/SEI Standard 41-06, the column is assumed rigid within the joint and the beam is assumed flexible.[1]
7. The flexural stiffness of the column in the frame sub-assemblage model may be computed from the measured flexural stiffness of the cantilever column at yield as described below.

Employing the model in Figure 3 and Figure 4 and the above assumptions, frame sub-assemblage drift, δ_{frame}, is computed

$$\delta_{frame} = 2\frac{\Delta_b}{L_b} + \theta_j$$

where

Δ_b= column tip displacement measured in the laboratory

$$\theta_j = V_b \cdot L_b \cdot \frac{2}{3EI_{c\_eff}} \cdot \frac{\left(\frac{L_c - h_b}{2}\right)^3}{h_b^2 + 4h_b\left(\frac{L_c - h_b}{2}\right) + 4\left(\frac{L_c - h_b}{2}\right)^2}$$

[1] For the case of columns from the PEER database with moderate to high axial loads (axial load greater than $0.1A_gf_c$), which are assumed to represent columns in a frame, beams are assumed rigid in the joint and columns are assumed flexible within the joint.

with

V_b = measured column shear load corresponding to displacement Δ_b

$EI_{c_{eff}}$ = frame sub-assemblage column effective elastic stiffness

and L_b, L_c, h_b, and h_c are as shown in Figure 3. Assuming that column effective elastic stiffness may be computed from the measured flexural stiffness of the column at yield:

$$EI_{c_{eff}} = EI_{b\_yield}\frac{I_c}{I_b} = \frac{V_{b\_yield}}{\Delta_{b\_yield}}\frac{(L_{b\_lab})^3}{3}\left(\frac{h_c}{h_b}\right)^3$$

where

EI_{b_yield} = measured effective elastic stiffness of the cantilever column at yield

I_c = gross-section moment of inertia of the column in the frame sub-assemblage

I_b = gross-section moment of inertia of the cantilever column tested in the laboratory

V_{b_yield} = cantilever column shear force measured in the laboratory at yield

Δ_{b_yield}= cantilever column tip displacement measured in the laboratory at yield

$L_{b\_lab} = \frac{L_b}{2}$

and h_b, and h_c are as shown in Figure 3.

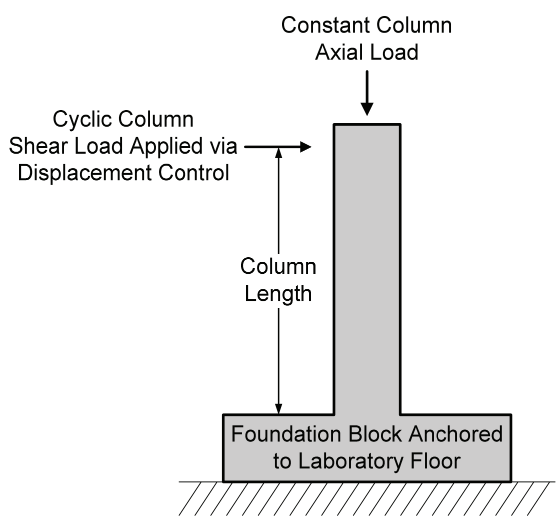


Figure 2: Cantilever column test setup

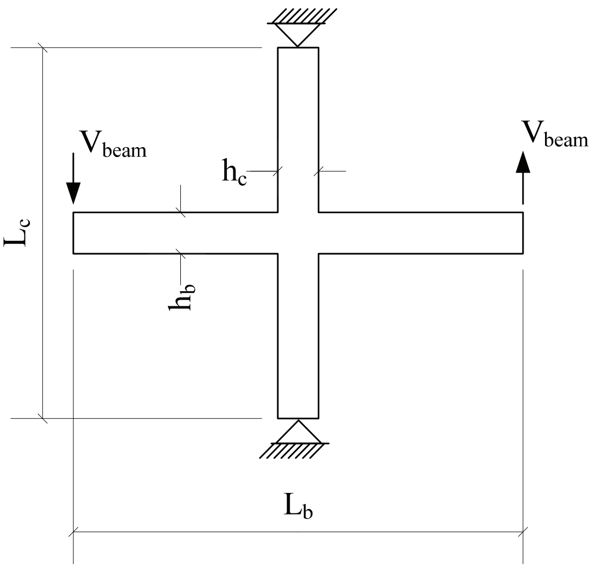


Figure 3: Frame sub-assemblage geometry and loading

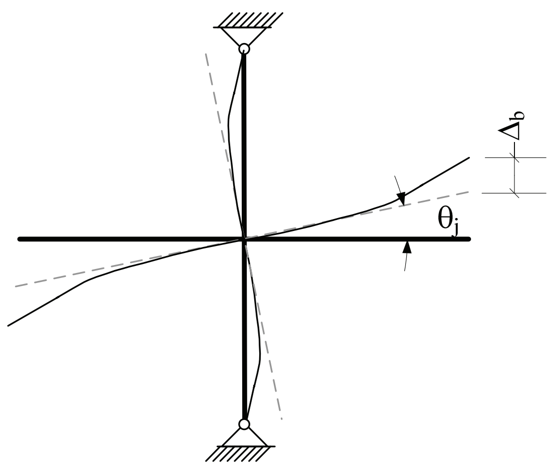


Figure 4: Frame sub-assemblage deformation due to elastic bending of the column and deformation of the beam.

As part of this study, data from cantilever column tests were used to develop families of fragility functions for six categories of frames. These include frame categories identified in the ACI 318 Code as well as categories developed for this study from the frame *component* categories identified in ASCE/SEI Standard 41-06 for evaluation of existing concrete frames. These six frame categories are defined as follows:

1. **ACI Special Moment Frames (SMF).** The expected seismic response of SMF is flexural yielding of beams. Thus, only columns with low axial loads were considered for inclusion in this category. Columns meeting the Section 21.5 ACI Code requirements (requirements for beams in SMF) were considered to be representative of beams in SMF. For the current study, 26 cantilever-column test specimens from 9 test programs were found meeting these criteria. These specimens exhibited flexural response under lateral loading.

2. **ACI Intermediate Moment Frames (IMF).** The desired seismic response of IMF is flexural yielding of beams, and this was the observed response mechanism for most frame sub-assemblage test specimens categorized as IMF. Thus, columns with low axial loads meeting the requirements of Sections 21.3.3 and 21.3.4 (shear strength and detailing requirements for beams in IMF) but not Section 21.3.5 were considered to be representative of beams in IMF. Six (6) specimens from one (1) test program were found to add to the IMF dataset.

3. **ACI Ordinary Moment Frames Controlled by Flexure-Shear or Shear Response of Beams (OMF-BYS/ASCE4).** As discussed above, frames meeting the ACI Code requirements for OMF could be expected to exhibit a number of different response mechanism under lateral loading including flexure-shear or shear response of beams or columns. However, no frame subassemblage 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 (as described above), and exhibiting flexure-shear or shear response were included in this category. Specimens in this category are identified as OMF-BYS to indicate that response is controlled by beam yielding with shear or shear failure of beams. Two (2) specimens from two (2) test programs were found that met these criteria. The response of one of these specimens was controlled by shear; the second specimen exhibited flexure-shear response.

   The seismic response of frames included in this category is expected to be representative of an existing frame that includes, using the ASCE/SEI Standard 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. Specimens in this category are identified as OMF-CYSM to indicate that response is controlled by column yielding with shear or shear failure of columns with moderate axial loads. Five (5) specimens from four (4) test programs were found that met these criteria. Forty percent (40%) of these specimens exhibited flexure-shear response while (60%) exhibited shear response.

The seismic response of frames included in this category is expected to be representative of an existing frame that includes, using the ASCE/SEI Standard 41-06 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. Specimens in this category are identified as OMF-CYSH to indicate that response is controlled by column yielding with shear or shear failure of columns for columns with high axial loads. Nine (9) specimens from four (4) test programs were found that met these criteria. Eighty-nine percent (89%) of these specimens exhibited flexure-shear response while (11%) exhibited shear response.

   The seismic response of frames included in this category is expected to be representative of an existing frame that includes, using the ASCE/SEI Standard 41-06 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.

7. **ACI Non-compliant Frames (NCF/ASCE3).** The frame subassemblage data set was supplemented with data from the PEER Column Database for specimens with designs that did not meet the requirements of Chapters 1-20 of the ACI Code. Specifically, damage data for two (2) cantilever column specimens from one (1) test program in which longitudinal steel was spliced with inadequate detailing or at the point of maximum moment demand were included. These specimens had axial load ratios less than 0.1Agfc and, thus, were considered to represent damage progression for beams in a frame. One of these specimens exhibited flexure-shear response; the other exhibited shear failure.

   The seismic response of frames included in this category is expected to be representative of existing frames that include, using the ASCE/SEI Standard 41-06 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 seismic performance of the column test specimens. The design parameters are defined on the basis of the ratio of the provided to required quantity, where the required quantity is that required by the ACI Code for SMF. References to specific sections of the ACI Code are provided. Because the response of column test specimens is assumed to represent that of beams in the moment frame, Code requirements are those pertaining to beams in SMF. In Table 2, s_t is the spacing for transverse steel, V_u is the component shear demand, V_n is the component shear strength, and ϕ is the strength reduction factor.

The data in Table 2 show that SMF specimens have design parameter statistics that are similar to those of the frame sub-assemblage test specimens. The data in Table 2 show also that IMF specimens do not meet shear strength requirements for SMF and have, on average, much higher shear stress demands than do the SMF specimens. Higher beam shear stress demands were found also for the IMF sub-assemblage specimens.

Given that numerical models and average configuration parameters from the frame sub-assemblage data set were used to transform column drift data to story drift data, column test specimen design parameters are not considered to significantly affect the range of applicability of the fragility functions.

Table 2: Design parameters for column test specimens used in the current study. For spacing of transverse reinforcement, s_t, the ratio of the provided to required quantity is listed.

Category	No. Spec's		Column Design			
			Axial load ratio	$\rho_{l\,(\%)}$	s_t (21.4.4.1)	$V_u/\phi V_n$ (21.4.5)
SMF	27	Min.	0.0	0.013	0.57	0.22
		Ave.	0.1	0.021	0.82	0.65
		Max.	0.1	0.036	0.96	0.96
		c.o.v.	0.4	0.25	0.15	0.33
IMF	6	Min.	0.0	0.016	0.76	1.84
		Ave.	0.0	0.016	0.76	1.90
		Max.	0.0	0.016	0.76	1.94
		c.o.v.	0.0	0.00	0.00	0.02
ASCE4	3	Min.	0.0	0.014	0.88	0.32
		Ave.	0.1	0.021	1.69	1.24
		Max.	0.3	0.032	4.54	4.97
		c.o.v.	0.9	0.24	0.56	0.80

Category	No. Spec's		Column Design			
			Axial load ratio	$\rho_{l\,(\%)}$	s_t (21.4.4.1)	$V_u/\phi V_n$ (21.4.5)
ASCE5	6	Min.	0.10	0.016	1.10	0.50
		Ave.	0.16	0.025	1.92	1.85
		Max.	0.45	0.031	3.23	5.16
		c.o.v.	0.51	0.161	0.34	0.73
ASCE6	10	Min.	0.61	0.018	1.12	0.29
		Ave.	0.75	0.022	1.94	0.94
		Max.	0.90	0.025	4.78	2.93
		c.o.v.	0.13	0.092	0.75	0.77
ASCE3	2	Min.	0.07	0.019	4.50	1.41
		Ave.	0.08	0.025	4.52	1.71
		Max.	0.09	0.030	4.54	2.01
		c.o.v.	0.15	0.310	0.01	0.25

3 Damage States

Damage states for concrete frames were established using the results of previous studies. Recommendations for the repair of damaged concrete frames (ATC 1998, ACI 546R 1996), previous investigation of the repair of earthquake damaged reinforced concrete 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) suggest the damage states listed in Table 3 and repair activities described in Table 4 are appropriate for the current study. The following sub-sections provide additional information about damage states.

Table 3: Damage states

Damage State	Frame Damage Characteristics	Triggering Damage Characteristic
C	Damage to finishes: cosmetic finishes exhibit damage but residual concrete crack widths are too narrow to require repair.	Hairline cracking of concrete at beam-column interface, within beams and columns, or within joint OR beam and/or column longitudinal reinforcement yields
0	Concrete Cracking: beams, joints or possibly column exhibit residual crack widths that require epoxy injection.	Residual concrete crack widths for beams, columns or joint exceed 0.02 in. OR yielding of beam or column longitudinal reinforcement OR yielding of joint transverse reinforcement.
1	Concrete Cracking: beams, joints or possibly column exhibit residual crack widths that require epoxy injection.	Residual concrete crack widths exceed 0.06 in. (1.5 mm)
2	Concrete Spalling: slabs, beams, joints or possibly columns exhibit spalling of cover concrete that exposes transverse but not longitudinal reinforcing steel.	Spalling of cover concrete possibly exposing transverse reinforcement.
3	Concrete Crushing: slabs, beams or joints exhibit concrete spalling that exposes longitudinal steel or crushing of core concrete.	Spalling of beam, column or joint cover concrete exposes longitudinal reinforcement OR strength loss initiates in laboratory testing.
4	Steel yielding, buckling and fracture: Reinforcing steel experiences severe inelastic deformation and requires replacement	Beam, or possibly column, longitudinal steel exhibits severe inelastic tensile strain, buckling or fracture.

Table 4: Repair activities

Damage State	Repair Activity	Details of Repair Activity[1]
C	Cosmetic Repair	Remove furnishings, ceilings and mechanical, electrical and plumping systems (as necessary) 5 feet either side of damaged area. Replace and repair finishes. Replace furnishings, ceilings and mechanical, electrical and plumping systems (as necessary)
0,1	Epoxy Inject Concrete	Remove furnishings, ceilings and mechanical, electrical and plumbing systems (as necessary) 5 feet either side of damaged area. Clean area adjacent to the cracks. Prepare cracks, as necessary, to receive the epoxy injection. Inject cracks. Replace and repair finishes. Replace furnishings, ceilings and mechanical, electrical and plumping systems as necessary.
2	Patch Concrete	Remove furnishings, ceilings and mechanical, electrical and plumbing systems (as necessary) 8 feet either side of damaged area. Clean area adjacent to the damaged concrete. Prepare spalled concrete and adjacent cracks, as necessary, to be patched and to receive the epoxy injection. Patch concrete with grout. Replace and repair finishes. Replace furnishings, ceilings and mechanical, electrical and plumping systems as necessary)
3	Replace Concrete	Remove furnishings, ceilings and mechanical, electrical and plumbing systems (as necessary) 15 feet either side of damaged area. Shore damaged member(s) a minimum of one level below (more levels may be required). Remove damaged concrete at least 1 inch beyond the exposed reinforcing steel. Place concrete forms. Place concrete. Remove forms. Remove shores after one week. Replace and repair finishes. Replace furnishings, ceilings and mechanical, electrical and plumping systems (as necessary).
4	Replace Reinforcing Steel	Remove furnishings, ceilings and mechanical, electrical and plumbing systems (as necessary) 15 feet either side of damaged component. Shore damaged member(s) a minimum of one level below (more levels may be required). Remove damaged component. Place and splice (as necessary) new reinforcing steel to existing, undamaged reinforcing. Place concrete forms. Place concrete. Remove forms. Remove shores after one week. Replace and repair finishes. Replace furnishings, ceilings and mechanical, electrical and plumping systems (as necessary).

1. See the subsections below for typical quantities of repair.

3.1 Damage State C: Cosmetic Repair

If component damage due to earthquake loading is limited to concrete cracking with narrow crack widths, repair will not be required to restore strength and stiffness; however, repair likely will be required to replace and restore surface finishes. Damage State *C* comprises activities to remove, repair and/or replace surface finishes, and is equivalent to Cosmetic Repair 1 defined in

FEMA 308 (ATC 1998). Discussions with structural engineers and contractors indicate that the extent of damage that an owner considers to require repair of surface finishes is highly variable. Examples of Damage State *C* are shown in Figure 5 and Figure 6.

Structures exhibiting Damage State *C* do not require structural repair.

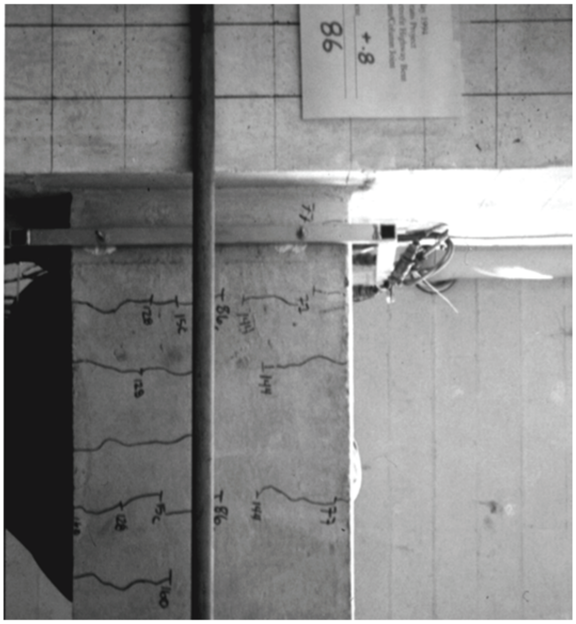


Figure 5: Laboratory testing of bridge frame sub-assemblage. Column shows initial flexural cracking. Cracks are marked in black (Lowes 1995). Cracking is representative of Damage State *C*.

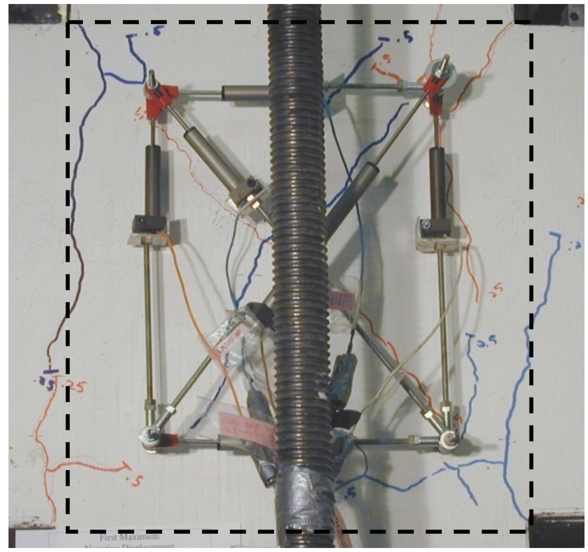


Figure 6: Laboratory testing of O sub-assemblage. Here, beam-column joint region (outlined by dashed line) shows initial cracking. Cracks are marked in red and blue. (Lehman et al. 2002). Damage is representative of Damage State *C*.

3.2 Damage States 0 and 1: Epoxy Injection of Concrete Cracks

If earthquake damage results in significant opening of concrete cracks, repair may be required to restore component stiffness and strength as well as to ensure that earthquake damage does not make the component vulnerable to water infiltration, corrosion, fire damage, etc. Injection of cracks with an epoxy resin or a cementitious grout is a commonly employed repair method for reinforced concrete components damaged under multiple types of loading (ACI Com. 546 1996) and previous research shows epoxy injection may be used to restore components to pre-earthquake conditions (Filiatrault 1992, French et al. 1990). Damage State 1 comprises activities to repair cracked concrete and is comparable to Structural Repair 1 defined in FEMA 308 (ATC 1998).

In linking epoxy resin injection with damage states, identifying the crack widths for which epoxy injection is an appropriate repair method is the critical issue. Crack widths as small at 0.002 in. (ATC 1998, Barlow 2008) can be epoxy injected. In practice, residual crack widths in excess of 0.01-0.02 in. are routinely epoxy injected (Barlow 2008), and FEMA 308 references ACI Com. 224 (1994a) to note that crack widths up to 0.012 in. can be tolerated in humid or moist air conditions. However, after the Loma Prieta and Northridge earthquakes, customary practice was to inject cracks approximately 0.06 in. (1/16th in.) and larger (Hamburger et al., 2008). Thus, in keeping with customary practice, it was decided that residual crack widths in excess of approximately 0.06 in. would determine initiation of damage requiring epoxy injection to restore strength and stiffness and ensure that a structure would not be vulnerable to environmental hazards.

In the past, researchers rarely measured residual crack widths, and in previous studies addressing beam-column joints (Pagni and Lowes 2006, Brown and Lowes 2007) maximum concrete crack width under load has been used in lieu of residual crack width data. However, while researchers often identify the drift level at which measurable cracking is initially observed, they rarely identify the point at which moderate cracking (i.e., crack widths of approximately 0.06 in.) is observed. For the laboratory specimens considered in this study, three data points were found linking story drift with moderate cracking (crack widths > 0.06 in.) for IMFs. Thus, it was not possible to develop directly from laboratory data fragility functions defining the probably of crack widths in excess of 0.06 in.

Instead, data were assembled for two damage states associated with cracking and epoxy injection:

- Damage State 0 is defined by residual crack widths in excess of 0.02 in. In laboratory tests, specimens were considered to exhibit Damage State 0 if, under load, maximum crack widths in beams, columns or joints exceed 0.02 in. Specimens were also considered to exhibit Damage State 0 if beam or column longitudinal reinforcement yielded or joint transverse reinforcement yielded, as yielding of this reinforcement could be expected to result in cracks remaining open, i.e. residual cracking, upon unloading of the specimen.

