論文 / 著書情報
Article / Book Information

論題(和文)	
Title(English)	Experimental fragility analysis of non-structural rc walls
著者(和文)	Rajbhandari Priyana, 小原 拓, 河野 進
Authors(English)	Priyana Rajbhandari, Taku Obara, Susumu Kono
出典(和文)	コンクリート工学年次論文集, Vol. 44, No. 2, pp. 187-192
Citation(English)	Proceedings of the Japan Concrete Institute, Vol. 44, No. 2, pp. 187-192
発行日 / Pub. date	2022, 7

- Technical Paper -

EXPERIMENTAL FRAGILITY ANALYSIS OF NON-STRUCTURAL RC WALLS

Priyana RAJBHANDARI[*1], Taku OBARA[*2], Susumu KONO[*3]

ABSTRACT

Experimental fragility parameters are computed for the non-structural reinforced concrete walls corresponding to the limit states defined by the Architectural Institute of Japan to assist in performance-based earthquake engineering using the past experimental results. The calculated fragility parameters are compared with those in the FEMA fragility specification. It is concluded that the fragility curves should be associated with clear definition of damage state or limit state, and the one obtained from experimental data should represent the population.

Keywords: experimental fragility, non-structural RC walls, reinforced concrete, PBEE

1. INTRODUCTION

Most of the building design practice around the world is such that the structural components are designed for life safety, and the non-structural components like infills and partition walls are chosen without giving much thought about the seismic performance. The popular choice for infill in reinforced concrete (RC) buildings is unreinforced masonry in many countries [1]. However, in Japan, lightly reinforced infill walls with openings are constructed monolithically with RC moment-resisting frames. The 2011 Tohoku earthquake and 2016 Kumamoto earthquake have shown that even if there is minor or no damage in structural components, damage to non-structural walls can cause severe dysfunction, sometimes leading to demolition [2].

The damage incurred by non-structural RC walls may not be life-threatening, like damage to structural components, but it hinders the functionality of the building and requires repair of some kind to be reinstated. It is observed that even though life safety has been achieved in the design, not much consideration has been put upon the resilience and damage states which can develop prior to collapse. Resilience and functionality can be targeted through performance objectives that precede collapse by means of non-prescriptive Performance-Based Earthquake Engineering (PBEE) framework. PBEE entails the design, evaluation, and construction of the facilities whose performance under common and extreme loads responds to the diverse needs and objectives of the stakeholders [3]. The next generation PBEE framework was developed by the Pacific Earthquake Engineering Research Center (PEER) in 1997, which divides the methodology into four primary steps: hazard analysis, structural analysis, damage analysis, and loss analysis, as described in Figure 1.

A key element of the framework is fragility functions for the components and elements of the seismic framing system. One such function is required for each component type. Component fragility denotes 'damage analysis' and relates the probability of exceeding one or more damage states to probable demand calculated from the structural model. This step enables the transition of mathematical structural response quantities to physical damage terms that are better understood by stakeholders

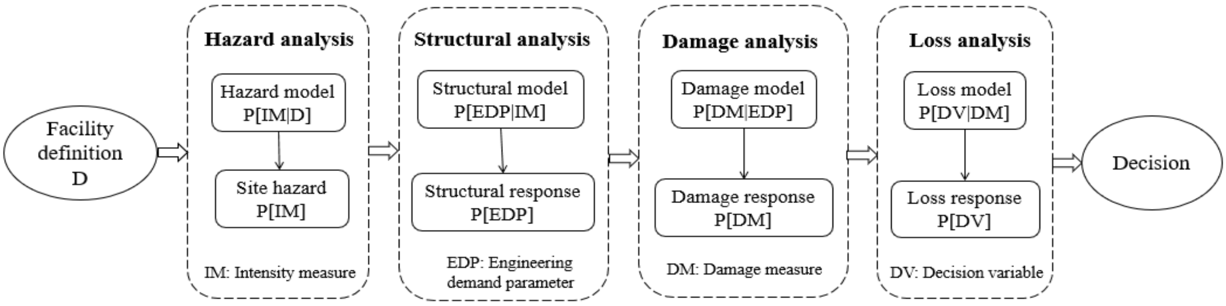

Fig. 1: General framework of PEER PBEE methodology (after Porter 2003 [22])

[*1] Dept. of Architecture and Building Engineering, Tokyo Institute of Technology, JCI Student Member

[*2] Assistant Professor, Tokyo Institute of Technology, JCI Member

[*3] Professor, Tokyo Institute of Technology, JCI Member

and can be subsequently quantified in terms of losses [4]. Damage states are typically characterized using descriptors of physical damage of the components that can be used to compute the repair effort required to restore the component to an undamaged state. Such descriptors can be crack width, spalling area of concrete, reinforcement yielding and buckling, etc.