- Damage State 1 is defined by residual crack widths in excess of 0.06 in. In laboratory tests, specimens were considered to exhibit Damage State 1 if, under load, maximum crack widths in beams, columns or joints exceed approximately 0.06 in.

Ultimately, fragility function parameters for Damage State 1 were determined by i) the fact that median drift for Damage State 1 should exceed that of Damage State 0, ii) the assumption that the median drift for Damage State 1 is approximately twice that of Damage State 0 (Lehman et al. (2001) define a similar damage state for bridge columns based on twice the yield displacement), and iii) using empirical data for Damage State 1. Figure 7 – 9 shows laboratory test specimens with damage that is representative of these damage states.

Calculation of loss associated with structural repair for Damage States 0 and 1 requires calculation of the length of crack to be injected (Barlow 2008). For frames for which response is determined by flexural yielding or flexure-shear response of beams and/or joint damage, images of frame subassemblage test specimens, such as that shown in Figure 7, suggest that crack length may be computed assuming the following:

1) Flexural cracks in beams will develop on the top and bottom of the beams and thus extend over the entire height of the beam under cyclic loading. Cracks are assumed to be spaced at 4 in. For Damage State 0, cracking requiring epoxy injection will extend a distance equal to half the beam depth along the length of the beam; for Damage State 1

cracking requiring epoxy injection will extend a distance equal to the beam depth along the length of the beam

2) Diagonal cracking in beam-column joints will occur parallel to both joint diagonals, be spaced at approximately 5 in., and be distributed over the entire joint area.

3) No cracking requiring epoxy injection will occur in columns.

4) Cracking in slabs will extend outward away from the beam axis and into the frame bay a distance equal to the ACI column-strip width (ACI 318 2008). Cracks will have the same spacing as observed in the beams, 4 in. Cracking will be distributed along the beam length a distance equal to half the beam depth.

For frames for which response is controlled by other mechanisms, an approach similar to that described above may be used to estimate crack length for epoxy injection, with exceptions described below.

1) For frames for which beam shear strength or non-compliant beam details determine response, crack patterns similar to those described above are expected with the exceptions that a) cracks in beams are expected to be oriented at 45 degrees and extend of the entire beam depths and b) cracking in beams and slab could be expected to extend along the entire clear span of the beam.

2) For frames for which response is controlled by flexural yielding of columns and joint damage, crack patterns similar to those described above for beams are expected in columns. Joint cracking similar to that described above is expected. Concrete cracking requiring injection would not be expected in beams or slabs.

3) For frames for which response is controlled by flexure-shear or shear action in columns or non-compliant column details, cracks in columns may be assumed spaced at 4 in., extend over a length equal to the member depth, and extend over the entire span of the column. Shear cracks may be assumed spaced to require injection along a length equal to the 1.4 times the member depth (i.e., cracks are assumed to be oriented at 45 degrees and extend over the entire depth of the member).

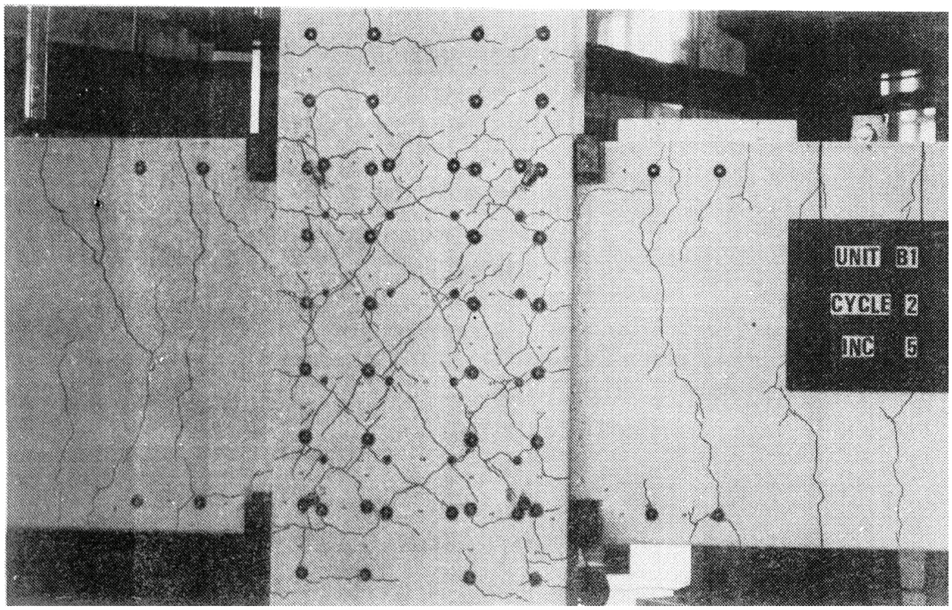


Figure 7: Special moment frame (SMF) sub-assemblage following testing to 0.82% story drift and yielding of beam longitudinal reinforcement. Beam, column and joint show cracking that is representative of Damage State 0. Figure 5.3 from Birss et al. (1987).

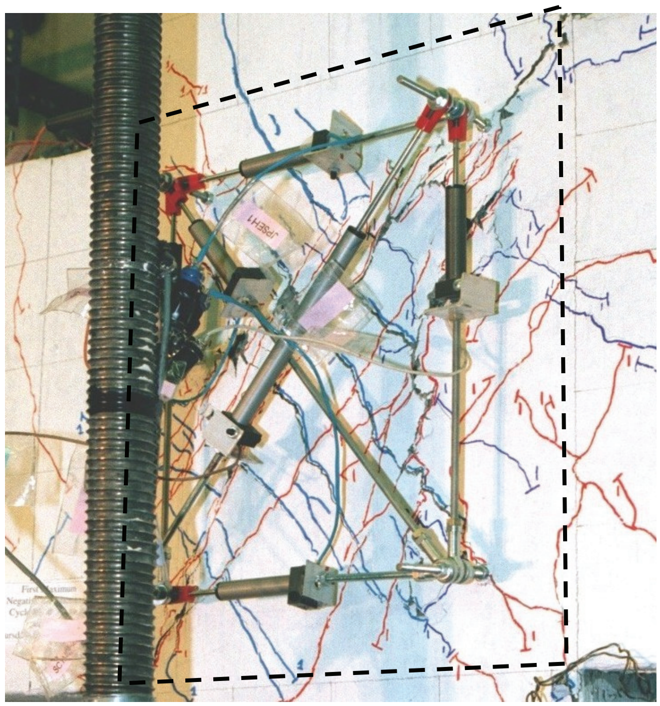
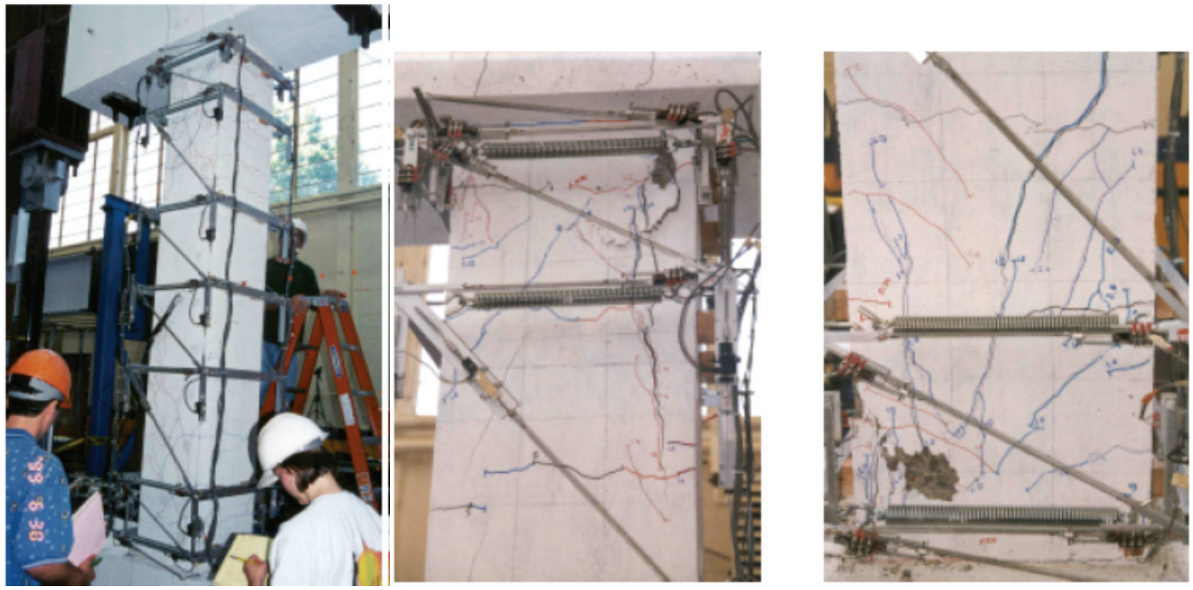


Figure 8: Laboratory testing of OMF sub-assemblage (see Figure 1). Here, concrete has cracked in the interior of the beam-column joint region (outlined by dashed line) and crack widths exceed 0.05 in. (Lehman et al. 2002).



a) ASCE5 category column test; crack widths > 0.03 in. **b) ASCE6 category column test, top (left) and bottom (right) shown, crack widths > 0.1 in.**

Figure 9: Laboratory testing of ASCE 5 and ASCE 6 column sub-assemblages. Here, concrete cracking is concentrated at column ends where flexural demands are highest (Sezen 2000).

3.3 Damage State 2: Patch Concrete

Under moderate to severe earthquake loading, damage may include spalling of surface concrete. In this case, replacement of the spalled concrete is required to ensure that exposed reinforcing steel is not vulnerable to corrosion, fire damage, etc. and to restore component strength and stiffness (Karayannis 1998). Patching is accomplished by removing any loosened concrete that has not spalled, cleaning the surface, and replacing the concrete with a mortar mix (ATC 1998). Damage State 2 does not include a significant effort to remove damaged concrete; Damage State 2 includes some of the activities identified in FEMA 308 (ATC 1998) Structural Repair 3.

In linking Damage State 2 with the previously defined damage states, the critical issue is to identify the extent of concrete spalling for which patching of spalled concrete is inadequate and a more extensive repair is required. Patching is considered to be inadequate if beam or column longitudinal reinforcement is exposed, as patching would not be expected to restore concrete-steel bond. Figure 10-12 provide examples of this damage state.

Calculation of loss associated with structural repair for Damage State 2 requires calculation of the length of crack to be injected (Barlow 2008) as well as the surface area to be patched. For frames for which response is determined by flexural yielding of beams and/or joint damage, images of frame subassemblage test specimens, such as that shown in Figures 8-10, suggest that crack length may be computed as in Damage State 1, with the exception that 1) crack spacing in the joint area will be reduced to 3 in. and 2) cracking in beams and slabs will be distributed over a length of the beam equal to the beam depth. The surface area of concrete to be patched can be estimated as follows:

1) Spalling of beam concrete will occur on the top and bottom of both beams, will extend vertically to approximately to mid-depth of the beam, and will extend a distance equal to 10% of the beam depth along the length of the beam.

2) Spalling will occur over 30% the joint area, on both sides of the joint.

3) No patching of column concrete will be required.

4) Spalling in slabs will extend outward away from the beam axis and into the frame bay a distance equal to half the ACI column-strip width (ACI 318 2008) and will extend over a length equal to 10% of the beam depth.

For frames for which response is controlled by mechanisms other than beam flexure in combination with joint damage, a similar approach may be used to estimate crack length for epoxy injection and concrete area for patching. Where column (beam) strength is expected to determine frame strength, cracking and spalling in beams (columns) may be neglected in estimating crack length and concrete area requiring repair. The process outlined for Damage States 0 and 1 may be used to estimate crack length for epoxy injection, with cracks spaced at 3 in. for Damage State 2. Concrete spalling in columns may be assumed to occur on both sides of the column extend over the entire column depth and extend away from the joint a distance equal to 10% of the column depth.

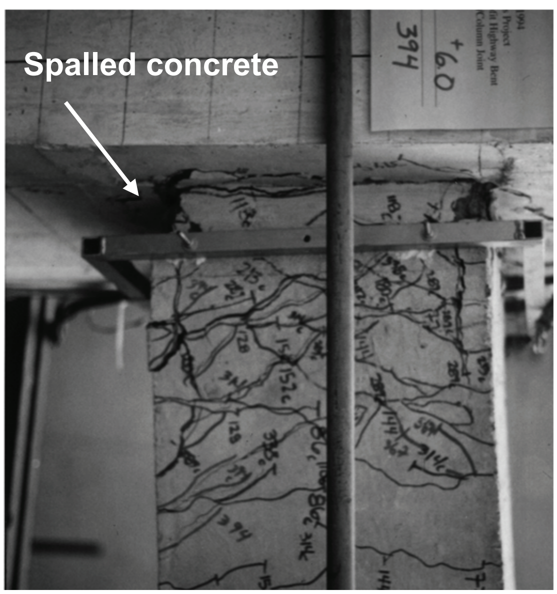


Figure 10: Laboratory testing of bridge frame sub-assemblage. Column shows initial spalling of cover concrete. (Lowes 1995).

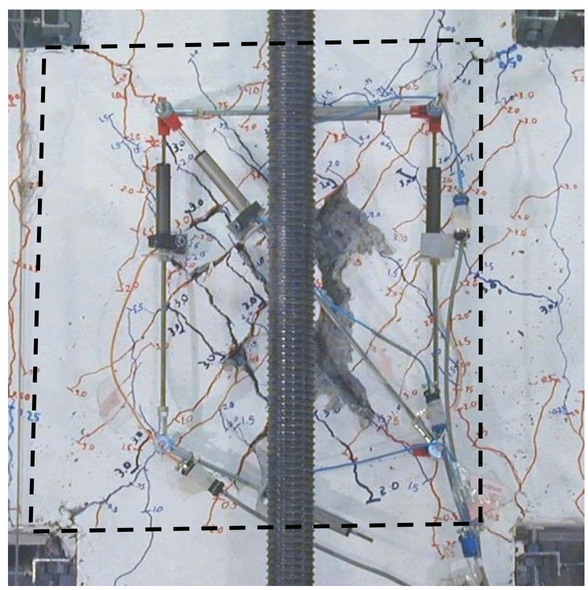


Figure 11: Laboratory testing of frame sub-assemblage (see Figure 1). Here, concrete has spalled in the interior of the beam-column joint region (outlined by dashed line). (Lehman et al. 2002).

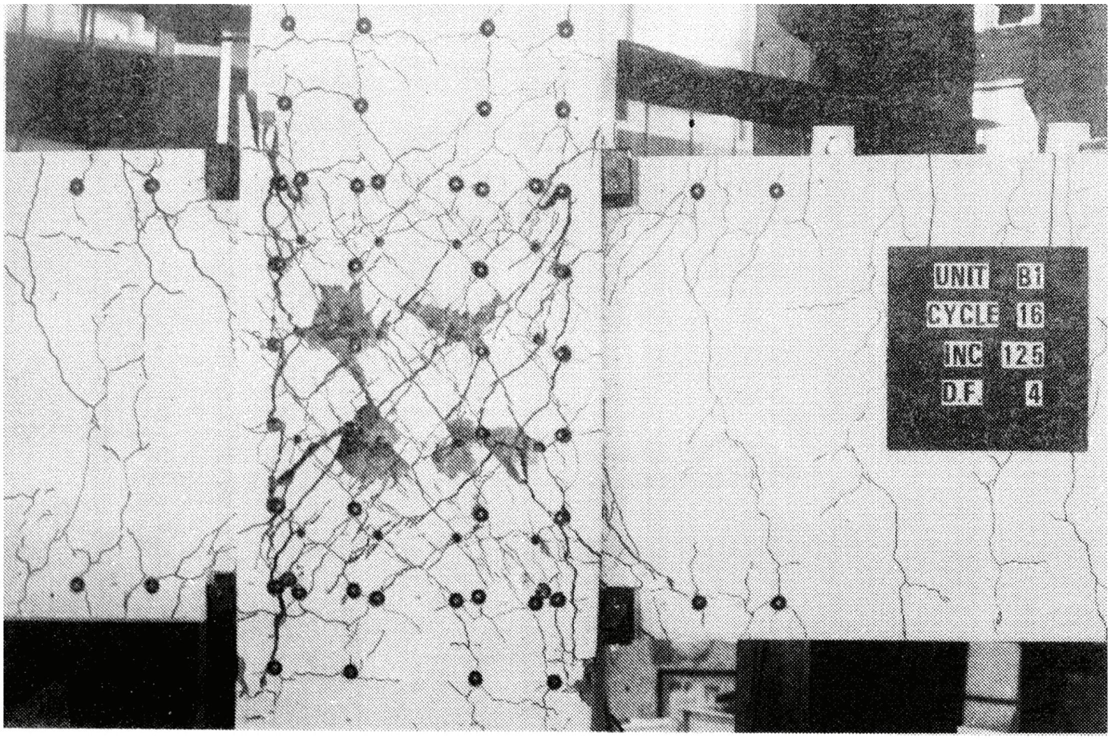


Figure 12: Intermediate moment frame (IMF) sub-assemblage following testing to 3.28% story drift. Joint cover concrete has spalled over a limited region without exposing column longitudinal reinforcements. Damage is representative of Damage State 2. Figure 5.3 from Birss et al. (1987).

3.4 Damage State 3: Removal and Replacement of Damaged Concrete

If spalling of cover concrete exposes longitudinal reinforcement or concrete damage extends to crushing of core concrete, it may be necessary to remove and recast beam and/or joint concrete. This damage states represents an expansion of the repair activities included in Damage State 2 and includes removal and replacement of damaged and potentially damaged concrete. In removing concrete, the objective is to ensure that only undamaged concrete remains as well as to ensure that a substantial volume of new material is placed around beam and/or column reinforcement to ensure that full bond capacity is recovered. Typically, the replacement material will be a standard concrete mix including sand and coarse aggregate. If more than 6 in. of concrete thickness is removed, mechanical anchorage devices, such as epoxy-embedded dowel bars, are recommended to ensure bond between new and existing concrete (ATC 1998, ACI 546R 1996). Examples of this Damage State are shown in Figure 13-16.

Calculation of loss associated with structural repair for Damage State 3 requires calculation of the length of concrete cracking to be epoxy injected as well as calculation of the volume of concrete to be removed and replaced. For all frames, the length of concrete crack to be injected may be estimated as for Damage State 2, with the exception that cracks within the volume of replaced concrete will not require epoxy injection For frames for which flexural or flexure-shear response of beams determines response or for which joint strength determine frame strength, images of frame subassemblage test specimens, such as that shown in Figures 13-15, suggest that volume of concrete to be replaced may be estimated as follows:

1) Beam volume extending along the beam length a distance equal to half the beam depth.

2) Joint volume.

3) No replacement of column (beam) concrete will be required.

4) If beam flexural strength determines frame strength, a slab volume equal to the entire slab depth, a distance outward away from the beam axis and into the frame bay equal to half the ACI column-strip width (ACI 318 2008), and a distance equal to half the beam depth.

For frames for which other mechanisms determine response, the volume of concrete to be replaced may be estimated in a similar manner as described above:

1) Where beam shear strength or non-compliant beam details determine frame strength, the volume of concrete to be replace includes the entire beam and joint volume as well as a slab volume equal to the entire slab depth, ACI column-strip width (ACI 318 2008), and a distance equal to beam clear span.

2) Where column flexural strength determines frame strength, the volume of concrete to be replaced includes the entire column volume extending a distance along the column equal to the column depth and the joint volume.

3) Where column flexure-shear or shear response or non-compliant column details determine frame strength, the volume of concrete to be replaced includes the entire column and joint volume.

Figure 13: Severe spalling of column cover concrete exposing longitudinal steel. Damage resulted from Izmit earthquake. Image by H. Sezen (1999), courtesy of National Information Service for Earthquake Engineering, EERC, UC Berkeley.

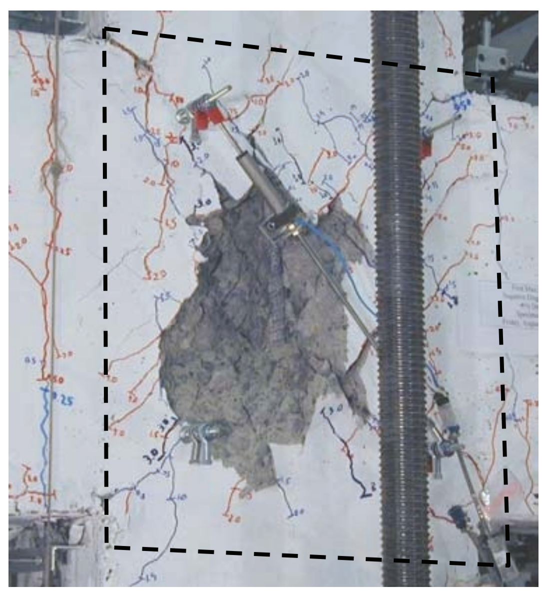


Figure 14: Laboratory testing of OMF sub-assemblage (see Figure 1). Here, concrete has spalled in the beam-column joint region (outlined by dashed line) exposing column longitudinal reinforcing steel (Lehman et al. 2002).

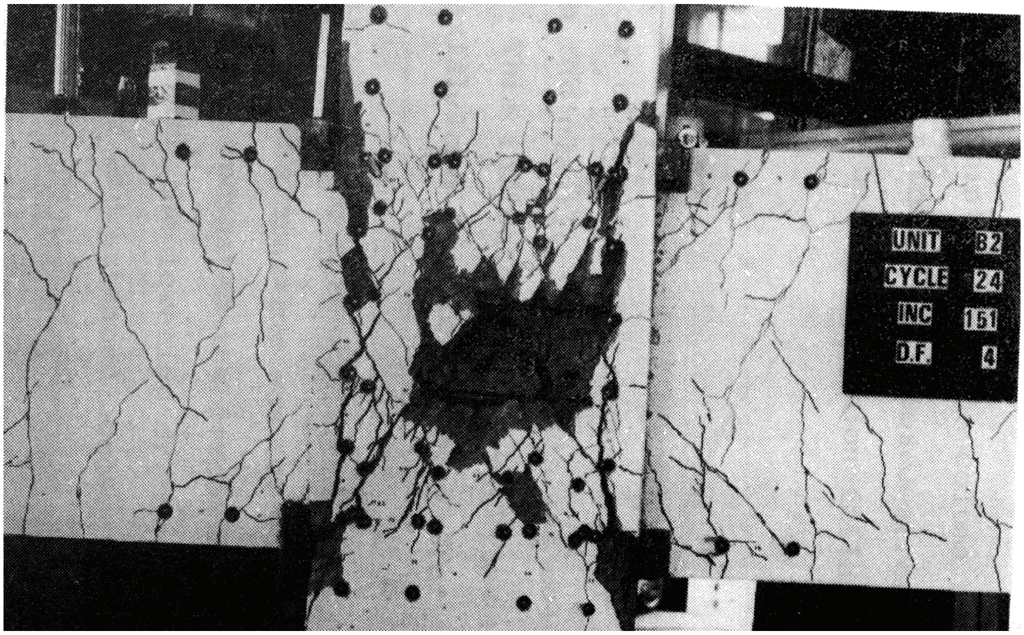
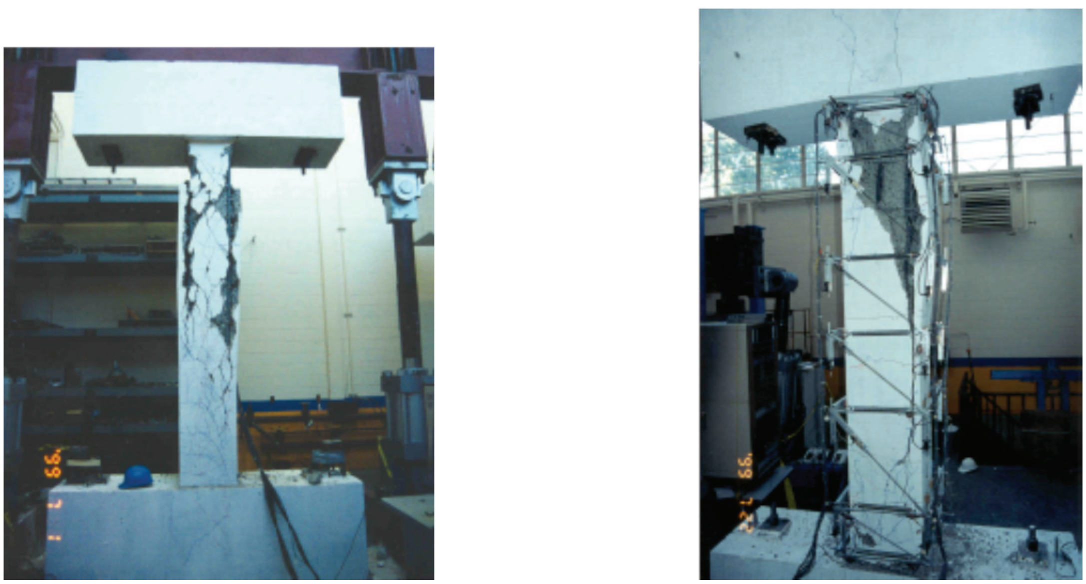


Figure 15: Intermediate moment frame (IMF) sub-assemblage following testing to 2.87% story drift. Crushing of joint concrete has exposed column longitudinal reinforcement. Damage is considered representative of Damage State 3. Figure 5.6 from Birss et al. 1987.



a) ASCE5 category column **b) ASCE6 category column**

Figure 16: Laboratory testing of ASCE 5 and ASCE 6 column sub-assemblages. Images show specimens following testing to strength loss (Sezen 2000).

3.5 Damage State 4: Removal and Replacement of Reinforcing steel

Under severe earthquake loading, beam longitudinal reinforcement may exhibit large plastic deformation, buckle, and fracture or column longitudinal steel may buckle in the joint region due to joint damage and column axial load. If this occurs, the reinforcement must be replaced.

Damage State 4 comprises the activities required to replace reinforcing steel and replace and repair surrounding concrete. These activities include shoring the structure, removing concrete using chipping or jack-hammering, removing the damaged sections of reinforcing steel, replacing the reinforcing steel, placing epoxy-embedded dowel bars as necessary, replacing beam and joint concrete, and epoxy injecting concrete cracks in surrounding regions. Typically, new and existing reinforcing steel are connected using a mechanical connection such as sleeve, splice, or threaded coupler (ATC 1996). Damage State 4 is comparable to Structural Repair 4 as defined by FEMA 308 (ATC 1996). Damage State 4 is required only if substantial yielding and/or buckling of beam or column reinforcement has occurred. Figure 17 and Figure 18 show damage representative of Damage State 4.

Calculation of loss associated with structural repair for Damage State 4 requires calculation of the volume of concrete to be removed and replaced, the number of steel reinforcing bars to be replaced, and the length of cracks in surrounding concrete to be epoxy injected. Economic loss associated with structural repair for Damage State 4 may be computed as the loss associated with structural repair for Damage State 3 plus the loss associated with replacing longitudinal reinforcing steel. Specifically, the number of reinforcing bars to be replaced is assumed equal to the number of beam longitudinal bars passing through the beam-column joint. It is expected that the cost associated with replacing beam longitudinal steel will be relatively small in comparison with the cost associated with shoring the structure to replace joint, beam and slab concrete as is required for Damage State 3.

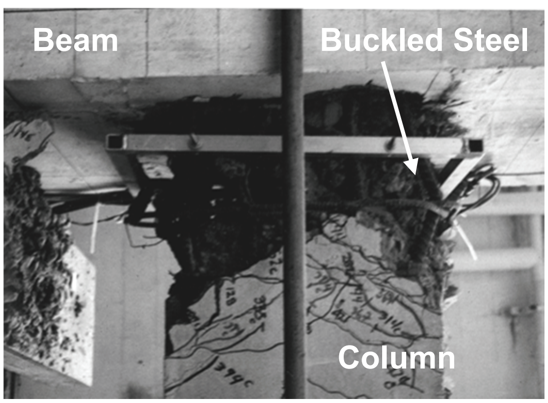


Figure 17: Laboratory testing of bridge frame sub-assemblage. Column shows initial flexural cracking. Cracks have been marked in black (Lowes 1995).

Figure 18: Laboratory testing of a bridge column. Column shows initial spalling that exposes transverse reinforcing steel. Image by D.E. Lehman 1996, courtesy of National Information Service for Earthquake Engineering, EERC, UC Berkeley.

4 Damage Data

Damage data for the 106 frame sub-assemblage and 35 frame-member test specimens were compiled and evaluated for use developing fragility functions. For frame sub-assemblage test specimens, damage-drift data provided by Pagni (2006) and Brown (2008) were augmented with data characterizing beam and frame damage, which were collected from original reports and papers. Brown (2008) provides damage-drift data for 14 *damage states*; these were augmented with 5 additional *damage states* to better characterize the progression of damage in frames. Table 5 lists these *damage states* and relates them to the five repair-specific damage states used in this study. If, for a particular test specimen, multiple damage-drift data points were available for any of the five repair-specific damage states, only the data point with the smallest drift value was used to generate fragility functions for this study. For column test specimens, damage data provided in the PEER Performance Database with augmented with yield data also provided in the Database. Table 5 lists the *damage states* employed in the PEER Performance Database and relates them to the five repair-specific damage states used in this study.

Table 5: Characterization of damage states

Damage States Used in This Study	Damage States Used by Brown (2008)	Frame Sub-assemblage Damage States Added for This Study	Damage States Used in the PEER Database
C. Cosmetic Repair	0. Initial hairline cracking in beam/column interface. 1. Initial hairline cracking in the joint. 2. Cracks in joint are measurable but widths are less than 0.02 in. (0.5mm).		
0. Epoxy Inject Concrete	3. Maximum crack width within the joint is greater than 0.02 in. (0.5 mm). 4. Joint transverse reinforcement yields.	• Beam or column longitudinal steel yields • Crack widths exceed 0.02 in. (0.5mm)	Longitudinal reinforcement yields
1. Epoxy Inject Concrete	5. Maximum crack width in the joint is greater than 0.05 in. (1.3 mm).	• Beam or column crack widths exceed 0.06 in. (1.5 mm)	
2. Patch Concrete	6. Initiation of beam bar slippage in the joint. 7. Spalling of at least 10% of the joint surface concrete.	• Beam or column cover concrete spalls	Initial spalling of cover concrete
3. Replace Concrete	0. 1. 2. 3. 4. 5. 6. 7. 8. Joint shear strength begins to deteriorate. 9. Spalling of more than 30% of the joint surface area. 10. Cracks with widths in excess of 0.05 in.(1.3 mm) extend into the beam and/or column. 11. Spalling of more than 80% of the joint	• Beam or column concrete spalls over a length greater than 10% of the height of the beam	Significant concrete spalling Loss of lateral strength

	surface area. 12. Crushing of concrete extends into joint core.		
4. Replace Reinforcin g Steel	13. Failure due to buckling of column longitudinal steel or loss of beam steel anchorage within the joint.	• Beam or column longitudinal steel buckles	Bar buckling

Table 6 lists means, medians and coefficients of variation for the different frame categories for data collected from frame sub-assemblage and cantilever column tests. Figure 19 shows damage versus story drift for the different data sets. These data show a number of trends of interest to the current study. With respect to damage prediction using frame sub-assemblage versus cantilever column data:

1. For Damage States 0-2 data from frame sub-assemblage tests and cantilever column tests indicate approximately the same relationship between story drift and damage.
2. For Damage State 3, 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%). The lower drift values for the column test data are attributed to the inability of the numerical model used to predict story drifts from column drifts to simulate increased frame flexibility due to joint and column damage at higher drift demands. It should be noted that for Damage State 3 cantilever column data are available only for SMF systems.
3. For Damage State 4, frame sub-assemblage data are not available. However, the median drift at which cantilever column data show on-set of Damage State 4 is not significantly larger than the median drift at which frame sub-assemblage data show the on-set of Damage State 3.

With respect to damage progression in the different frame categories:

1. For Damage States C-1 and all of the frame categories except the OMF-CYSH /ASCE6 frames, the median drift at which the Damage State initiates is approximately the same. For these frame categories, where there are discrepancies in median drift, they are not consistent for all damage states and are often associated with small data sets. Specimens in frame category OMF-CYSH/ASCE6 exhibit onset of Damage States C and 0 at significantly lower median drifts than do specimens in the other frame categories.
2. For Damage State 1, these are very few data, but these data suggest the onset of the damage state occurs at approximately the same drift level for all frames.
3. For NCF/ASCE3 and OMF-CYSH/ASCE6 specimens, the median drift at onset of Damage State 2 is larger than that for onset of Damage State 3. For these specimens, severe cracking results in loss of lateral strength and onset of Damage State 3 prior to significant spalling. Thus, for these frame categories, Damage State 2 is considered irrelevant and fragilities are not developed for this damage state.
4. For Damage State 2, the median drift at onset of the damage state gradually decreases from 2.8% to 1.6% as the detailing and design requirements become less stringent. Exceptions to this are frame categories NCF/ASCE3 and OMF-CYSH/ASCE6, which are discussed above.

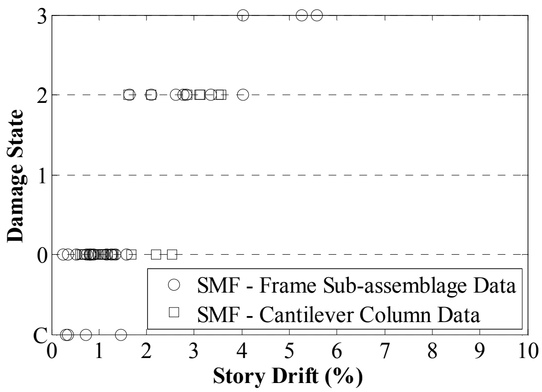
5. For Damage State 3, the median drift at onset of the damage state decreases more rapidly from 5.25% to 0.71% as the detailing and design requirements become less stringent.

Coefficients of variation (c.o.v.) for the different data sets are consistent with previous studies and exhibit the following characteristics.

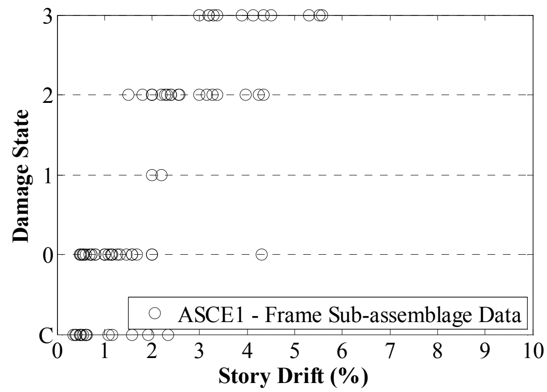
1. For small data sets, c.o.v.s vary widely, and for all of the data sets, c.o.v.s range from less than 1% to 153%.
2. For the larger data sets, the range of c.o.v.s is reduced to 20%-80%.
3. For the larger data sets and damage states not associated with initial cracking, for which drift an onset could be expected to be highly dependent on specimen gross dimensions, c.o.v.s range from 20% to 30%.

Table 6: Damage State Statistics for Different Data Sets

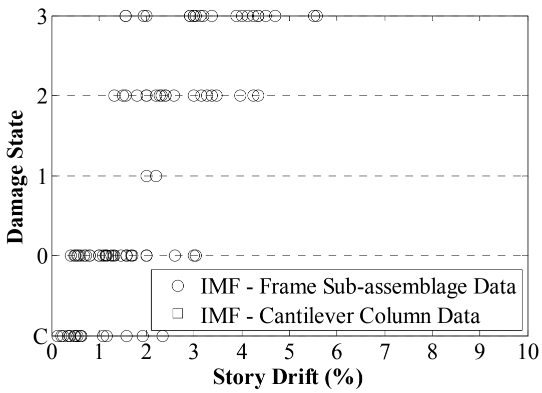
		Frame Sub-assemblage Data						Cantilever Column Data						Combined Data		
		SMF	ASCE 1	IMF	ASCE 2	OMF-BYJS	NCF / ASCE3	SMF	IMF	OMF-BYS / ASCE4	OMF-CYSM / ASCE5	OMF-CYSH / ASCE6	NCF / ASCE3	SMF	IMF	NCF / ASCE3
Damage State C	no. of specimens	6	18	22	10	9	17			1	4	10	1			18
	mean drift (%)	0.71	0.85	0.73	0.35	0.42	0.67			0.19	0.50	0.19	0.59			0.66
	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
	mean drift (%)	0.83	1.20	1.37	1.58	1.19	0.84	1.11	1.30	1.15	1.34	0.40	0.59	1.02	1.36	0.79
	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					
	mean drift (%)		2.10	2.07	1.83	1.75	2.00				3.02					
	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		
	mean drift (%)	2.76	2.65	2.61	1.97	2.15	2.90	2.84		2.11	1.87	1.37		2.80		
	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
	mean drift (%)	5.02	4.07	3.59	2.79	3.18	2.34	3.51		2.61	2.41	0.74	1.66	4.18		2.27
	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
Damage State 4	no. of specimens					1	7	5								
	mean drift (%)					5.00	3.25	4.18								
	median drift (%)					5.00	3.25	4.08								
	coeff. of var. (%)						0.19	0.25								



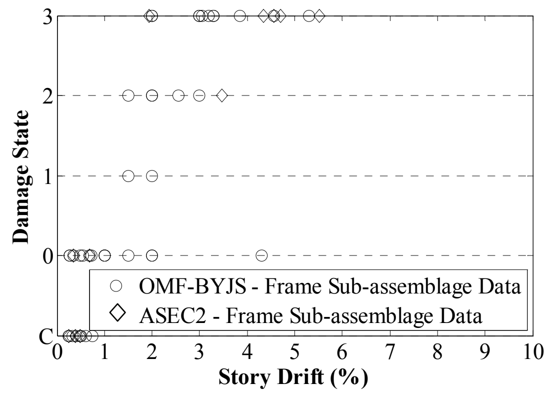
a) SMF data



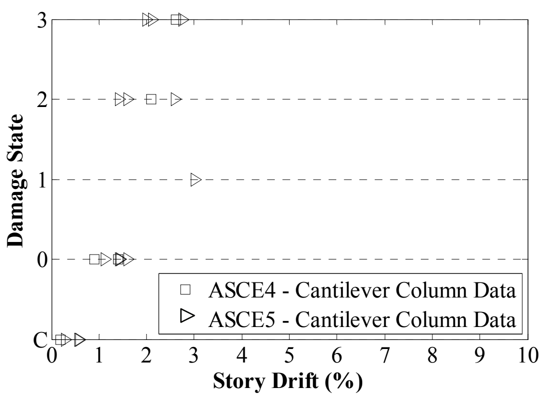
b) ASCE1 data



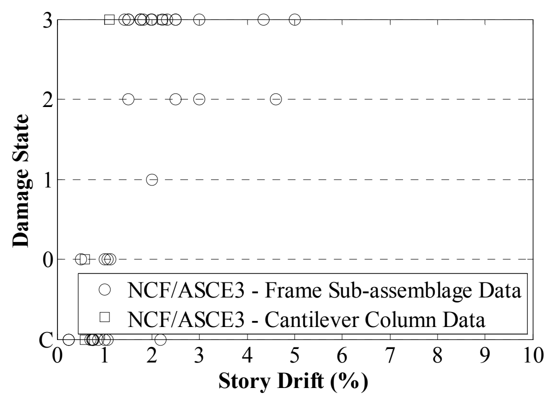
c) IMF data



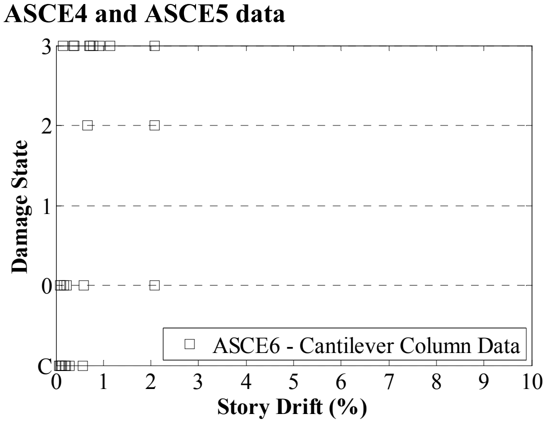
d) ASCE2 and OMF-BYJS data



e) ASCE4 and ASCE5 data



f) NCF / ASCE3 data



g) ASCE6 data

Figure 19: Damage State versus drift for different data sets.

5 Fragility Functions

The data in Table 6 were used as a basis for determining parameters for suites of fragility functions for different categories of concrete frames. First, the data in Table 6 and damage state descriptions were reviewed to determine the most appropriate data sets for use in developing fragility functions as well as the damage states for which fragilities were most critical. Second, the Method of Maximum Likelihood was used to fit cumulative probability functions to the final data sets, assuming the data were lognormally distributed. Third, Lilliefors goodness of fit testing was done to evaluate the assumption of the lognormal distribution. Finally, the computed distribution parameters were adjusted to facilitate application in practice and to account for uncertainty associated with the size of the data sets. The following paragraphs discuss these steps in more detail.

First the data in Table 6 and damage state descriptions were reviewed to determine the most appropriate data sets for use in developing suites of fragilities. Given the large number of frame categories and that frame categories were established using ACI Code and ASCE/SEI Standard 41-06 criteria, it was expected that the same suite of fragility functions could be used for more than one frame category. Also, it was expected that for some frame categories, the relatively small size of the data set would not justify development of a unique suite of fragilities. Finally, as noted previously, review of the data suggested some data sets were not appropriate for use in developing the fragility functions. Ultimately, fragilities were developed for the following frame categories using the following data sets:

1. ***SMF.*** A unique suite of fragility functions was developed for SMFs as specimens meeting these criteria exhibited significantly better performance than other frame categories. For Damage States 0-2, data from both frame and cantilever column tests were used, when available. For Damage State 3, the median drift at onset of the damage state was significantly lower for cantilever column tests than frame sub-assemblage tests. This was attributed to modeling error in predicting frame drift from cantilever column drift; specifically the assumption that columns and joints remain elastic even at large drift levels. Thus, the fragility function for Damage State 3 was developed using only frame sub-assemblage test data.

2. ***ASCE1.*** A unique suite of fragility functions was developed for ASCE1 frames. These frames could potentially be grouped with IMF. Approximately the same specimens met the requirements for both categories. The behavior of most specimens in both categories was approximately the same: beam yielding with strength loss initiating at moderate ductility levels due to joint damage (in some cases strength loss was not significant and in some cases joint strength was not sufficient to develop the flexural strength of the beams). A review of the data in Table 6 and Figure 19 shows similar statistics for frames in these categories. However, the median drift at onset of Damage State 3 is 4.2% for ASCE1 frames and 3.6% for IMFs; this was deemed significant enough to warrant investigation of unique suites for the two categories of frames.