Many studies have investigated the collapse fragility of building components primarily based on incremental dynamic analysis as per FEMA P-695 [5] methodology. There are few studies that focus on the fragility of RC wall damage states that precede collapse [4], [6], [7], [8] and even fewer studies on the fragility of non-structural RC walls [9], [10]. To the best of one's knowledge, there has been no research on the component fragility of non-structural RC walls for limit states defined in the 2004 AIJ draft Guidelines [11]. As an attempt to provide information about the importance of component fragility in the PBEE framework, this paper discusses a general methodology to compute the experimental component fragility through an example of non-structural walls. Since all the information leading to specific limit states are essential, six non-structural RC walls previously tested by the co-authors [12], [13] are used in the study. Fragility parameters are obtained for the non-structural RC walls based on the experiments on six walls.

2. SUMMARY OF THE TESTED SPECIMENS

Full-scale experiment was conducted on eight non-structural RC walls. The detailed experimental program and principal findings are reported in Yuniarsyah et al. [12], and Tani et al. [13]. Yuniarsyah et al. [12] studied the experimental seismic behavior of lightly reinforced concrete walls, and Tani et al. [13] carried out experiments on RC non-structural walls with a focus on damage reduction and seismic behavior improvement. Six walls are chosen in this study as non-structural RC walls. Geometry and loading details are shown in Table 1. One specimen was loaded under quasi-static cyclic loading with zero axial load, and the rest five were loaded with constant compressive axial load in addition to quasi-static cyclic loading.

Table 1: Specimen details

Specimen name	NSW1	NSW2	NSW3	NSW4	NSW5	NSW6
Thickness (mm)	120					200
Length (mm)	1050					900
Height (mm)	2100					1800
Shear span ratio	1			2	1	1
Vertical rebar at end region	2-D13 (SD345)				4-D13 (SD295A)	4-D13 (SD345)
Vertical bars	D10@250 (SD295)					D10@200 double (SD295A)
Horizontal bars	D10@250 single (ρ_{wh}=0.24%)		D10@125 single (ρ_{wh}=0.48%)		D10@60 single (ρ_{wh}=0.99%)	D10@100 double (ρ_{wh}=0.71%)
Concrete strength (N/mm^2)	24.2				22.2	
Axial load ratio	0	0.15			0.14	

3. DAMAGE EVALUATION

Damage to non-structural walls is usually associated with repair function and hence member damage level. The 2004 AIJ Draft Guidelines [11] has defined member damage level in four stages and associated these damage levels to four limit states. The limit states are laid down for the damage evaluation of earthquake-resistant RC buildings. They are defined based on the condition of steel bar and concrete. The focus of the present study is the damage analysis of non-structural walls for the damage states leading to collapse. So, three limit states: serviceability, repairability I, and repairability II are considered. Table 2 shows the formal definition of these limit states as per the 2004 AIJ Guidelines for structural members. The relationship between hysteretic characteristics of members and limit states are set for defining the limit states in Table 2. Since the limit states are originally decided based on repairability of members and since the purpose of this study is to develop fragility curve for loss estimation using repair functions, damage evaluation primarily based on the definition given in Table 2 is used.

Since the criteria for each limit state is not defined in terms of the measured quantity, in order to remove subjectivity from damage evaluation, Obara et al. [14] simplified the limit state definition used in the 2015 AIJ Draft Guidelines for Prestressed concrete flexural members [15] to define the quantitative criteria for each limit state as shown in Table 3. Egawa et al. [16] modified the criteria (of Table 3) to apply it for the RC members and carried out the drift-damage evaluation for

Table 2: Relation between limit states and member damage (After 2004 AIJ Guidelines [11])

Limit state	Damage state		Residual crack width (mm)	Required repair work
	Longitudinal reinforcement	Concrete		
Serviceability	Elastic	Nearly elastic	≤0.2	no repair
Repairability I	Slightly yielded	In good condition	0.2-1	small repair work
Repairability II	No buckling	Healthy core	1-2	extensive repair work

Table 3: Quantification of AIJ Limit states (interpreted by Obara et al.^{*1} [14])

Limit state	Damage state			
	Longitudinal reinforcement	Concrete	Residual crack width	Residual drift
Serviceability	minor yielding is allowed	less than 0.9fc'	<0.2mm	<0.10%
Repairability I	yielding is allowed to 1%	minor cracks	<1.0mm	<0. 25%
Repairability II	buckling	cover spalling	<2.0mm	<0.50%

fc': compressive strength of concrete

^{*1}: only the criteria relevant to the present study are shown

Table 4: Drift (%) reaching four criteria for three limit states in the 2004 AIJ Guidelines
(After Egawa et al. [16])