3. ***IMF.*** A unique suit of fragility functions was developed for IMFs as discussed above.

4. ***ASCE2 and OMF-BYJS***. A single suite of fragility functions was developed to predict damage for frames meeting either the ASCE2 or OMF-BYJS category requirements. Approximately the same specimens met the requirements for both categories. Specimens in these categories exhibited approximately the same behavior: joint failure prior to beam yielding or beam yielding with strength loss at moderate ductility levels due to joint damage. The median drift at the onset of Damage States 1-3 for these two categories of frames did not differ by more than 13%. The behavior of specimens in both categories was approximately the same, and the statistics for both categories were approximately the same. For Damage States C and 0, median drifts differed significantly; however, as these damage states were not of primary concerned this was ignored. Only frame sub-assemblage data were available and used for these categories.

5. ***OMF-BYS/ASCE4 and OMF-CYSM/ASCE5***. A single suite of fragility functions was developed to predict damage for frames meeting either the OMF-BYS/ASCE4 or OMF-CYSM/ASCE5 category requirements. Specimens in these categories exhibited flexure-shear or shear failures in beams or columns; because column axial loads were low to moderate, damage progression was similar for both categories of frames. For these two categories and Damage States 0-3, the median drift at the onset of the damage state differed by as much as 30%. However, the size of the data sets was small for these frame categories and there was no consistent trend in the data. Thus, a single suite of fragilities was considered to be appropriate.

   For these frame categories, fragilities were developed using frame drifts predicted from cantilever column tests. Use of column test data was considered reasonable for these categories of frame because a) expected damage would be concentrated in beams or columns for these categories of frames and b) median drifts at the onset of Damage State 3 were relatively small (2.5%). Thus, model errors (resulting from the assumption of elastic response in joints and columns or beams) could be expected to be relatively small.

6. ***NCF/ASCE3***. A unique suite of fragility functions was developed to predict damage for frames with non-compliant details. Non-compliant details included column longitudinal steel spliced above the joint, in some cases with inadequate splice detailing; beam top and bottom longitudinal reinforcement not continuous through the joint; ACI Code-specified minimum volume of transverse reinforcement was not provided in beams and/or columns. Non-compliant detailing typically contributed to reduced drift capacity. For this category of frames, fragilities were developed primarily using frame sub-assemblage data. A few data points from column tests were available for Damage States 0 and 3. These data were consistent with the frame data and were used in developing fragilities.

7. ***OMF-CYSH / ASCE6***. A unique suite of fragility functions was developed to predict damage for frames in this category. Specimens in this category, with inadequate column shear strength and high column axial loads, exhibited extremely limited drift capacity and extremely rapid damage progression. Only cantilever column test data were available for use in developing fragilities. However, as damage would be expected to concentrate in columns and the median drift at the onset of Damage State 3 was very small (0.71%), this was considered reasonable.

Table 7 lists the lognormal distribution parameters computed using the above data sets. It was assumed that the data were lognormal distributed such that

$$P[DM \geq dm|DP = x] = \Phi\left(\frac{ln(x/\theta)}{\beta_r}\right)$$

where 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 to determine θ and β_r to provide a best fit to the data. The computed θ and β_r values are approximately, but not exactly, equal to the median and c.o.v. for each data set.

Table 7: Computed and assigned fragility function parameters

	Damage State C		Damage State 0		Damage State 1				Damage State 2				Damage State 3			
	θ	β_r	θ	β_r	θ	β_r	β_u	β	θ	β_r	β_u	β	θ	β_r	β_u	β
SMF	0.62	0.58	0.93	0.45	No Data		0.40	0.40	2.71	0.28	0.10	0.30	4.98	0.15	0.25	0.30
ASCE 1	0.71	0.58	1.04	0.52	2.10	0.07	0.40	0.40	2.54	0.30	0.10	0.30	3.99	0.20	0.25	0.30
IMF	0.56	0.74	1.22	0.50	2.06	0.06	0.40	0.40	2.46	0.36		0.30	3.40	0.35	0.10	0.40
ASCE 2 / OMF-BYJS	0.38	0.38	0.82	0.79	1.73	0.20	0.30	0.40	2.24	0.27	0.13	0.30	3.22	0.32	0.15	0.40
ASCE4 / ASCE 5	0.40	0.51	1.27	0.20	No Data		0.40	0.40	1.88	0.27	0.12	0.30	2.43	0.15	0.35	0.40
NCF / ASCE3	0.53	0.68	0.75	0.39	No Data		0.40	0.40	2.68	0.46			2.12	0.37	0.10	0.40
ASCE 6	0.16	0.59	0.23	0.95	No Data		0.40	0.40	1.18	0.79			0.55	0.86		0.50

Table 8 lists the results of Lilliefors "goodness of fit" testing. In Table 8, "PASS" indicates that the hypothesis that the data are lognormally distributed cannot be rejected; while "FAIL" indicates that it can be rejected. No result is provided where the data set included four (4) or fewer elements and was too, therefore, too small to test. Note that the θ and β_r values listed in Table 7 are tested. Lilliefors testing suggests that approximately half of the frame data sets are lognormally distributed. However, as use of the lognormal distribution for definition of fragilities is the standard, evaluation of other distributions was not considered.

Table 8: Lilliefors "goodness of fit" test results.

Frame Category	Lilliefors "goodness of fit" test result (5% significance level)				
	DS C	DS 0	DS 1	DS 2	DS 3
SMF	PASS	PASS		PASS	
ASCE 1	FAIL	FAIL		FAIL	PASS
IMF	FAIL	PASS		PASS	PASS
ASCE 2 / OMF-BYJS	PASS	FAIL		FAIL	FAIL
ASCE4 / ASCE 5	FAIL	PASS		PASS	PASS
NCF / ASCE3	PASS	PASS			FAIL
ASCE 6	PASS	FAIL			PASS

Table 9: Proposed fragility function parameters for performance-based seismic design and evaluation of RC frames

	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

Table 9 lists lognormal distribution parameters proposed for use in performance-based seismic design and evaluation of concrete frames. For Damage States 2 and 3, the θ values in Table 9 are the computed θ values listed in Table 7, rounded to the nearest 0.25% to facilitate use. For Damage State 1, θ values were chosen to a) be consistent with the few data points available for this Damage State and b) fall between median drifts associated with Damage States 0 and 2. In all but two cases, the β values listed in Table 9 are defined as

$$\beta = \sqrt{{\beta_r}^2 + {\beta_u}^2},$$

where β_r was computed using the data and the Method of Maximum Likelihood and β_u was assigned 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". As shown in Table 7, β_u ranges from 0.1 to 0.4, with larger values assigned where data sets are small and to maintain consistent trends in dispersion across damage states and frame categories. It should be noted that in two cases (highlighted in grey in Table 7) β values less than the computed β_r value were assigned to provide consistency with the remaining data sets. Figure 18 - 25 show empirical fragilities and smoothed fragilities that employ the θ and β values listed in Table 9.

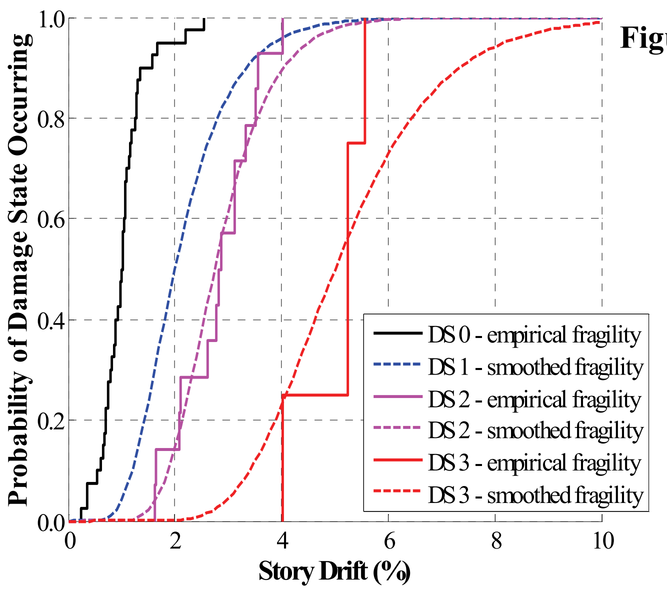


Figure 20: Fragility functions for SMF

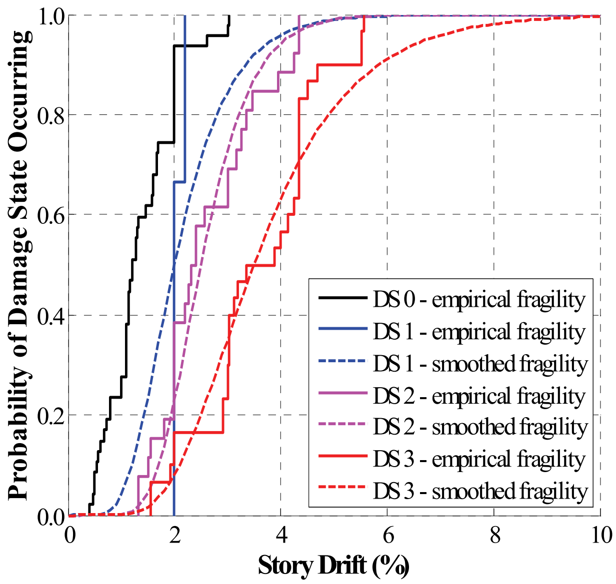


Figure 21: Fragility functions for IMF

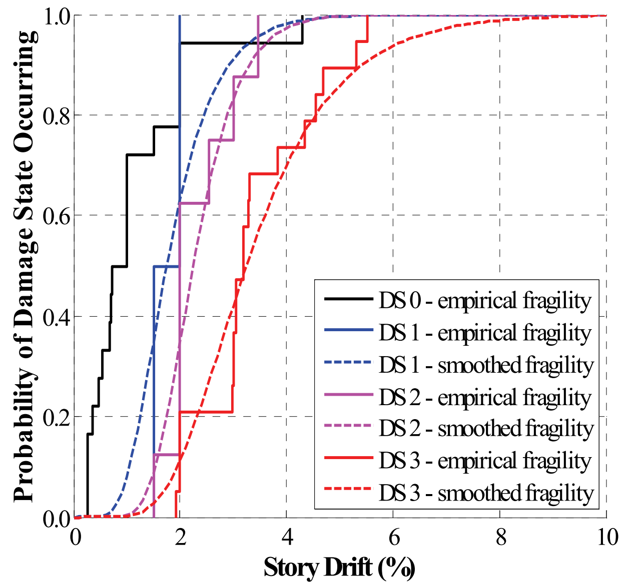


Figure 23: Fragility functions for OMF-BYJS / ASCE2

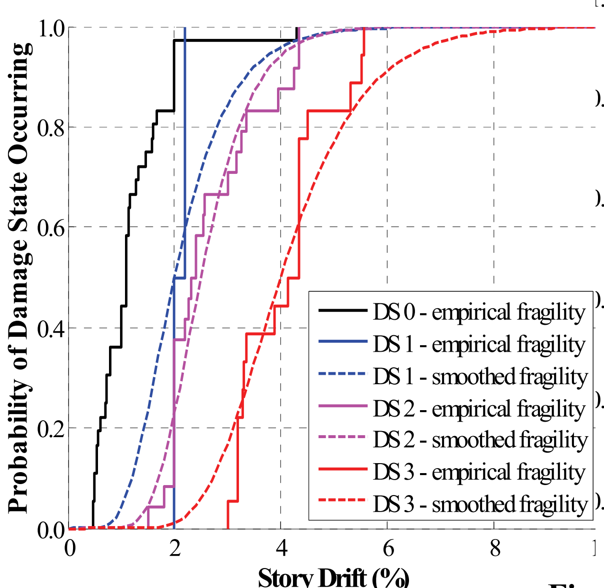


Figure 22: Fragility functions for ASCE

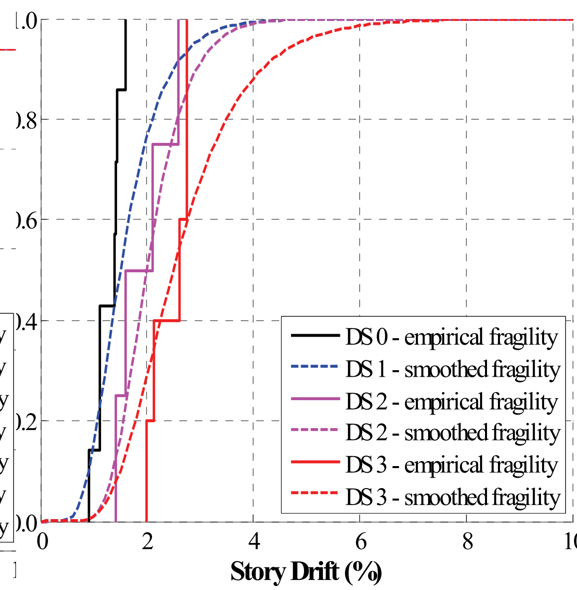


Figure 24: Fragility functions for ASCE4/ ASCE5

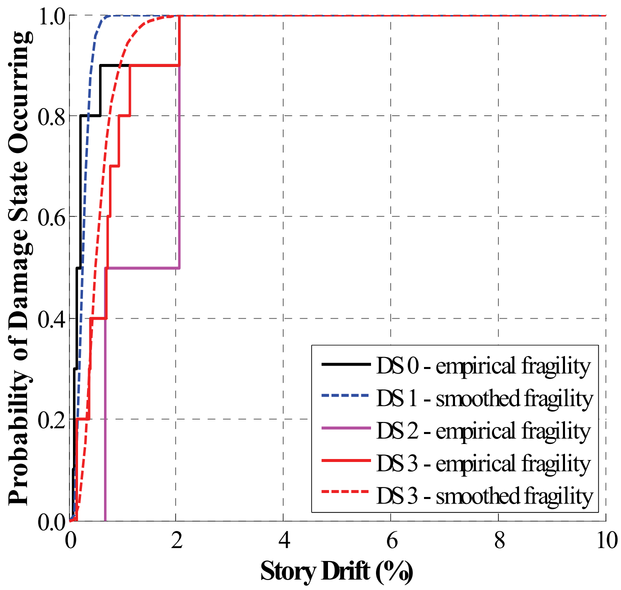


Figure 25: Fragility functions for ASCE6

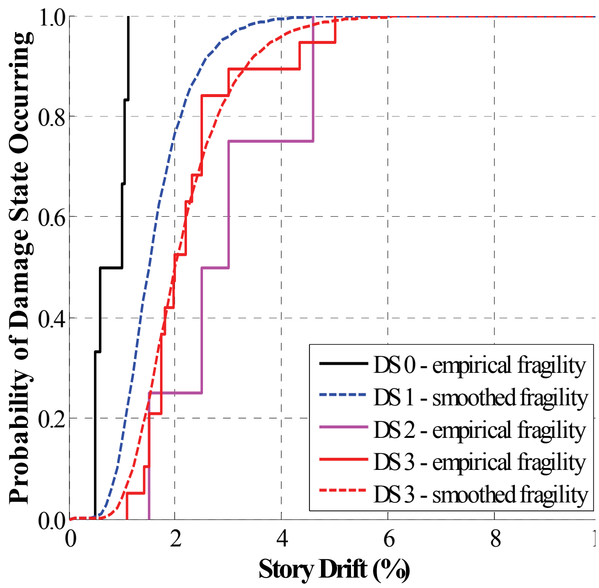


Figure 26: Fragility functions for NCF/ASCE3

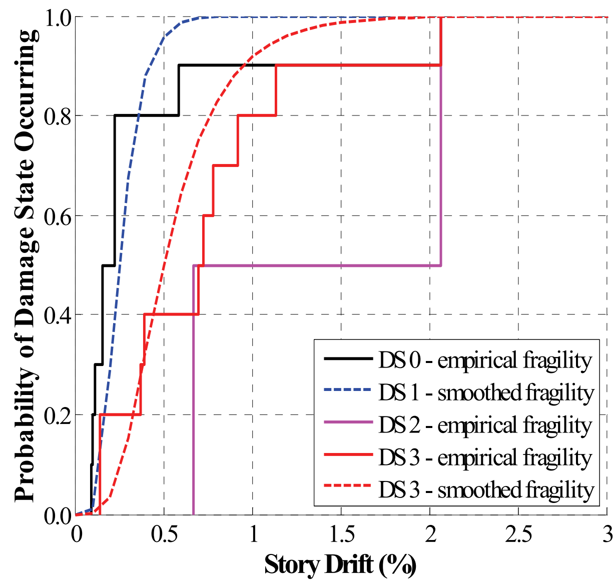


Figure 27: Fragility functions for ASCE6 (reduced drift range)

6 Summary

Fragility functions were developed for eight categories of reinforced concrete moment frames, where categories were defined on the basis of ACI Code requirements as well as ASCE/SEI Standard 41-06, Supplement 1, Chapter 6 specifications. These fragilities were developed using data from experimental testing of frame and column sub-assemblages. Fragiity functions define the likelihood that a frame will exhibit a specific level of damage and require a specific typ of repair to restore the frame to pre-earthquake (essentially undamaged) conditions. Fragility functions were developed for three damage states considered to best represent the range of damage and subsequent repair activities that determine economic losses in RC frames subjected to earthquake loading. These damage states are Damage State 1 – Epoxy Inject Concrete, Damage State 2 – Patch Concrete, and Damage State 3 – Replace Concrete.

The collected damage-drift data and fragility function parameters computed from these data follow expected trends with most frame systems exhibiting similar performance at low drift levels, while the onset of more severe damage is predicted to occur at higher median drifts in frames with ductile detailing and complying with more stringent design requirements. For Damage States 2 and 3, the median drifts associated with onset of damage were computed directly from the experimental data and the empirical dispersion was increased to account for uncertainty in system response, the limited size of the data set and to produce consistent dispersions across damage states and frame categories. For Damage State 1, the median drift associated with onset of damage was determined based on a very few data points and understanding of system response. For Damage State 1, dispersion was chosen equal to the maximum dispersion proposed for Damage State 2 to account for the lack of data.

One-page summaries of the fragility functions are included in Appendix B.

7 References

ACI Committee 224 (1994). "Control of Cracking in Concrete Structures" *ACI Manual of Concrete Practice*, Detroit, Michigan: American Concrete Institute.

ACI Committee 318 (2005). "Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318-R05)." Farmington Hills, MI: American Concrete Institute.

ACI-ASCE Joint Committee 352 (2002). "Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures (ACI 352-R02)." Farmington Hills, MI: American Concrete Institute.

Alire, D. 2002. "Seismic Evaluation of Existing Unconfined Reinforced Concrete Beam-Column Joints." *M.S. Thesis.* Seattle: University of Washington.

ASCE/SEI 41, 2007. *Seismic Rehabilitation of Existing Buildings*, American Society of Civil Engineers, Reston, VA.

Applied Technology Council (1998). *FEMA 308: Repair of Earthquake Damaged Concrete and Masonry Wall Buildings*. Washington D.C.: FEMA.

Attaala, S. A. and Agbabian, M. S. (2004). "Performance of Interior Beam-Column Joints Cast from High Strength Concrete under Seismic Loads." *Advances in Structural Engineering.* 7(2):147-157.

Barlow, P. (2008). Phone conversation with Pete Barlow, Concrete Tech, Seattle, WA.

Beckingsale, C. W., R. Park and T. Paulay (1980). "Post Elastic Behavior of Reinforced Concrete Beam-Column Joints." Ph.D. Dissertation. New Zealand: University of Canterbury.

Berry, M.; Parish, M.; and Eberhard, M. (2004). "PEER Structural Performance Database User's Manual." *PEER Research Report.* Berkeley, CA: University of California. (http://nisee.berkeley.edu/spd).

Birss, G.R., T. Paulay and R. Park (1978). "The Elastic Behavior of Earthquake Resistant Reinforced Concrete Interior Beam-Column Joints." *M.S. Thesis*. New Zealand: University of Canterbury.

Brown, P.C. (2008). "Probabilistic Earthquake Damage Prediction for Reinforced Concrete Building Components." *M.S. Thesis*. Seattle: University of Washington.

Brown, P. and L.N. Lowes (2007). "Fragility Functions for Modern Reinforced Concrete Beam-Column Joints." *Earthquake Spectra*. 23(2): 263-289.

Dhakala, R., Pan, T., Irawan, P. et al. (2005). "Experimental study on the dynamic response of gravity-designed reinforced concrete connections." *Engineering Structures*. 27: 75–87.

Durrani, A. and J. Wight (1982). "Experimental and Analytical Study of Internal Beam to Column Connections Subjected to Reversed Cyclic Loading." *Report UMEE 82R3*.

Endoh, Y., T. Kamura, S. Otani and H. Aoyama (1991). "Behavior of R/C Beam-Column Connections Using Light Weight Concrete." *Transaction of the Japan Concrete Institute* (13): 319-326.

Fujii, S. and Morita,. S. (1991). "Comparison between interior and exterior RC beam-Column joint behavior". *ACI SP 123Design of Beam-Column Joints for Seismic Resistance.* Ed. J. Jirsa. Framington Hills: American Concrete Institute: 145-165.

Hamburger et al. (2008), "ATC-58-1, Review Panel Meeting Minutes" December 15, 2008.

Hayashi K., M. Teraoka, A.A. Mollick and Y. Kanoh (1993). "Bond Properties of Main Reinforcing Bars and Restoring Force Characteristics in RC Interior Beam-Column Sub-assemblages Using High Strength Materials." *Proceeding of the Japan Concrete Institute* 15 (2): 583-588.

Higashi, Y., and Ohwada, Y. (1969). "Failing behaviors of reinforced concrete beam-column connections subjected to lateral load". *Memories of Faculty of Technology Tokyo Metropolitan University*. Tokyo, Japan: Tokyo Metropolitan University. 91-101

Joh, O., Y. Goto and T. Shibata (1991a). "Influence of Transverse Joint and Beam Reinforcement and Relocation of Plastic Hinge Region on Beam-Column Joint Stiffness Deterioration." *ACI SP-123: Design of Beam-Column Joints for Seismic Resistance*. Ed. J. Jirsa. Framington Hills: ACI: 187-223.

Joh, O., Y. Goto and T. Shibata (1991b). "Behavior of Reinforced Concrete Beam-Column Joints with Eccentricity." *ACI SP-123: Design of Beam-Column Joints for Seismic Resistance*. Ed. J. Jirsa. Framington Hills: ACI: 317-357.

Kanda, M., Shirai, N., Adachi, H., and Sato, T. (1988). "Analytical Study on Elasto-Plastic Hysteretic Behaviors of Reinforced Concrete Members." *Transactions of the Japan Concrete Institute,* 10.

Kitayama, K., S. Otani and H. Aoyama (1991). "Development of Design Criteria for RC Interior Beam-Column Joints." *ACI SP-123: Design of Beam-Column Joints for Seismic Resistance*: 96-123.

Lehman, D. E., Gookin, S. E., Nacamuli, A. M. & Moehle, J. P. (2001). “Repair of Earthquake-Damaged Bridge Columns.” *ACI Structural Journal*, 98(2): 233-242.

Lynn, A., Moehle, J.P., Mahin, S.A., and Holmes, W.T. (1996). “Seismic Evaluation of Existing Reinforced Concrete Building Columns “ *Earthquake Spectra* 12(4): 715-735

Meinheit, D. F. and Jirsa, J. O. (1977). “The Shear Strength of Rinforced Concrete Beam-column Joints.” *Technical Report CESRL Report No. 77-1*. University of Texas at Austin.

Milburn, J.R. and R. Park (1982). “Behavior of Reinforced Concrete Beam-Column Joints Designed to NZS 3101.” *M.S. Thesis*. New Zealand: University of Canterbury.

Mo, Y.L., and Wang, S.J. (2000). “Seismic Behavior of RC Columns with Various Tie Configurations.” *J. Struct. Eng.*, 126(10), 1122-1130.

Noguchi, H. and Kashiwazaki, T. (1992). “Experimental Studies on Shear Performances of RC Interior Column-beam Joints.” *Proceedings of the Tenth World Conference on Earthquake Engineering, Madrid, Spain*. 3163-3168

Oka, K. and Shiohara, H. (1992). “Tests On High-strength Concrete Interior Beam-column Joint Sub-assemblages.” *Proceedings of the Tenth World Conference on Earthquake Engineering*, *Madrid, Spain*. 3163-3168.

Otani, S., Y. Kobayashi and H. Aoyama (1984). “Reinforced Concrete Interior Beam-Column Joints Under Simulated Earthquake Loading.” *Report 113,* Japan: University of Tokyo Bunkyo-Ku.

Pagni, C.A. and L.N. Lowes (2006). “Fragility Functions for Older Reinforced Concrete Beam-Column Joints.” *Earthquake Spectra*. 22(1): 215-238.

Park, R., and Paulay, T. (1990). “Use of Interlocking Spirals for Transverse Reinforcement in Bridge Columns.” *Strength and Ductility of Concrete Substructures of Bridges, RRU (Road Research Unit) Bulletin 84*, 1, 77-92.

Park, R. and D. Ruitong (1988). “A Comparison of the Behavior of Reinforced Concrete Beam-Column Joints Designed for Ductility and Limited Ductility.” *Bulletin of the New-Zealand National Society for Earthquake Engineering* 21 (4): 255-278.

Pessiki, S. P. and Conley, C. ect (1990). “Seismic Behavior of Lightly Reinforced Concrete Column and Beam-column joint details.” *Technical Report NCEER-90-0014*. Buffalo, New York: National Center for Earthquake Engineering Research, State University of New York at Buffalo.

Pujol, S. (2002). "Drift Capacity of Reinforced Concrete Columns Subjected to Displacement Reversals." *Ph.D. Thesis*. Purdue University.

Sezen, H. (2000). “Seismic Behavior and Modeling of Reinforced Concrete Building Columns” *Ph.D. Thesis*. Berkeley: University of California.

Shiohara,H., and Kusuhara, F. (2006). “Benchmark Test for Validation of Mathematical Models for Non-linear and Cyclic Behavior of R/C Beam-Column Joints,” Report. Tokyo: The University of Tokyo.

Soesianawati, M.T., Park, R., and Priestley, M.J.N. (1986). “Limited Ductility Design of Reinforced Concrete Columns.” *Report 86-10*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Supaviriyakit, T. and Pimanmas, A. (2008). “Comparative Performance Of Sub-standard Interior Reinforced Concrete Beam–column Connection with Various Joint Reinforcing Details.” *Materials and Structures.* (41):543–557

Tanaka, H., and Park, R. (1990). “Effect of Lateral Confining Reinforcement on the Ductile Behavior of Reinforced Concrete Columns.” *Report 90-2*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Teraoka, M., Y. Kanoh, K. Tanaka and K. Hayashi (1990). “Strength and Ductility Behavior of RC Interior Beam-Column Joint Using High Strength Concrete.” *Proceeding of the Japan Concrete Institute* 12 (2): 633-638.

Teraoka, M., Y. Kanoh, K. Hayashi and S. Sasaki (1997). “Behavior of Beam-and-Column Sub-assemblages in an RC Frame.” *Proceedings of the First International Conference on High Strength Concrete July 13-18, Kona Hawaii*.

Thomsen, J., and Wallace, J. (1994). “Lateral Load Behavior of Reinforced Concrete Columns Constructed Using High-Strength Materials.” *ACI Struct. J.*, 91(5), 605-615.

Walker, S. (2001). “Seismic Performance of Existing RC Beam-Column Joints.” *MS Thesis.* Seattle: University of Washington.

Wehbe, N., and Saiidi, M.S., and Sanders, D. (1998). “Confinement of Rectangular Bridge Columns for Modrate Seismic Areas.” *National Center for Earthquake Engineering Research (NCEER) Bulletin*, 12(1).

Xin, X. (1992). “Behavior of Reinforced Concrete Interior Beam-Column Joints Designed Using High Strength Concrete and Steel.” *MS thesis*. University of Canterbury, New Zealand

Zaid, S.S.S., S. Otani and H. Shiohara (2001). “Behavior of Reinforced Concrete Beam-Column Connections Under Earthquake Loading.” *Ph.D. Dissertation.* Japan: University of Tokyo.

8 Appendix A – Experimental Data

Table 10: Geometric properties for frame sub-assemblage test specimens

Frame Sub-assemblage Tests		Frame Class	Geometric properties						
			Scale - assuming full scale is column depth > 12 in.	Beam length [in]	Beam width [in]	Beam depth [in]	Column Length [in]	Column width [in]	Column depth [in]
Shiohara et al. (2006)	A1	IMF	0.98	88.6	11.8	11.8	42.1	11.8	11.8
Beckingsale et al. (1980)	B11	SMF	1.00	203.9	14.0	24.0	135.0	18.0	18.0
Beckingsale et al. (1980)	B12	SMF	1.00	203.9	14.0	24.0	135.0	18.0	18.0
Beckingsale et al. (1980)	B13	SMF	1.00	203.9	14.0	24.0	135.0	18.0	18.0
Birss et al. (1978)	B1	SMF	1.00	192.0	14.0	24.0	135.0	18.0	18.0
Birss et al. (1978)	B2	IMF	1.00	192.0	14.0	24.0	135.0	18.0	18.0
Dhakala et al. (2007)	DPI-C1	NC	1.00	212.6	11.8	21.7	145.7	13.8	19.7
Dhakala et al. (2007)	DPI-C4	NC	1.00	212.6	11.8	21.7	145.7	15.7	15.7
Durrani & Wight (1982)	X1	IMF	1.00	98.3	11.0	16.5	88.5	14.3	14.3
Durrani & Wight (1982)	X2	SMF	1.00	98.3	11.0	16.5	88.5	14.3	14.3
Durrani & Wight (1982)	X3	IMF	1.00	98.3	11.0	16.5	88.5	14.3	14.3
Endoh et al. (1991)	A1	OMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Endoh et al. (1991)	HC	IMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Fujii & Morita (1991)	A1	OMF	0.72	89.0	6.3	9.8	59.1	8.7	8.7
Fujii & Morita (1991)	A2	OMF	0.72	89.0	6.3	9.8	59.1	8.7	8.7
Fujii & Morita (1991)	A3	OMF	0.72	89.0	6.3	9.8	59.1	8.7	8.7
Fujii & Morita (1991)	A4	OMF	0.72	89.0	6.3	9.8	59.1	8.7	8.7
Hayashi et al. (1993)	NO47	OMF	1.00	118.1	11.8	15.7	78.7	15.7	15.7
Joh et al. (1991)	B1	OMF	0.98	118.1	5.9	13.8	68.9	11.8	11.8
Joh et al. (1991)	B2	OMF	0.98	118.1	7.9	13.8	68.9	11.8	11.8
Joh et al. (1991)	B8-HH	SMF	0.98	118.1	7.9	13.8	68.9	11.8	11.8
Joh et al. (1991)	B8-HL	OMF	0.98	118.1	7.9	13.8	68.9	11.8	11.8
Joh et al. (1991)	B8-LH	IMF	0.98	118.1	7.9	13.8	68.9	11.8	11.8
Joh et al. (1991)	B8-MH	IMF	0.98	118.1	7.9	13.8	68.9	11.8	11.8
Kitayama et al.(1991)	KOAC1	IMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Kitayama et al.(1991)	KOAJ1	IMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Milburn & Park (1982)	Unit 1	SMF	1.00	226.0	9.0	18.0	126.7	12.0	16.0
Milburn & Park (1982)	Unit 2	SMF	1.00	226.0	9.0	18.0	126.7	12.0	16.0
Meinheit & Jirsa (1981)	MJ1	IMF	1.00	192.2	11.0	18.0	144.2	13.0	18.0
Meinheit & Jirsa (1981)	MJ12	IMF	1.00	192.2	11.0	18.0	144.2	13.0	18.0
Meinheit & Jirsa (1981)	MJ13	IMF	1.00	192.2	11.0	18.0	144.2	13.0	18.0
Meinheit & Jirsa (1981)	MJ2	IMF	1.00	192.2	11.0	18.0	144.2	13.0	18.0
Meinheit & Jirsa (1981)	MJ3	IMF	1.00	192.2	11.0	18.0	144.2	13.0	18.0
Meinheit & Jirsa (1981)	MJ5	IMF	1.00	192.2	11.0	18.0	144.2	13.0	18.0
Meinheit & Jirsa (1981)	MJ6	IMF	1.00	192.2	11.0	18.0	144.2	13.0	18.0
Noguchi et al. (1992)	NKOKJ1	OMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Noguchi et al. (1992)	NKOKJ3	OMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Noguchi et al. (1992)	NKOKJ4	IMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Noguchi et al. (1992)	NKOKJ5	IMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Noguchi et al. (1992)	NKOKJ6	IMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Oka & Shiohara (1992)	OSJ1	IMF	0.98	112.6	9.4	11.8	56.7	11.8	11.8
Oka & Shiohara (1992)	OSJ10	IMF	0.98	112.6	9.4	11.8	56.7	11.8	11.8
Oka & Shiohara (1992)	OSJ11	OMF	0.98	112.6	9.4	11.8	56.7	11.8	11.8
Oka & Shiohara (1992)	OSJ2	IMF	0.98	112.6	9.4	11.8	56.7	11.8	11.8
Oka & Shiohara (1992)	OSJ4	IMF	0.98	112.6	9.4	11.8	56.7	11.8	11.8
Oka & Shiohara (1992)	OSJ5	IMF	0.98	112.6	9.4	11.8	56.7	11.8	11.8
Oka & Shiohara (1992)	OSJ6	IMF	0.98	112.6	9.4	11.8	56.7	11.8	11.8
Oka & Shiohara (1992)	OSJ7	IMF	0.98	112.6	9.4	11.8	56.7	11.8	11.8
Oka & Shiohara (1992)	OSJ8	OMF	0.98	112.6	9.4	11.8	56.7	11.8	11.8
Otani et al. (1984)	J1	IMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Otani et al. (1984)	J2	IMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Otani et al. (1984)	J3	IMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Otani et al. (1984)	J4	IMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Otani et al. (1984)	J5	IMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Otani et al. (1984)	J6	NC	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Otani et al. (1984)	S1	SMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Otani et al. (1984)	S2	SMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Otani et al. (1984)	S3	SMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Otani et al. (1984)	S4	SMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Otani et al. (1984)	S6	NC	0.98	106.3	7.9	11.8	57.9	11.8	11.8

Table 10: Geometric properties for frame sub-assemblage test specimens (continued)

Frame Sub-assemblage Tests		Frame Class	Geometric properties						
			Scale - assuming full scale is column depth > 12 in.	Beam length [in]	Beam width [in]	Beam depth [in]	Column Length [in]	Column width (out-of-plane) [in]	Column depth [in]
Park & Ruitong (1988)	Unit 1	IMF	1.00	166.9	9.0	18.0	97.4	12.0	16.0
Park & Ruitong (1988)	Unit 2	IMF	1.00	166.9	9.0	18.0	97.4	12.0	16.0
Park & Ruitong (1988)	Unit 3	IMF	1.00	166.9	9.0	18.0	97.4	12.0	16.0
Park & Ruitong (1988)	Unit 4	IMF	1.00	166.9	9.0	18.0	97.4	12.0	16.0
Alire (2002), Walker	PEER09	OMF	1.00	160.0	16.0	20.0	84.0	16.0	18.0
Alire (2002), Walker	PEER14	OMF	1.00	160.0	16.0	20.0	84.0	16.0	18.0
Alire (2002), Walker	PEER15	OMF	1.00	160.0	16.0	20.0	84.0	16.0	18.0
Alire (2002), Walker	PEER22	OMF	1.00	160.0	16.0	20.0	84.0	16.0	18.0
Alire (2002), Walker	PEER41	OMF	1.00	160.0	16.0	20.0	84.0	16.0	18.0
Pessiki et (1990a)	1	NC	1.00	138.0	14.0	24.0	130.5	16.0	16.0
Pessiki et al. (1990)	2	NC	1.00	138.0	14.0	24.0	130.5	16.0	16.0
Pessiki et al. (1990)	3	NC	1.00	138.0	14.0	24.0	130.5	16.0	16.0
Pessiki et al. (1990)	4	NC	1.00	138.0	14.0	24.0	130.5	16.0	16.0
Pessiki et al. (1990)	5	NC	1.00	138.0	14.0	24.0	130.5	16.0	16.0
Pessiki et al. (1990)	6	NC	1.00	138.0	14.0	24.0	130.5	16.0	16.0
Pessiki et al. (1990)	7	NC	1.00	138.0	14.0	24.0	130.5	16.0	16.0
Pessiki et al. (1990)	8	NC	1.00	138.0	14.0	24.0	130.5	16.0	16.0
Pessiki et al. (1990)	9	NC	1.00	138.0	14.0	24.0	130.5	16.0	16.0
Supaviriyakit et al. (2007)	SP-J1	NC	1.00	118.1	6.9	11.8	50.0	7.9	13.8
Supaviriyakit et al. (2007)	SP-J2	NC	1.00	118.1	6.9	11.8	50.0	7.9	13.8
Supaviriyakit et al. (2007)	SP-J3A	NC	1.00	118.1	6.9	11.8	50.0	7.9	13.8
Supaviriyakit et al. (2007)	SP-J3B	NC	1.00	118.1	6.9	11.8	50.0	7.9	13.8
Supaviriyakit et al. (2007)	SP-J4	NC	1.00	118.1	6.9	11.8	50.0	11.8	15.7
Teraoka et al. (1997)	HJ-12	IMF	1.00	110.2	11.8	15.7	70.9	15.7	15.7
Teraoka et al. (1997)	HJ-14	IMF	1.00	110.2	11.8	15.7	70.9	15.7	15.7
Teraoka et al. (1997)	HJ-2	IMF	1.00	118.1	11.8	15.7	78.7	15.7	15.7
Teraoka et al. (1997)	HJ-4	OMF	1.00	118.1	11.8	15.7	78.7	15.7	15.7
Teraoka et al. (1997)	HJ-5	OMF	1.00	118.1	11.8	15.7	78.7	15.7	15.7
Teraoka et al. (1997)	HJ-6	IMF	1.00	118.1	11.8	15.7	78.7	15.7	15.7
Teraoka et al. (1997)	HJ-7	IMF	1.00	118.1	11.8	15.7	78.7	15.7	15.7
Teraoka et al. (1997)	HJ-8	IMF	1.00	118.1	11.8	15.7	78.7	15.7	15.7
Teraoka et al. (1997)	HJ-9	IMF	1.00	118.1	11.8	15.7	78.7	15.7	15.7
Teraoka et al. (1997)	HNO1	IMF	1.00	110.2	11.8	15.7	70.9	15.7	15.7
Teraoka et al. (1997)	HNO2	IMF	1.00	110.2	11.8	15.7	70.9	15.7	15.7
Teraoka et al. (1997)	HNO3	IMF	1.00	110.2	11.8	15.7	70.9	15.7	15.7
Teraoka et al. (1997)	HNO4	IMF	1.00	110.2	11.8	15.7	70.9	15.7	15.7
Teraoka et al. (1997)	HNO5	IMF	1.00	110.2	11.8	15.7	70.9	15.7	15.7
Teraoka et al. (1997)	HNO6	IMF	1.00	110.2	11.8	15.7	70.9	15.7	15.7
Xin (1992)	Unit1	IMF	1.00	137.8	9.8	19.7	97.2	11.8	17.7
Xin (1992)	Unit2	IMF	1.00	137.8	9.8	19.7	97.2	11.8	17.7
Xin (1992)	Unit3	SMF	1.00	137.8	9.8	19.7	97.2	11.8	17.7
Xin (1992)	Unit4	IMF	1.00	137.8	9.8	19.7	97.2	11.8	17.7
Xin (1992)	Unit5	IMF	1.00	137.8	9.8	19.7	97.2	11.8	17.7
Xin (1992)	Unit6	IMF	1.00	137.8	9.8	19.7	97.2	11.8	17.7
Zai et al. (2001)	S1	IMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8
Zai et al. (2001)	S2	IMF	0.98	106.3	7.9	11.8	57.9	11.8	11.8

Note that "beam length" is the total distance between the points of applied load (or reaction points) at the ends of the two beam segments. Similarly "column length" is the total distance between the points of applied lateral load (or reaction points) at the ends of the column segments.