	Criteria	**NSW1**	**NSW2**	**NSW3**	**NSW4**	**NSW5**	**NSW6**
Serviceability Limit state (S LS)	$\varepsilon_{long.} \leq \varepsilon_y$	0.16	0.19	0.17	0.27	0.11	0.23
	$\varepsilon_{cv} \leq$ strain at $2/3f_c$	0.06	0.09	0.10	0.09	0.03	0.03
	$W_{re} \leq 0.2$mm	0.25	0.25	0.50	0.75	0.75	0.50
	$R_{re} \leq 0.1\%$	0.25	0.50	0.75	1.50	0.50	0.73
Drift triggering S LS		**0.06**	**0.09**	**0.10**	**0.09**	**0.03**	**0.03**
Repairability I Limit state (RI LS)	$\varepsilon_{long.} \leq 1\%$	0.17	0.25	0.35	0.42	0.46	0.44
	Vertical cracking	0.75	1.00	0.50	1.50	0.50	0.50
	$0.2 < W_{re} \leq 1$mm	0.50	0.50	1.50	-	1.50	1.50
	$0.1\% < R_{re} \leq 0.25\%$	0.50	1.00	1.51	2.00	1.00	1.50
Drift triggering RI LS		**0.17**	**0.25**	**0.35**	**0.42**	**0.46**	**0.44**
Repairability II Limit state (RII LS)	Buckling is not allowed	-	1.00	-	2.00	2.00	3.00
	Concrete cover may spall, but core is healthy	1.50	1.00	1.00	2.00	1.00	1.50
	$1 < W_{re} \leq 2$mm	0.75	0.75	-	-	-	-
	$0.25 < R_{re} \leq 0.5\%$	0.75	-	-	2.00	1.50	2.00
Drift triggering RII LS		**0.75**	**0.75**	**1.00**	**2.00**	**1.00**	**1.50**

$\varepsilon_{long.}$: strain in longitudinal reinforcement, ε_y: yield strain of reinforcement, ε_{cv}: strain in cover concrete
f_c: compressive strength of concrete, W_{re}: residual crack width, R_{re}: residual drift

six non-structural RC walls considered in the present study for three limit states. As non-structural walls can be categorized as something that requires significant repair cost, drift is chosen as engineering demand parameter (EDP) to obtain fragility parameters in this study [17]. The drift values considered in the present study and governing drift ratio for each limit state are shown in Table 4. It should be noted that the governing drift ratio is taken as the minimum drift ratio that triggers any of the four criteria of a given limit state.

4. EXPERIMENTAL FRAGILITY FUNCTIONS

Fragility functions are developed to quantify the probability of exceedance or occurrence of a specific AIJ limit state as a function of peak drift. Fragility parameters are calculated for the tested walls and compared with fragility parameters established in the FEMA P-58 [18] for non-structural walls.

4.1. For tested walls

Based on the method proposed by Porter et al. [19], experimental fragility curves are drawn for the non-structural RC walls. The fragility function is defined by lognormal cumulative probability distribution (CDF) which is described by two parameters: CDF median (θ) and associated lognormal standard deviation (β). The mathematical expression for fragility function is:

$$F_{EDP}(x) = \phi\left(\frac{\ln\left(\frac{x}{\theta}\right)}{\beta}\right) \tag{1}$$

where,

$\phi()$: standard normal CDF

The values of θ and β are computed in this study using the following formula [10]:

$$\theta = \exp\left(\frac{1}{N}\sum_{i=1}^{N} ln(EDP_i)\right) \quad (2)$$

$$\beta_r = \sqrt{\frac{1}{M-1}\sum_{i=1}^{M}\left(\ln\left(\frac{d_i}{\theta}\right)\right)^2} \quad (3)$$

$$\beta = \sqrt{\beta_r^2 + \beta_u^2} \quad (4)$$

where,

M : total number of observations
i : specimen index
d_i : EDP value for observation i
β_r : random variability in test data
β_u : uncertainty that the test represents actual conditions of installation, loading sample size.

Since the tests meet at least one of the criteria mentioned in Porter et al. [19],

$$\beta_u = 0.25 \quad (5)$$

In order to remove the spurious values of demand, Pierce's criterion [20] is used to test and eliminate the outliers. The computed fragility parameters for non-structural RC walls shown in Table 5.

Table 5: Fragility parameters corresponding to the 2004 AIJ limit states

	θ	β
Serviceability LS	0.08	0.35
Repairability I LS	0.37	0.36
Repairability II LS	0.97	0.38

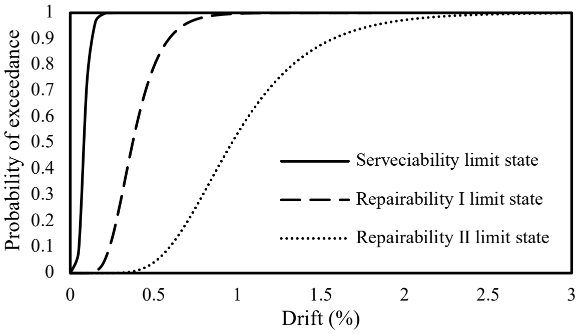

Fig. 2: Fragility curve for non-structural RC walls using six specimens

Using these parameters, fragility curves are drawn for the three AIJ limit states as shown in Figure 2. To evaluate the normality of given data, the developed fragility parameters are tested for Lilliefors goodness-of-fit test [21] and are deemed acceptable at 5% significance level.