Table 11: Design parameters for frame sub-assemblage test specimens

Frame Sub-assemblage Tests		Frame Class	f'c [psi]	Column				Beam				Joint			
				Axial Load /Ag*f'c (%)	Long. Reinf. Ratio (%)	Trans. Reinf. Ratio (high) (%)	Trans. Reinf. Ratio (low) (%)	Top Long. Reinf. Ratio (%)	Bottom Long. Reinf. Ratio (%)	Trans. Reinf. Ratio (high) (%)	Trans. Reinf. Ratio (low) (%)	Trans. Reinf. Ratio (%)	Ratio of Prov'd. to Req'd. Column Depth (~20db)	Shear Stress Demand / √fc [psi]	∑Mc/∑Mb at Joint
Shiohara et al. (2006)	A1	IMF	4061	8.6	2.82	0.51	0.51	0.28	0.28	0.42	0.42	0.51	2.14	3.87	3.69
Beckingsale et al.	B11	SMF	5207	4.1	2.18	2.56	0.64	1.04	0.52	0.52	0.13	2.56	1.67	7.76	1.42
Beckingsale et al.	B12	SMF	5018	4.3	2.18	2.56	0.64	0.78	0.78	0.52	0.13	2.56	1.67	7.88	1.42
Beckingsale et al.	B13	SMF	4554	44.1	2.18	2.05	0.52	0.78	0.78	0.52	0.13	2.05	1.67	8.91	2.10
Birss et al. (1978)	B1	SMF	4047	5.3	2.60	1.32	0.50	1.16	1.16	0.50	0.19	1.32	1.64	12.66	1.16
Birss et al. (1978)	B2	IMF	4569	43.9	2.60	0.33	0.13	1.16	1.16	0.50	0.19	0.33	1.64	11.89	1.20
Dhakala et al. (2007)	DPI-C1	NC	4583	10.0	2.24	0.00	0.00	2.44	0.97	0.52	0.52	0.00	0.60	31.74	0.87
Dhakala et al. (2007)	DPI-C4	NC	4743	10.0	2.45	0.00	0.00	2.92	1.46	0.52	0.52	0.00	0.48	44.16	0.58
Durrani & Wight (1982)	X1	IMF	4981	5.4	3.10	1.06	1.06	1.33	0.98	1.02	1.02	1.06	1.02	11.03	1.58
Durrani & Wight (1982)	X2	SMF	4881	5.6	3.10	2.12	2.12	1.33	0.98	1.02	1.02	2.12	1.02	11.24	1.65
Durrani & Wight (1982)	X3	IMF	4501	5.3	2.06	1.06	1.06	1.00	0.73	1.02	1.02	1.06	1.02	8.74	1.34
Endoh et al. (1991)	A1	OMF	4438	6.4	3.57	0.56	0.56	1.69	0.84	1.41	1.41	0.56	0.63	23.38	1.70
Endoh et al. (1991)	HC	IMF	6016	4.7	2.68	0.42	0.42	1.43	1.43	0.57	0.57	0.42	1.74	10.46	1.61
Fujii & Morita (1991)	A1	OMF	5833	7.6	3.29	0.68	0.68	1.57	1.57	0.44	0.44	0.68	0.42	46.87	0.82
Fujii & Morita (1991)	A2	OMF	5833	7.6	3.29	0.68	0.68	1.57	1.57	0.44	0.44	0.68	1.09	17.55	1.74
Fujii & Morita (1991)	A3	OMF	5833	22.7	3.29	1.36	1.36	1.57	1.57	0.44	0.44	1.36	0.42	46.87	0.75
Fujii & Morita (1991)	A4	OMF	5833	22.7	3.29	1.36	1.36	1.57	1.57	0.44	0.44	1.36	0.42	46.87	0.75
Hayashi et al. (1993)	NO47	OMF	7865	18.0	2.13	0.92	0.77	1.42	1.42	0.38	0.31	0.92	1.14	10.29	2.15
Joh et al. (1991)	B1	OMF	3901	14.6	1.97	0.25	0.25	0.72	0.72	0.38	0.38	0.25	1.21	6.24	1.66
Joh et al. (1991)	B2	OMF	3269	16.6	1.97	0.25	0.25	0.54	0.54	0.28	0.28	0.25	1.21	6.72	1.62
Joh et al. (1991)	B8-HH	SMF	3713	15.3	1.97	0.72	0.36	0.54	0.54	0.85	0.42	0.72	1.21	6.21	2.16
Joh et al. (1991)	B8-HL	OMF	3974	14.3	1.97	0.72	0.72	0.54	0.54	0.28	0.28	0.72	1.21	5.99	2.18
Joh et al. (1991)	B8-LH	IMF	3429	16.6	1.97	0.25	0.13	0.54	0.54	0.85	0.42	0.25	1.21	6.45	2.17
Joh et al. (1991)	B8-MH	IMF	4076	14.0	1.97	0.52	0.26	0.54	0.54	0.85	0.42	0.52	1.21	5.91	2.19
Kitayama et al.(1991)	KOAC1	IMF	3713	7.7	2.36	0.33	0.33	1.57	0.79	0.63	0.63	0.33	1.94	9.27	3.13
Kitayama et al.(1991)	KOAJ1	IMF	3726	7.6	2.36	0.33	0.33	1.77	0.88	0.63	0.63	0.33	1.19	13.23	2.26
Milburn & Park (1982)	Unit 1	SMF	5990	10.0	2.19	4.89	2.45	1.54	1.54	1.54	0.77	4.89	1.67	13.12	1.35
Milburn & Park (1982)	Unit 2	SMF	6802	10.0	2.19	3.92	1.96	1.20	1.20	1.54	0.77	3.92	1.37	9.17	1.64
Meinheit & Jirsa (1981)	MJ1	IMF	3801	40.1	2.06	0.62	0.31	1.92	1.19	0.57	0.29	0.62	0.65	23.12	0.89
Meinheit & Jirsa (1981)	MJ12	IMF	5101	30.3	4.33	2.89	1.44	1.92	1.19	0.57	0.29	2.89	0.65	19.93	1.54
Meinheit & Jirsa (1981)	MJ13	IMF	5992	25.1	4.33	1.87	0.93	1.92	1.19	0.57	0.29	1.87	0.65	18.36	1.65
Meinheit & Jirsa (1981)	MJ2	IMF	6062	25.3	4.33	0.62	0.31	1.92	1.19	0.57	0.29	0.62	0.65	18.26	1.65
Meinheit & Jirsa (1981)	MJ3	IMF	3861	39.3	6.67	0.62	0.31	1.92	1.19	0.57	0.29	0.62	0.65	22.94	1.62
Meinheit & Jirsa (1981)	MJ5	IMF	5201	3.9	4.33	0.62	0.31	1.92	1.19	0.57	0.29	0.62	0.65	19.73	1.51
Meinheit & Jirsa (1981)	MJ6	IMF	5331	48.2	4.33	0.62	0.31	1.92	1.19	0.57	0.29	0.62	0.65	19.48	1.38
Noguchi et al. (1992)	NKOKJ1	OMF	10153	12.0	2.95	0.50	0.50	1.99	1.55	0.57	0.57	0.50	0.66	19.41	1.41
Noguchi et al. (1992)	NKOKJ3	OMF	15519	12.0	3.24	1.00	1.00	2.21	2.21	0.57	0.57	1.00	0.66	19.73	1.47
Noguchi et al. (1992)	NKOKJ4	IMF	10153	12.0	2.95	1.00	1.00	1.99	1.55	0.57	0.57	1.00	0.66	19.41	1.41
Noguchi et al. (1992)	NKOKJ5	IMF	10153	12.0	3.54	0.50	0.50	2.21	2.21	0.57	0.57	0.50	0.66	24.55	1.34
Noguchi et al. (1992)	NKOKJ6	IMF	7760	12.0	2.95	0.50	0.50	1.77	1.55	0.57	0.57	0.50	0.66	20.85	1.46
Oka & Shiohara (1992)	OSJ1	IMF	11777	11.4	0.56	0.46	0.46	1.58	1.23	0.47	0.47	0.46	0.77	15.11	1.75
Oka & Shiohara (1992)	OSJ10	IMF	5685	11.8	0.56	0.46	0.46	1.58	1.23	0.47	0.47	0.46	0.70	24.31	1.33
Oka & Shiohara (1992)	OSJ11	OMF	5685	11.8	1.27	0.46	0.46	3.58	2.79	0.47	0.47	0.46	0.87	29.00	1.38
Oka & Shiohara (1992)	OSJ2	IMF	11777	11.4	0.59	0.46	0.46	1.47	1.47	0.47	0.47	0.46	0.33	37.39	0.96
Oka & Shiohara (1992)	OSJ4	IMF	10559	12.7	0.56	0.46	0.46	1.76	1.76	0.47	0.47	0.46	0.95	16.11	1.55
Oka & Shiohara (1992)	OSJ5	IMF	10559	12.7	0.56	0.46	0.46	1.58	1.23	0.47	0.47	0.46	0.58	21.25	1.40
Oka & Shiohara (1992)	OSJ6	IMF	11487	11.7	0.56	0.23	0.23	1.58	1.23	0.47	0.47	0.23	0.72	16.28	1.68
Oka & Shiohara (1992)	OSJ7	IMF	11487	11.7	0.56	0.46	0.46	1.23	0.88	0.47	0.47	0.46	0.72	11.98	2.09
Oka & Shiohara (1992)	OSJ8	OMF	11487	11.7	1.27	0.46	0.46	3.58	2.79	0.47	0.47	0.46	0.88	19.94	1.63
Otani et al. (1984)	J1	IMF	3727	7.6	2.25	0.33	0.33	1.69	0.84	0.63	0.63	0.33	1.22	13.04	1.55
Otani et al. (1984)	J2	IMF	3486	8.1	2.25	0.66	0.66	1.69	0.84	0.63	0.63	0.66	1.22	13.51	1.54
Otani et al. (1984)	J3	IMF	3486	8.1	2.25	1.99	1.99	1.69	0.84	0.63	0.63	1.99	1.22	13.51	1.54
Otani et al. (1984)	J4	IMF	3727	30.5	2.25	0.33	0.33	1.69	0.84	0.63	0.63	0.33	1.22	13.04	1.83
Otani et al. (1984)	J5	IMF	3727	7.6	1.41	0.33	0.33	1.69	0.84	0.63	0.63	0.33	1.22	13.00	1.17
Otani et al. (1984)	J6	NC	3727	22.8	0.95	0.50	0.50	0.84	0.63	0.32	0.32	0.50	1.22	7.60	2.24
Otani et al. (1984)	S1	SMF	3727	22.9	0.95	1.98	1.98	0.84	0.63	0.63	0.63	1.98	1.42	6.29	1.55
Otani et al. (1984)	S2	SMF	3486	8.1	0.95	1.98	1.98	0.84	0.63	0.63	0.63	1.98	1.42	6.50	1.54
Otani et al. (1984)	S3	SMF	3486	24.5	0.95	1.98	1.98	0.83	0.59	0.63	0.63	1.98	1.56	7.67	1.54
Otani et al. (1984)	S4	SMF	3727	22.9	0.95	1.98	1.98	0.99	0.66	0.63	0.63	1.98	1.05	7.69	1.83
Otani et al. (1984)	S6	NC	3727	22.9	0.95	0.50	0.50	0.84	0.63	0.32	0.32	0.50	1.42	6.31	2.24

Table 11: Design parameters for frame sub-assemblage test specimens (continued)

Frame Sub-assemblage Tests		Frame Class	f'c [psi]	Column				Beam				Joint			
				Axial Load /Ag*f'c (%)	Long. Reinf. Ratio (%)	Trans. Reinf. Ratio (high) (%)	Trans. Reinf. Ratio (low) (%)	Top Long. Reinf. Ratio (%)	Bottom Long. Reinf. Ratio (%)	Trans. Reinf. Ratio (high) (%)	Trans. Reinf. Ratio (low) (%)	Trans. Reinf. Ratio (%)	Ratio of Prov'd. to Req'd. Column Depth (~20db)	Shear Stress Demand / √fc [psi]	∑Mc/∑Mb at Joint
Park & Ruitong (1988)	Unit 1	IMF	6657	1.9	1.30	1.73	0.77	0.96	0.38	0.31	0.14	1.73	1.79	4.71	1.89
Park & Ruitong (1988)	Unit 2	IMF	5221	3.0	2.0	2.1	0.9	1.2	0.6	0.4	0.2	2.11	0.96	7.42	1.88
Park & Ruitong (1988)	Unit 3	IMF	5250	2.5	1.5	0.8	0.3	1.0	0.4	0.3	0.1	0.75	1.79	5.33	1.73
Park & Ruitong (1988)	Unit 4	IMF	5816	2.7	1.8	1.1	0.5	1.2	0.6	0.4	0.2	1.09	0.96	7.02	1.78
Alire (2002), Walker	PEER09	OMF	8769	10.0	1.2	0.0	0.0	0.9	0.6	1.1	1.1	0.00	0.84	8.05	1.53
Alire (2002), Walker	PEER14	OMF	4607	10.0	1.5	0.0	0.0	0.8	0.4	1.1	1.1	0.00	1.01	7.65	1.82
Alire (2002), Walker	PEER15	OMF	8917	10.0	3.5	0.0	0.0	0.8	0.6	1.4	1.4	0.00	0.98	8.71	3.41
Alire (2002), Walker	PEER22	OMF	5571	10.0	2.8	0.0	0.0	1.1	0.8	1.1	1.1	0.00	0.83	15.00	1.86
Alire (2002), Walker	PEER41	OMF	4784	10.0	3.5	0.0	0.0	1.9	1.9	1.4	1.4	0.00	0.61	35.44	1.13
Pessiki et (1990a)	1	NC	5000	27.3	1.9	0.0	0.0	1.2	0.6	0.3	0.3	0.00	0.61	18.52	0.86
Pessiki et al. (1990)	2	NC	4000	34.2	1.9	0.0	0.0	1.2	0.6	0.3	0.3	0.00	0.61	20.77	0.78
Pessiki et al. (1990)	3	NC	4000	34.2	1.8	0.0	0.0	1.2	0.6	0.3	0.3	0.00	0.61	20.77	0.74
Pessiki et al. (1990)	4	NC	4000	34.2	1.9	0.0	0.0	1.2	0.6	0.3	0.3	0.00	0.61	21.47	0.77
Pessiki et al. (1990)	5	NC	3750	36.6	1.9	0.2	0.2	1.2	0.6	0.3	0.3	0.22	0.61	22.19	0.72
Pessiki et al. (1990)	6	NC	3000	45.6	1.9	0.4	0.4	1.2	0.6	0.3	0.3	0.45	0.61	23.96	0.63
Pessiki et al. (1990)	7	NC	3000	45.6	1.9	0.0	0.0	0.7	0.5	0.3	0.3	0.00	0.69	15.95	0.95
Pessiki et al. (1990)	8	NC	3000	45.7	1.9	0.0	0.0	0.7	0.3	0.3	0.3	0.00	0.78	13.22	1.14
Pessiki et al. (1990)	9	NC	4000	9.8	1.9	0.0	0.0	0.7	0.5	0.3	0.3	0.00	0.79	12.00	1.00
Supaviriyakit et al.	SP-J1	NC	3814	12.5	2.9	0.0	0.0	1.3	1.3	0.1	0.1	0.00	1.26	16.85	2.12
Supaviriyakit et al.	SP-J2	NC	3089	12.5	2.9	0.0	0.0	1.3	1.3	0.1	0.1	0.00	1.26	18.79	1.90
Supaviriyakit et al.	SP-J3A	NC	4163	12.5	2.9	0.9	0.9	1.3	1.3	0.1	0.1	0.87	1.26	16.10	2.21
Supaviriyakit et al.	SP-J3B	NC	3437	12.5	2.9	1.2	1.2	1.3	1.3	0.1	0.1	1.17	1.26	17.78	2.01
Supaviriyakit et al.	SP-J4	NC	3336	12.5	2.5	0.0	0.0	1.3	1.3	0.1	0.1	0.00	1.44	10.39	3.44
Teraoka et al. (1997)	HJ-12	IMF	12801	20.0	2.7	1.2	1.2	2.6	2.6	0.8	0.8	1.21	0.62	22.73	1.14
Teraoka et al. (1997)	HJ-14	IMF	17068	20.0	2.7	1.2	1.2	2.6	2.6	0.8	0.8	1.21	0.62	21.66	1.99
Teraoka et al. (1997)	HJ-2	IMF	7823	20.0	2.2	0.9	0.9	0.7	0.7	0.5	0.5	0.85	0.83	7.69	2.28
Teraoka et al. (1997)	HJ-4	OMF	7823	20.0	2.2	0.9	0.9	1.4	1.4	0.4	0.4	0.85	1.13	10.35	2.07
Teraoka et al. (1997)	HJ-5	OMF	7823	20.0	2.2	0.9	0.9	1.0	1.0	0.4	0.4	0.85	0.67	11.58	1.81
Teraoka et al. (1997)	HJ-6	IMF	7823	20.0	2.2	0.9	0.9	0.7	0.7	0.4	0.4	0.85	0.50	11.65	1.86
Teraoka et al. (1997)	HJ-7	IMF	12801	20.0	2.2	1.2	1.2	1.9	1.9	0.5	0.5	1.21	0.88	12.05	1.86
Teraoka et al. (1997)	HJ-8	IMF	12801	20.0	2.2	1.2	1.2	1.3	1.3	0.5	0.5	1.21	0.62	11.35	1.92
Teraoka et al. (1997)	HJ-9	IMF	12801	20.0	2.2	1.2	1.2	1.0	1.0	0.5	0.5	1.21	0.50	12.10	1.83
Teraoka et al. (1997)	HNO1	IMF	12858	17.0	2.9	1.3	1.0	1.3	1.3	0.5	0.4	1.26	0.85	11.80	2.03
Teraoka et al. (1997)	HNO2	IMF	12858	17.0	2.9	1.3	1.0	1.3	1.3	0.6	0.5	1.26	0.85	11.80	2.03
Teraoka et al. (1997)	HNO3	IMF	12858	17.0	2.9	1.3	1.0	2.5	2.5	0.6	0.5	1.26	0.85	16.19	1.39
Teraoka et al. (1997)	HNO4	IMF	12858	17.0	2.9	1.3	1.3	2.5	2.5	0.7	0.7	1.26	0.62	22.52	1.16
Teraoka et al. (1997)	HNO5	IMF	16954	13.0	2.9	1.3	1.0	0.7	0.7	0.6	0.5	1.26	0.85	2.59	1.82
Teraoka et al. (1997)	HNO6	IMF	16954	13.0	2.9	1.3	1.3	2.5	2.5	0.7	0.7	1.26	0.62	19.44	1.26
Xin (1992)	Unit1	IMF	4482	0.0	2.8	1.6	1.2	0.6	0.6	0.3	0.2	1.59	1.71	8.79	3.19
Xin (1992)	Unit2	IMF	5918	0.1	1.9	1.2	1.2	0.6	0.6	0.7	0.7	1.17	1.31	7.62	2.38
Xin (1992)	Unit3	SMF	6164	0.2	3.3	1.6	1.5	0.6	0.6	0.7	0.7	1.59	1.31	7.46	3.83
Xin (1992)	Unit4	IMF	6846	0.2	2.4	1.3	1.2	0.3	0.3	0.7	0.7	1.31	1.31	3.49	5.85
Xin (1992)	Unit5	IMF	8804	0.2	3.3	1.9	1.8	0.8	0.8	0.7	0.7	1.91	0.95	8.10	3.04
Xin (1992)	Unit6	IMF	8601	0.3	3.3	1.9	1.8	1.0	0.5	1.0	1.0	1.91	0.72	7.77	3.21
Zai et al. (2001)	S1	IMF	3481	4.6	3.8	0.4	0.4	0.7	0.7	0.6	0.6	0.37	1.59	6.53	4.44
Zai et al. (2001)	S2	IMF	3481	4.6	3.8	0.4	0.4	2.2	2.2	1.1	1.1	0.37	1.51	20.42	1.74

Table 12: Category, response mode and damage data for frame sub-assemblage specimens

Frame Sub-assemblage Tests		ACI Frame Class	ASCE Frame Class	Resp. Mode	Damage Data for DS
Shiohara et al. (2006)	A1	IMF	1	BYJF	C,0,2,3
Beckingsale et al.	B11	SMF	1	BY	C,0,2
Beckingsale et al.	B12	SMF	1	BY	0,2
Beckingsale et al.	B13	SMF	1	BY	0,2
Birss et al. (1978)	B1	SMF	1	BY	0,2
Birss et al. (1978)	B2	IMF	1	BY	C,0,1,2
Dhakala et al. (2007)	DPI-C1	NC	6	JF	C,0,3
Dhakala et al. (2007)	DPI-C4	NC	6	JF	C,0,3
Durrani & Wight (1982)	X1	IMF	1	BYJF	0,2,3
Durrani & Wight (1982)	X2	SMF	1	BY	C,0,3
Durrani & Wight (1982)	X3	IMF	1	BY	C,0,3
Endoh et al. (1991)	A1	OMF	6	JF	C,3
Endoh et al. (1991)	HC	IMF	1	BY	0
Fujii & Morita (1991)	A1	OMF	6	JF	0,3
Fujii & Morita (1991)	A2	OMF	1	JF	0,3
Fujii & Morita (1991)	A3	OMF	6	JF	0,3
Fujii & Morita (1991)	A4	OMF	6	JF	0,3
Hayashi et al. (1993)	NO47	OMF	1	BYJF	0,3
Joh et al. (1991)	B1	OMF	1	BYJF	C,0,3
Joh et al. (1991)	B2	OMF	1	BY	C,0,3
Joh et al. (1991)	B8-HH	SMF	1	BYJF	C,0,2
Joh et al. (1991)	B8-HL	OMF	1	BYJF	C,0,2,3
Joh et al. (1991)	B8-LH	IMF	1	BY	C,0,3
Joh et al. (1991)	B8-MH	IMF	1	BY	C,0,3
Kitayama et al.(1991)	KOAC1	IMF	1	BY	3
Kitayama et al.(1991)	KOAJ1	IMF	1	JF	3
Milburn & Park (1982)	Unit 1	SMF	1	BY	C,0,2,3
Milburn & Park (1982)	Unit 2	SMF	1	BYJF	C,0,3
Meinheit & Jirsa (1981)	MJ1	IMF	6	JF	3
Meinheit & Jirsa (1981)	MJ12	IMF	2	BYJF	3
Meinheit & Jirsa (1981)	MJ13	IMF	2	JF	3
Meinheit & Jirsa (1981)	MJ2	IMF	2	BYJF	C,0,3
Meinheit & Jirsa (1981)	MJ3	IMF	6	JF	2,3
Meinheit & Jirsa (1981)	MJ5	IMF	2	JF	3
Meinheit & Jirsa (1981)	MJ6	IMF	2	JF	0
Noguchi et al. (1992)	NKOKJ1	OMF	2	BYJF	0,3
Noguchi et al. (1992)	NKOKJ3	OMF	2	JF	3
Noguchi et al. (1992)	NKOKJ4	IMF	2	BYJF	0
Noguchi et al. (1992)	NKOKJ5	IMF	6	JF	3
Noguchi et al. (1992)	NKOKJ6	IMF	2	JF	3
Oka & Shiohara (1992)	OSJ1	IMF	1	BYJF	0
Oka & Shiohara (1992)	OSJ10	IMF	6	JF	0
Oka & Shiohara (1992)	OSJ11	OMF	6	JF	0
Oka & Shiohara (1992)	OSJ2	IMF	6	JF	0
Oka & Shiohara (1992)	OSJ4	IMF	1	BY	0
Oka & Shiohara (1992)	OSJ5	IMF	2	JF	0
Oka & Shiohara (1992)	OSJ6	IMF	1	BYJF	0
Oka & Shiohara (1992)	OSJ7	IMF	1	BY	0
Oka & Shiohara (1992)	OSJ8	OMF	2	JF	0
Otani et al. (1984)	J1	IMF	1	BY	C,0,2,3
Otani et al. (1984)	J2	IMF	1	BY	C,0,3
Otani et al. (1984)	J3	IMF	1	BY	C,0
Otani et al. (1984)	J4	IMF	1	BYJF	0,2,3
Otani et al. (1984)	J5	IMF	1	BYJF	C,0,3
Otani et al. (1984)	J6	NC	1	BY	C,0,3
Otani et al. (1984)	S1	SMF	1	BY	0
Otani et al. (1984)	S2	SMF	1	BY	0
Otani et al. (1984)	S3	SMF	1	BY	0
Otani et al. (1984)	S4	SMF	1	BYJF	0
Otani et al. (1984)	S6	NC	1	BY	0

Frame Sub-assemblage Tests		Frame Class	ASCE Frame Class	Resp. Mode	Damage Data for DS
Park & Ruitong (1988)	Unit 1	IMF	1	BY	C,0,2
Park & Ruitong (1988)	Unit 2	IMF	1	BY	C,0,2,3
Park & Ruitong (1988)	Unit 3	IMF	1	BY	0,2
Park & Ruitong (1988)	Unit 4	IMF	1	BYJF	0,2
Alire (2002), Walker	PEER09	OMF	4	BYJF	C,0,2,3
Alire (2002), Walker	PEER14	OMF	4	BYJF	C,0,1,3
Alire (2002), Walker	PEER15	OMF	4	BYJF	C,0,2,3
Alire (2002), Walker	PEER22	OMF	5	BYJF	C,0,1,2,3,4
Alire (2002), Walker	PEER41	OMF	6	JF	0,3
Pessiki et (1990a)	1	NC	6	CYJF	C,1,2,3
Pessiki et al. (1990)	2	NC	6	CYJF	C,3
Pessiki et al. (1990)	3	NC	6	CYJF	C,3,4
Pessiki et al. (1990)	4	NC	6	CYJF	C,3,4
Pessiki et al. (1990)	5	NC	6	CYJF	C,3,4
Pessiki et al. (1990)	6	NC	6	CYJF	C,3,4
Pessiki et al. (1990)	7	NC	6	CYJF	C,3,4
Pessiki et al. (1990)	8	NC	5	CYJF	C,3,4
Pessiki et al. (1990)	9	NC	5	CYJF	C,3,4
Supaviriyakit et al.	SP-J1	NC	6	JF	C,3
Supaviriyakit et al.	SP-J2	NC	6	BY	C,2,3
Supaviriyakit et al.	SP-J3A	NC	1	JF	C,3
Supaviriyakit et al.	SP-J3B	NC	1	JF	C,3
Supaviriyakit et al.	SP-J4	NC	1	BY	C,0,2,3
Teraoka et al. (1997)	HJ-12	IMF	6	BYJF	0,2,3
Teraoka et al. (1997)	HJ-14	IMF	2	BYJF	0,2,3
Teraoka et al. (1997)	HJ-2	IMF	1	BY	0,2,3
Teraoka et al. (1997)	HJ-4	OMF	1	BYJF	0,2,3
Teraoka et al. (1997)	HJ-5	OMF	1	BYJF	0,2
Teraoka et al. (1997)	HJ-6	IMF	1	BYJF	0,2
Teraoka et al. (1997)	HJ-7	IMF	1	BYJF	0,2
Teraoka et al. (1997)	HJ-8	IMF	1	BYJF	0,2
Teraoka et al. (1997)	HJ-9	IMF	1	BYJF	0,2
Teraoka et al. (1997)	HNO1	IMF	1	JF	C,3
Teraoka et al. (1997)	HNO2	IMF	1	BY	C,3
Teraoka et al. (1997)	HNO3	IMF	1	JF	C,3
Teraoka et al. (1997)	HNO4	IMF	6	JF	C,3
Teraoka et al. (1997)	HNO5	IMF	1	JF	C
Teraoka et al. (1997)	HNO6	IMF	2	JF	C,3
Xin (1992)	Unit1	IMF	1	BY	C,0,2,3
Xin (1992)	Unit2	IMF	1	BY	0,2
Xin (1992)	Unit3	SMF	1	BY	0,2,3
Xin (1992)	Unit4	IMF	1	BY	0,2
Xin (1992)	Unit5	IMF	1	BY	0,2
Xin (1992)	Unit6	IMF	1	BY	0,2
Zai et al. (2001)	S1	IMF	1	BY	0,1,2
Zai et al. (2001)	S2	IMF	2	BY	C,0,1,2

Table 13: ACI Code compliance data

Frame Sub-assemblage Tests		ACI Frame Class	Joint meets req'ts for SFM	Beam meets req't for SMF	Col. meets req't for SMF	Frame meets req'ts for SMF	Joint meets req'ts for IFM	Beam meets req't for IMF	Col. meets req't for IMF	Frame meets req'ts for IMF	Beam meets req't for OMF	Col. meets req't for OMF	Frame meetso ther req't for OMF	Frame meets req'ts for OMF
Shiohara et al. (2006)	A1	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Beckingsale et al.	B11	SMF	1	1	1	1	1	1	1	1	1	1	1	1
Beckingsale et al.	B12	SMF	1	1	1	1	1	1	1	1	1	1	1	1
Beckingsale et al.	B13	SMF	1	1	1	1	1	1	1	1	1	1	1	1
Birss et al. (1978)	B1	SMF	1	1	1	1	1	1	1	1	1	1	1	1
Birss et al. (1978)	B2	IMF	0	1	1	0	1	1	1	1	1	1	1	1
Dhakala et al. (2007)	DPI-C1	NC	0	0	0	0	0	0	0	0	1	1	0	0
Dhakala et al. (2007)	DPI-C4	NC	0	0	0	0	0	0	0	0	1	1	0	0
Durrani & Wight (1982)	X1	IMF	0	1	1	0	1	1	1	1	1	1	1	1
Durrani & Wight (1982)	X2	SMF	1	1	1	1	1	1	1	1	1	1	1	1
Durrani & Wight (1982)	X3	IMF	0	1	1	0	1	1	1	1	1	1	1	1
Endoh et al. (1991)	A1	OMF	0	1	0	0	1	1	0	0	1	1	1	1
Endoh et al. (1991)	HC	IMF	0	1	1	0	1	1	1	1	1	1	1	1
Fujii & Morita (1991)	A1	OMF	0	0	0	0	1	0	1	0	1	1	1	1
Fujii & Morita (1991)	A2	OMF	0	0	0	0	1	0	1	0	1	1	1	1
Fujii & Morita (1991)	A3	OMF	0	0	0	0	1	0	1	0	1	1	1	1
Fujii & Morita (1991)	A4	OMF	0	0	0	0	1	0	1	0	1	1	1	1
Hayashi et al. (1993)	NO47	OMF	0	0	0	0	1	0	1	0	1	1	1	1
Joh et al. (1991)	B1	OMF	0	0	0	0	1	0	1	0	1	1	1	1
Joh et al. (1991)	B2	OMF	0	0	0	0	1	0	1	0	1	1	1	1
Joh et al. (1991)	B8-HH	SMF	1	1	0	1	1	1	1	1	1	1	1	1
Joh et al. (1991)	B8-HL	OMF	1	0	0	0	1	0	1	0	1	1	1	1
Joh et al. (1991)	B8-LH	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Joh et al. (1991)	B8-MH	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Kitayama et al.(1991)	KOAC1	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Kitayama et al.(1991)	KOAJ1	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Milburn & Park (1982)	Unit 1	SMF	1	1	0	1	1	1	1	1	1	1	1	1
Milburn & Park (1982)	Unit 2	SMF	1	1	0	1	1	1	1	1	1	1	1	1
Meinheit & Jirsa (1981)	MJ1	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Meinheit & Jirsa (1981)	MJ12	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Meinheit & Jirsa (1981)	MJ13	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Meinheit & Jirsa (1981)	MJ2	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Meinheit & Jirsa (1981)	MJ3	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Meinheit & Jirsa (1981)	MJ5	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Meinheit & Jirsa (1981)	MJ6	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Noguchi et al. (1992)	NKOKJ1	OMF	0	0	0	0	1	0	1	0	1	1	1	1
Noguchi et al. (1992)	NKOKJ3	OMF	0	0	0	0	1	0	1	0	1	1	1	1
Noguchi et al. (1992)	NKOKJ4	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Noguchi et al. (1992)	NKOKJ5	IMF	0	0	0	0	1	1	1	1	1	1	1	1
Noguchi et al. (1992)	NKOKJ6	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Oka & Shiohara (1992)	OSJ1	IMF	0	0	0	0	1	1	1	1	1	0	1	0
Oka & Shiohara (1992)	OSJ10	IMF	0	1	0	0	1	1	1	1	1	0	1	0
Oka & Shiohara (1992)	OSJ11	OMF	0	0	0	0	1	0	1	0	1	1	1	1
Oka & Shiohara (1992)	OSJ2	IMF	0	1	0	0	1	1	1	1	1	0	1	0
Oka & Shiohara (1992)	OSJ4	IMF	0	0	0	0	1	1	1	1	1	0	1	0
Oka & Shiohara (1992)	OSJ5	IMF	0	0	0	0	1	1	1	1	1	0	1	0
Oka & Shiohara (1992)	OSJ6	IMF	0	0	0	0	1	1	1	1	1	0	1	0
Oka & Shiohara (1992)	OSJ7	IMF	0	1	0	0	1	1	1	1	1	0	1	0
Oka & Shiohara (1992)	OSJ8	OMF	0	0	0	0	1	0	1	0	1	1	1	1
Otani et al. (1984)	J1	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Otani et al. (1984)	J2	IMF	0	1	1	0	1	1	1	1	1	1	1	1
Otani et al. (1984)	J3	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Otani et al. (1984)	J4	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Otani et al. (1984)	J5	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Otani et al. (1984)	J6	NC	0	0	0	0	1	0	1	0	1	0	1	0
Otani et al. (1984)	S1	SMF	1	1	0	1	1	1	1	1	1	0	1	0
Otani et al. (1984)	S2	SMF	1	1	1	1	1	1	1	1	1	0	1	0
Otani et al. (1984)	S3	SMF	1	1	1	1	1	1	1	1	1	0	1	0
Otani et al. (1984)	S4	SMF	1	1	0	1	1	1	1	1	1	0	1	0
Otani et al. (1984)	S6	NC	0	0	0	0	1	0	1	0	1	0	1	0

Table 13: ACI Code compliance data (continued)

Frame Sub-assemblage Tests		Frame Class	Joint meets req'ts for SFM	Beam meets req't for SMF	Col. meets req't for SMF	Frame meets req'ts for SMF	Joint meets req'ts for IFM?	Beam meets req't for IMF	Col. meets req't for IMF	Frame meets req'ts for IMF	Beam meets req't for OMF	Col. meets req't for OMF	Frame meetsother req't for OMF	Frame meets req'ts for OMF
Park & Ruitong (1988)	Unit 1	IMF	0	0	0	0	1	1	1	1	0	1	1	0
Park & Ruitong (1988)	Unit 2	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Park & Ruitong (1988)	Unit 3	IMF	0	0	0	0	1	1	1	1	0	1	1	0
Park & Ruitong (1988)	Unit 4	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Alire (2002), Walker	PEER09	OMF	0	1	1	0	0	1	0	0	1	1	1	1
Alire (2002), Walker	PEER14	OMF	0	1	1	0	0	1	0	0	1	1	1	1
Alire (2002), Walker	PEER15	OMF	0	1	1	0	0	1	0	0	1	1	1	1
Alire (2002), Walker	PEER22	OMF	0	1	1	0	0	1	0	0	1	1	1	1
Alire (2002), Walker	PEER41	OMF	0	1	0	0	0	1	0	0	1	1	1	1
Pessiki et (1990a)	1	NC	0	0	0	0	0	1	0	0	1	1	0	0
Pessiki et al. (1990)	2	NC	0	0	0	0	0	1	0	0	1	1	0	0
Pessiki et al. (1990)	3	NC	0	0	0	0	0	1	0	0	1	1	0	0
Pessiki et al. (1990)	4	NC	0	0	0	0	0	1	0	0	1	1	0	0
Pessiki et al. (1990)	5	NC	0	0	0	0	1	1	0	0	1	1	0	0
Pessiki et al. (1990)	6	NC	0	0	0	0	1	1	0	0	1	1	0	0
Pessiki et al. (1990)	7	NC	0	0	0	0	0	1	0	0	1	1	0	0
Pessiki et al. (1990)	8	NC	0	1	0	0	0	1	0	0	1	1	0	0
Pessiki et al. (1990)	9	NC	0	0	0	0	0	1	0	0	1	1	0	0
Supaviriyakit et al.	SP-J1	NC	0	0	0	0	0	0	0	0	1	0	0	0
Supaviriyakit et al.	SP-J2	NC	0	0	0	0	0	0	0	0	1	0	0	0
Supaviriyakit et al.	SP-J3A	NC	0	0	0	0	1	0	0	0	1	0	0	0
Supaviriyakit et al.	SP-J3B	NC	0	0	0	0	1	0	0	0	1	0	0	0
Supaviriyakit et al.	SP-J4	NC	0	0	0	0	0	0	0	0	1	0	0	0
Teraoka et al. (1997)	HJ-12	IMF	0	0	0	0	1	1	1	1	1	1	1	1
Teraoka et al. (1997)	HJ-14	IMF	0	0	0	0	1	1	1	1	1	1	1	1
Teraoka et al. (1997)	HJ-2	IMF	0	0	0	0	1	1	1	1	1	1	1	1
Teraoka et al. (1997)	HJ-4	OMF	0	0	0	0	1	0	1	0	1	1	1	1
Teraoka et al. (1997)	HJ-5	OMF	0	0	0	0	1	0	1	0	1	1	1	1
Teraoka et al. (1997)	HJ-6	IMF	0	0	0	0	1	1	1	1	1	1	1	1
Teraoka et al. (1997)	HJ-7	IMF	0	0	0	0	1	1	1	1	1	1	1	1
Teraoka et al. (1997)	HJ-8	IMF	0	0	0	0	1	1	1	1	1	1	1	1
Teraoka et al. (1997)	HJ-9	IMF	0	0	0	0	1	1	1	1	1	1	1	1
Teraoka et al. (1997)	HNO1	IMF	0	0	0	0	1	1	1	1	1	1	1	1
Teraoka et al. (1997)	HNO2	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Teraoka et al. (1997)	HNO3	IMF	0	0	0	0	1	1	1	1	1	1	1	1
Teraoka et al. (1997)	HNO4	IMF	0	0	0	0	1	1	1	1	1	1	1	1
Teraoka et al. (1997)	HNO5	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Teraoka et al. (1997)	HNO6	IMF	0	0	0	0	1	1	1	1	1	1	1	1
Xin (1992)	Unit1	IMF	1	0	1	0	1	1	1	1	1	1	1	1
Xin (1992)	Unit2	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Xin (1992)	Unit3	SMF	1	1	0	1	1	1	1	1	1	1	1	1
Xin (1992)	Unit4	IMF	0	0	0	0	1	1	1	1	0	1	1	0
Xin (1992)	Unit5	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Xin (1992)	Unit6	IMF	0	1	0	0	1	1	1	1	1	1	1	1
Zai et al. (2001)	S1	IMF	0	1	1	0	1	1	1	1	1	1	1	1
Zai et al. (2001)	S2	IMF	0	1	1	0	1	1	1	1	1	1	1	1

Notes: For compliance with ACI Code requirements, 1 indicates compliance with requirement and 0 indicates lack of compliance. Response modes: BY indicates frame response is controlled by flexural yielding of the beam, BYJF indicates strength is initially determined by beam yielding but strength loss in excess of 20% of maximum strength is observed during the test, JF indicates that strength is determined by failure of the joint, CY indicates frame response is controlled by column yielding and CYJF indicates strength is initially determined by column yielding but strength loss in excess of 20% of maximum strength is observed during the test.

Table 14: Design parameters for column specimens

Cantilever Column Test		f'c	Axial load ratio	Depth	Out-of-Plane Dim.	Story Height for Specimen	ρ_l	Min. spacing of trans. reinf.	Max. spacing of trans. reinf.
		(psi)	(%)	(in.)	(in.)	(in.)	(%)	(in.)	(in.)
Aboutaha et al. 1999	SC3	3176	0.0	18.0	36.0	95.9	1.88	16.0	16.0
Aboutaha et al. 1999	SC9	2320	0.0	36.0	18.0	95.9	1.88	16.0	16.0
Bett et al. 1985	No. 1-1	4336	10.4	12.0	12.0	35.9	2.44	8.3	8.3
Imai and Yamamoto 1986	No. 1	3930	7.2	19.7	15.7	64.9	2.66	3.9	3.9
Kanda et al. 1988	85PDC-2	4046	10.6	9.8	9.8	59.0	1.62	2.0	2.0
Kanda et al. 1988	85PDC-3	4046	10.6	9.8	9.8	59.0	1.62	2.0	2.0
Kanda et al. 1988	85STC-1	4046	10.6	9.8	9.8	59.0	1.62	2.0	2.0
Kanda et al. 1988	85STC-2	4046	10.6	9.8	9.8	59.0	1.62	2.0	2.0
Kanda et al. 1988	85STC-3	4046	10.6	9.8	9.8	59.0	1.62	2.0	2.0
Lynn et al. 1996	2SLH18	4800	7.3	18.0	18.0	115.9	1.94	18.0	18.0
Lynn et al. 1996	3SLH18	3901	8.9	18.0	18.0	115.9	3.03	18.0	18.0
Lynn et al. 1998	2CLH18	4800	7.3	18.0	18.0	115.9	1.94	18.0	18.0
Lynn et al. 1998	3CLH18	3901	8.9	18.0	18.0	115.9	3.03	18.0	18.0
Mo and Wang 2000	C3-1	3828	10.7	15.7	15.7	110.1	2.14	2.1	2.1
Ono et al. 1989	CA025C	3741	25.7	7.9	7.9	23.6	2.13	2.8	2.8
Ono et al. 1989	CA060C	3741	61.6	7.9	7.9	23.6	2.13	2.8	2.8
Park and Paulay 1990	No. 9	3901	10.0	23.6	15.7	140.3	1.88	3.1	6.3
Pujol 2002	No. 10-1-2.25N	5293	7.8	12.0	6.0	53.9	2.45	2.2	2.2
Pujol 2002	No. 10-1-2.25S	5293	7.8	12.0	6.0	53.9	2.45	2.2	2.2
Pujol 2002	No. 10-2-2.25N	5061	8.2	12.0	6.0	53.9	2.45	2.2	2.2
Pujol 2002	No. 10-2-2.25S	5061	8.2	12.0	6.0	53.9	2.45	2.2	2.2
Pujol 2002	No. 10-3-1.5N	4655	8.9	12.0	6.0	53.9	2.45	1.5	1.5
Pujol 2002	No. 10-3-1.5S	4655	8.9	12.0	6.0	53.9	2.45	1.5	1.5
Pujol 2002	No. 10-3-2.25N	3973	10.4	12.0	6.0	53.9	2.45	2.2	2.2
Pujol 2002	No. 10-3-2.25S	3973	10.4	12.0	6.0	53.9	2.45	2.2	2.2
Sezen and Moehle	No. 1	3060	15.1	18.0	18.0	115.9	2.47	12.0	12.0
Sezen and Moehle	No. 2	3060	60.5	18.0	18.0	115.9	2.47	12.0	12.0
Sezen and Moehle	No. 4	3161	14.6	18.0	18.0	115.9	2.47	12.0	12.0
Soesianawati et al. 1986	No. 1	6743	10.0	15.7	15.7	125.8	1.51	3.3	6.7
Takemura and Kawashima, 1997	Test 1 (JSCE-4)	5206	2.7	15.7	15.7	97.9	1.58	2.8	2.8
Takemura and Kawashima, 1997	Test 2 (JSCE-5)	5177	2.7	15.7	15.7	97.9	1.58	2.8	2.8
Takemura and Kawashima, 1997	Test 3 (JSCE-6)	4974	2.9	15.7	15.7	97.9	1.58	2.8	2.8
Takemura and Kawashima, 1997	Test 4 (JSCE-7)	4814	3.0	15.7	15.7	97.9	1.58	2.8	2.8
Takemura and Kawashima, 1997	Test 5 (JSCE-8)	5336	2.7	15.7	15.7	97.9	1.58	2.8	2.8
Takemura and Kawashima, 1997	Test 6 (JSCE-9)	5206	2.7	15.7	15.7	97.9	1.58	2.8	2.8
Tanaka and Park 1990	No. 5	4640	10.0	21.6	21.6	129.8	1.25	4.3	8.7
Tanaka and Park 1990	No. 6	4640	10.0	21.6	21.6	129.8	1.25	4.3	8.7
Thomsen and Wallace 1994	A1	14892	0.0	6.0	6.0	46.9	2.45	1.0	1.5
Thomsen and Wallace 1994	B1	12688	0.0	6.0	6.0	46.9	2.45	1.0	1.5
Thomsen and Wallace 1994	B2	12093	10.0	6.0	6.0	46.9	2.45	1.0	1.5
Thomsen and Wallace 1994	C1	9788	0.0	6.0	6.0	46.9	2.45	1.0	1.5
Thomsen and Wallace 1994	C2	10817	10.0	6.0	6.0	46.9	2.45	1.0	1.5
Umehara and Jirsa 1982	CUW	5061	16.2	9.0	16.1	35.8	3.01	3.5	3.5
Wehbe et al. 1998	A1	3944	9.8	24.0	14.9	183.6	2.22	4.3	4.3
Wehbe et al. 1998	B1	4075	9.2	24.0	14.9	183.6	2.22	3.3	3.3
Xiao and Martirossyan 1998	HC4-8L16-T10-0.1P	12470	9.6	10.0	10.0	40.0	2.46	2.0	2.0
Xiao and Martirossyan 1998	HC4-8L19-T10-0.1P	11020	10.0	10.0	10.0	40.0	3.55	2.0	2.0
Zhou et al. 1987	No. 104-08	2871	80.1	6.3	6.3	12.6	2.22	1.6	1.6
Zhou et al. 1987	No. 114-08	2871	80.1	6.3	6.3	12.6	2.22	1.6	1.6
Zhou et al. 1987	No. 124-08	2871	80.1	6.3	6.3	12.6	2.22	1.6	1.6
Zhou et al. 1987	No. 204-08	3060	80.0	6.3	6.3	25.2	2.22	1.6	1.6
Zhou et al. 1987	No. 223-09	3060	90.0	6.3	6.3	25.2	2.22	1.6	1.6
Zhou et al. 1987	No. 302-07	4176	70.1	6.3	6.3	37.8	2.22	1.6	1.6
Zhou et al. 1987	No. 312-07	4176	70.1	6.3	6.3	37.8	2.22	1.6	1.6
Zhou et al. 1987	No. 322-07	4176	70.1	6.3	6.3	37.8	2.22	1.6	1.6

Table 15: Frame category, response mode and damage state data for column specimens

Cantilever Column Test		ACI Frame Category	ASCE Category	Failure Mode	Damage Data
Aboutaha et al. 1999	SC3	OMF Beam	4	Shear	C,0,3
Aboutaha et al. 1999	SC9	NC Beam	0	Shear	C,0,2,3
Bett et al. 1985	No. 1-1	OMF Col	5	Shear	C,0,2,3
Imai and Yamamoto 1986	No. 1	OMF Beam	4	Flex.-Shear	0.0
Kanda et al. 1988	85PDC-2	SMF	0	Flexure	0.0
Kanda et al. 1988	85PDC-3	SMF	0	Flexure	0.0
Kanda et al. 1988	85STC-1	SMF	0	Flexure	0.0
Kanda et al. 1988	85STC-2	SMF	0	Flexure	0.0
Kanda et al. 1988	85STC-3	SMF	0	Flexure	0.0
Lynn et al. 1996	2SLH18	NC Beam	3	Flex.-Shear	3
Lynn et al. 1996	3SLH18	NC Beam	3	Shear	0,3
Lynn et al. 1998	2CLH18	NC Beam	0	Flex.-Shear	3
Lynn et al. 1998	3CLH18	NC Beam	0	Shear	3
Mo and Wang 2000	C3-1	SMF	0	Flexure	0
Ono et al. 1989	CA025C	OMF Col	5	Flex.-Shear	C,0,2,3
Ono et al. 1989	CA060C	OMF Col	6	Flex.-Shear	C,0,2,3
Park and Paulay 1990	No. 9	SMF	0	Flexure	0,4
Pujol 2002	No. 10-1-2.25N	SMF	0	Flexure	0
Pujol 2002	No. 10-1-2.25S	SMF	0	Flexure	0
Pujol 2002	No. 10-2-2.25N	SMF	0	Flexure	0
Pujol 2002	No. 10-2-2.25S	SMF	0	Flexure	0
Pujol 2002	No. 10-3-1.5N	SMF	0	Flexure	0
Pujol 2002	No. 10-3-1.5S	SMF	0	Flexure	0
Pujol 2002	No. 10-3-2.25N	SMF	0	Flexure	0
Pujol 2002	No. 10-3-2.25S	SMF	0	Flexure	0
Sezen and Moehle	No. 1	OMF Col	5	Flex.-Shear	C,0,1,2,3
Sezen and Moehle	No. 2	OMF Col	6	Flex.-Shear	C,0,2,3
Sezen and Moehle	No. 4	OMF Col	5	Flex.-Shear	C,0,3
Soesianawati et al. 1986	No. 1	SMF	0	Flexure	0,2,3,4
Takemura and Kawashima, 1997	Test 1 (JSCE-4)	IMF	0	Flexure	0
Takemura and Kawashima, 1997	Test 2 (JSCE-5)	IMF	0	Flexure	0
Takemura and Kawashima, 1997	Test 3 (JSCE-6)	IMF	0	Flexure	0
Takemura and Kawashima, 1997	Test 4 (JSCE-7)	IMF	0	Flexure	0
Takemura and Kawashima, 1997	Test 5 (JSCE-8)	IMF	0	Flexure	0
Takemura and Kawashima, 1997	Test 6 (JSCE-9)	IMF	0	Flexure	0
Tanaka and Park 1990	No. 5	SMF	0	Flexure	0,2,3,4
Tanaka and Park 1990	No. 6	SMF	0	Flexure	0,2,3,4
Thomsen and Wallace 1994	A1	SMF	0	Flexure	0
Thomsen and Wallace 1994	B1	SMF	0	Flexure	0
Thomsen and Wallace 1994	B2	SMF	0	Flexure	0,2,3
Thomsen and Wallace 1994	C1	SMF	0	Flexure	0
Thomsen and Wallace 1994	C2	SMF	0	Flexure	0,2,3,
Umehara and Jirsa 1982	CUW	OMF Col	5	Shear	0,3
Wehbe et al. 1998	A1	SMF	0	Flexure	0,2,4
Wehbe et al. 1998	B1	SMF	0	Flexure	0,2,4
Xiao and Martirossyan 1998	HC4-8L16-T10-0.1P	SMF	0	Flexure	0,2,
Xiao and Martirossyan 1998	HC4-8L19-T10-0.1P	SMF	0	Flexure	0,2,4
Zhou et al. 1987	No. 104-08	OMF Col	6	Shear	C,0,3
Zhou et al. 1987	No. 114-08	OMF Col	6	Shear	C,0,3
Zhou et al. 1987	No. 124-08	OMF Col	6	Flex.-Shear	C,0,3
Zhou et al. 1987	No. 204-08	OMF Col	6	Flex.-Shear	C,0,3
Zhou et al. 1987	No. 223-09	OMF Col	6	Flex.-Shear	C,0,3
Zhou et al. 1987	No. 302-07	OMF Col	6	Flex.-Shear	C,0,3
Zhou et al. 1987	No. 312-07	OMF Col	6	Flex.-Shear	C,0,3
Zhou et al. 1987	No. 322-07	OMF Col	6	Flex.-Shear	C,0,3

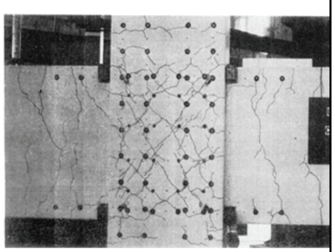
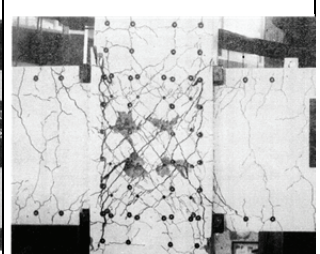
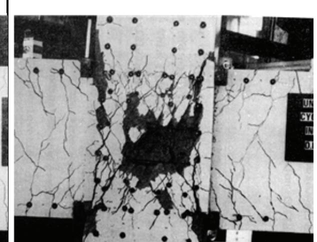
9 Appendix B – Fragility Function Specification

Fragility Specification

RC Frame (high ductility – ACI SMF)—B10**.00

Developer and Date:	Laura Lowes and Jingjuan Jane Li, August 2009
NISTIR Classification:	
Basic Composition:	Reinforced concrete and paint both sides
Normative Quantity (unit):	Frame volume and surface area
Demand Parameter:	Story drift
No. of Damage States (other than DS0-undamaged):	3
Type of Damage States (sequential, mutually exclusive, simultaneous, etc.):	Sequential

Damage States

	DS1	DS2	DS3
Description:	Beams, joints or columns exhibit residual crack widths > 0.06 in. No significant spalling No fracture or buckling of reinf	Beams, joints or columns exhibit residual crack widths > 0.06 in. Spalling of cover concrete exposes transverse reinforcement but not longitudinal reinforcement. No fracture or buckling of reinf	Beams, joints or columns exhibit residual crack widths > 0.06 in. Spalling of cover concrete exposes a significant length of longitudinal reinforcement. Crushing of core concrete may occur. No fracture or buckling of reinf
Illustrations:			

Fragility Parameters

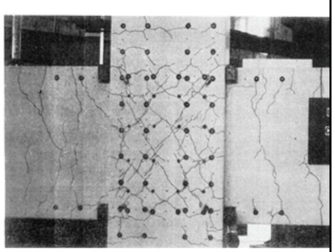
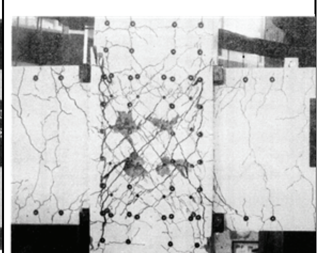
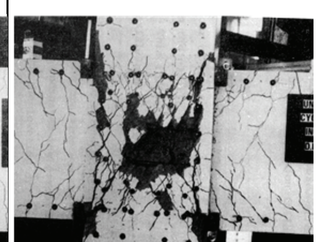
	DS1	DS2	DS3
Median Demand (θ):	2.0%	2.75	5.0%
Dispersion (β):	0.4	0.3	0.3
Probability (simultaneous DS):			
Correlation:			

Fragility Specification

RC Frame (high ductility – ASCE1)—B10**.00

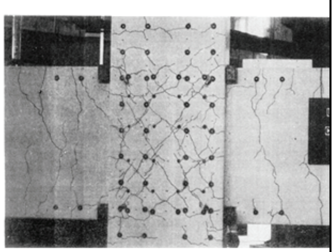
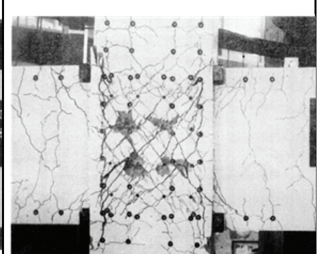
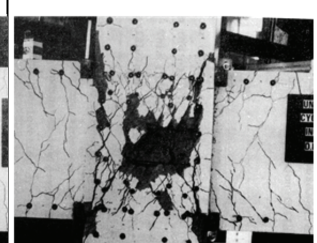
Developer and Date:	Laura Lowes and Jingjuan Jane Li, August 2009
NISTIR Classification:	
Basic Composition:	Reinforced concrete and paint both sides
Normative Quantity (unit):	Frame volume and surface area
Demand Parameter:	Story drift
No. of Damage States (other than DS0-undamaged):	3
Type of Damage States (sequential, mutually exclusive, simultaneous, etc.):	Sequential

Damage States

	DS1	**DS2**	**DS3**
Description:	Beams, joints or columns exhibit residual crack widths > 0.06 in. No significant spalling No fracture or buckling of reinf	Beams, joints or columns exhibit residual crack widths > 0.06 in. Spalling of cover concrete exposes transverse reinforcement but not longitudinal reinforcement. No fracture or buckling of reinf	Beams, joints or columns exhibit residual crack widths > 0.06 in. Spalling of cover concrete exposes a significant length of longitudinal reinforcement. Crushing of core concrete may occur. No fracture or buckling of reinf
Illustrations:			

Fragility Parameters

Median Demand (θ):	2.0%	2.5%	4.0%
Dispersion (β):	0.4	0.3	0.3
Probability (simultaneous DS):			
Correlation:			

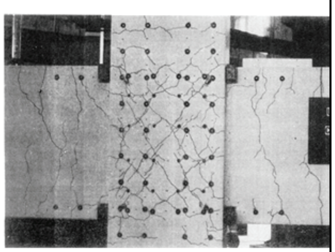
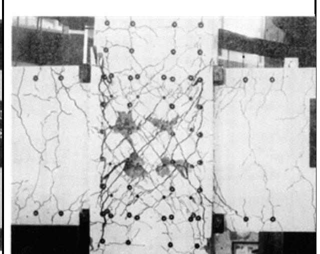
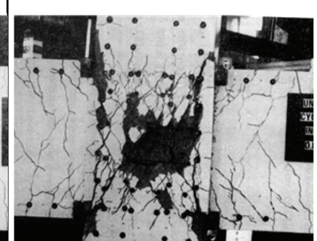
<table>
<tr><th colspan="4"><u>Fragility Specification</u>
RC Frame (moderate ductility – ACI IMF)—B10**.00</th></tr>
<tr><td>Developer and Date:</td><td colspan="3">Laura Lowes and Jingjuan Jane Li, August 2009</td></tr>
<tr><td>NISTIR Classification:</td><td colspan="3"></td></tr>
<tr><td>Basic Composition:</td><td colspan="3">Reinforced concrete and paint both sides</td></tr>
<tr><td>Normative Quantity (unit):</td><td colspan="3">Frame volume and surface area</td></tr>
<tr><td>Demand Parameter:</td><td colspan="3">Story drift</td></tr>
<tr><td>No. of Damage States (other than DS0-undamaged):</td><td colspan="3">3</td></tr>
<tr><td>Type of Damage States (sequential, mutually exclusive, simultaneous, etc.):</td><td colspan="3">Sequential</td></tr>
<tr><th colspan="4">Damage States</th></tr>
<tr><th></th><th>DS1</th><th>DS2</th><th>DS3</th></tr>
<tr><td>Description:</td><td>Beams, joints or columns exhibit residual crack widths > 0.06 in.
No significant spalling
No fracture or buckling of reinf</td><td>Beams, joints or columns exhibit residual crack widths > 0.06 in.
Spalling of cover concrete exposes transverse reinforcement but not longitudinal reinforcement.
No fracture or buckling of reinf</td><td>Beams, joints or columns exhibit residual crack widths > 0.06 in.
Spalling of cover concrete exposes a significant length of longitudinal reinforcement.
Crushing of core concrete may occur.
No fracture or buckling of reinf</td></tr>
<tr><td>Illustrations:</td><td></td><td></td><td></td></tr>
<tr><th colspan="4">Fragility Parameters</th></tr>
<tr><td>Median Demand (θ):</td><td>2.0%</td><td>2.5%</td><td>3.5%</td></tr>
<tr><td>Dispersion (β):</td><td>0.4</td><td>0.3</td><td>0.4</td></tr>
<tr><td>Probability (simultaneous DS):</td><td></td><td></td><td></td></tr>
<tr><td>Correlation:</td><td></td><td></td><td></td></tr>
</table>

Fragility Specification

RC Frame (moderate ductility – ACI OMF / ASCE2)—B10**.00

Developer and Date:	Laura Lowes and Jingjuan Jane Li, August 2009
NISTIR Classification:	
Basic Composition:	Reinforced concrete and paint both sides
Normative Quantity (unit):	Frame volume and surface area
Demand Parameter:	Story drift
No. of Damage States (other than DS0-undamaged):	3
Type of Damage States (sequential, mutually exclusive, simultaneous, etc.):	Sequential

Damage States

	DS1	DS2	DS3
Description:	Beams, joints or columns exhibit residual crack widths > 0.06 in. No significant spalling No fracture or buckling of reinf	Beams, joints or columns exhibit residual crack widths > 0.06 in. Spalling of cover concrete exposes transverse reinforcement but not longitudinal reinforcement. No fracture or buckling of reinf	Beams, joints or columns exhibit residual crack widths > 0.06 in. Spalling of cover concrete exposes a significant length of longitudinal reinforcement. Crushing of core concrete may occur. No fracture or buckling of reinf
Illustrations:			

Fragility Parameters

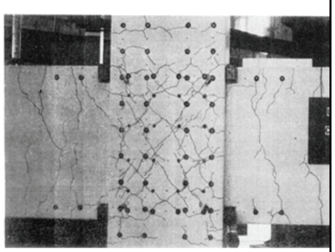
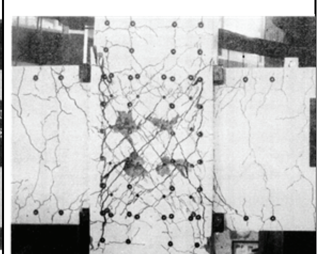
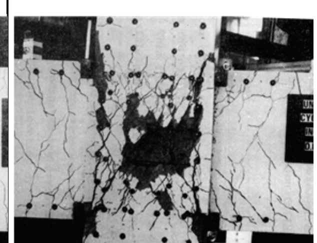
	DS1	DS2	DS3
Median Demand (θ):	1.75%	2.25%	3.5%
Dispersion (β):	0.4	0.3	0.4
Probability (simultaneous DS):			
Correlation:			

Fragility Specification

RC Frame (low ductility – ASCE4/5)—B10**.00

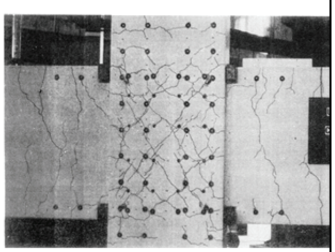
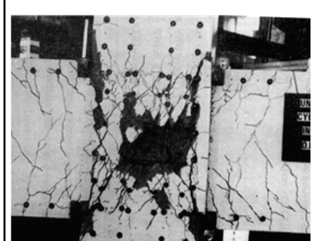
Developer and Date:	Laura Lowes and Jingjuan Jane Li, August 2009
NISTIR Classification:	
Basic Composition:	Reinforced concrete and paint both sides
Normative Quantity (unit):	Frame volume and surface area
Demand Parameter:	Story drift
No. of Damage States (other than DS0-undamaged):	3
Type of Damage States (sequential, mutually exclusive, simultaneous, etc.):	Sequential

Damage States

	DS1	DS2	DS3
Description:	Beams, joints or columns exhibit residual crack widths > 0.06 in. No significant spalling No fracture or buckling of reinf	Beams, joints or columns exhibit residual crack widths > 0.06 in. Spalling of cover concrete exposes transverse reinforcement but not longitudinal reinforcement. No fracture or buckling of reinf	Beams, joints or columns exhibit residual crack widths > 0.06 in. Spalling of cover concrete exposes a significant length of longitudinal reinforcement. Crushing of core concrete may occur. No fracture or buckling of reinf
Illustrations:			

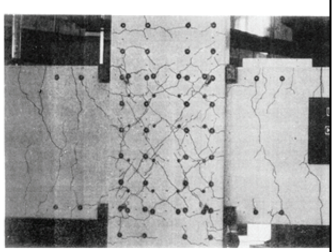
Fragility Parameters

Median Demand (θ):	1.5%	2.0%	2.5%
Dispersion (β):	0.4	0.3	0.4
Probability (simultaneous DS):			
Correlation:			

Fragility Specification RC Frame (low ductility – ACI NCF / ASCE3)—B10**.00			
Developer and Date:	Laura Lowes and Jingjuan Jane Li, August 2009		
NISTIR Classification:			
Basic Composition:	Reinforced concrete and paint both sides		
Normative Quantity (unit):	Frame volume and surface area		
Demand Parameter:	Story drift		
No. of Damage States (other than DS0-undamaged):	2		
Type of Damage States (sequential, mutually exclusive, simultaneous, etc.):	Sequential		
Damage States			
	DS1		**DS3**
Description:	Beams, joints or columns exhibit residual crack widths > 0.06 in. No significant spalling No fracture or buckling of reinf		Beams, joints or columns exhibit residual crack widths > 0.06 in. Spalling of cover concrete exposes a significant length of longitudinal reinforcement. Crushing of core concrete may occur. No fracture or buckling of reinf
Illustrations:			
Fragility Parameters			
Median Demand (θ):	1.5%		2.0%
Dispersion (β):	0.4		0.4
Probability (simultaneous DS):			
Correlation:			

Fragility Specification RC Frame (no ductility – ACI NCF / ASCE3)—B10**.00	
Developer and Date:	Laura Lowes and Jingjuan Jane Li, August 2009
NISTIR Classification:	
Basic Composition:	Reinforced concrete and paint both sides
Normative Quantity (unit):	Frame volume and surface area
Demand Parameter:	Story drift
No. of Damage States (other than DS0-undamaged):	2
Type of Damage States (sequential, mutually exclusive, simultaneous, etc.):	Sequential

Damage States

	DS1		DS3
Description:	Beams, joints or columns exhibit residual crack widths > 0.06 in. No significant spalling No fracture or buckling of reinf		Beams, joints or columns exhibit residual crack widths > 0.06 in. Spalling of cover concrete exposes a significant length of longitudinal reinforcement. Crushing of core concrete may occur. No fracture or buckling of reinf
Illustrations:			

Fragility Parameters

Median Demand (θ):	0.25%		0.5%
Dispersion (β):	0.4		0.5
Probability (simultaneous DS):			
Correlation:			