4.2. Comparison to FEMA P-58 fragility specification

FEMA P-58 has given the values of fragility parameters for three different damage states. The description of the damage states is given in Table 6. Value of median drift ratio and associated lognormal standard deviation for non-structural RC walls established in the FEMA P-58 Fragility Specification is given in Table 7. Figure 3 shows the fragility curve for given parameters.

Table 6: Definition of damage states for non-structural RC wall in FEMA P-58

DS1	**DS2**	**DS3**
1mm≤W_{cr}≤3mm	crushed core concrete	sliding of wall
	buckling of vertical rebar	fracture of rebar
	localized crack, W_{cr}>3mm	distributed cracking, W_{cr}>3mm

W_{cr}: maximum crack width

Table 7: Fragility parameters for non-structural RC walls defined in FEMA P-58

	θ	β
DS1	0.55	0.36
DS2	1.09	0.30
DS3	1.30	0.36

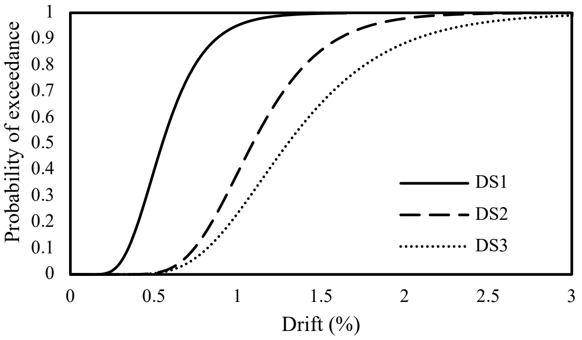

Fig. 3: Fragility curve for non-structural RC walls based on FEMA P-58

It can be inferred from the FEMA fragility specification that the damage states DS1, DS2 and DS3 require minor, intermediate, and major repair respectively. Based on required repair work (Ref. Table 2), Repairability I LS and Repairability II LS are comparable to DS1 and DS2 respectively. On comparing the median drift for both the systems, FEMA observed a much higher drift for all the damage states. From the definition of damage state from Table 6, it is evident that these damage states are different from the AIJ limit states. This signifies that the fragility parameters are determined based on the definition of damage states and it should therefore only be established with the coherent definition of the limit states or damage states. While this discrepancy is mainly attributed to the different characterization of damage states, there can be difference in fragility parameters even if the damage states are identical. Some factors that contribute to the discrepancies include experimental setup, loading protocol, specimen configurations, and uncertainties associated with material properties.

5. DISCUSSIONS

In the damage evaluation for the walls in Table 4, it is observed that the damage to concrete and longitudinal reinforcement dictates the drift that triggers serviceability and repairability I limit state, respectively. For repairability II limit state, residual crack width and damage to concrete were the controlling criteria.

By the definition of AIJ limit states, it appears that the demand parameter (drift ratio in this case) for all the criteria should not have significant deviation. For example, longitudinal reinforcement is expected to yield when the residual crack width in concrete is around 0.2mm (Ref. Serviceability limit state in Table 2). However, in the damage evaluation of serviceability limit state, it is noticed that the drift ratio for concrete criterion ($\varepsilon_{cv} \leq strain\ at\ 2/3f_c$) is much smaller than the drift ratio for other criteria. The reason for this variation might be the quantification of concrete criterion 'nearly elastic' (Ref. Table 2) as $\varepsilon_{cv} \leq strain\ at\ 2/3f_c$ (Ref. Table 4) or the definition itself in the 2004 AIJ Guidelines.

The median drift at which the non-structural RC wall exceeds serviceability, repairability I and repairability II limit states defined by 2004 AIJ Guidelines comes out to be 0.08%, 0.37%, and 0.97% respectively. In practical design in Japan, the inter-story drift for the safety limit state is 0.5% for shear wall buildings and 1% for moment-resisting frame buildings. Even though the non-structural walls are used in the framed buildings, the median drift for repairability II limit state (0.97%) is very large. If the AIJ limit states can be associated with a thorough description of repair and loss functions, fragility function for the limit states can be used in the PBEE framework. Even though fragility functions are not helpful in understanding the phenomenon that leads to damage, it assists the practitioners in accessing the performance of building component and ultimately the building.

Drift-based fragility function provides information about the probability of exceeding (or experiencing) a specific limit state as a function of peak drift ratio experienced by the wall, which is crucial in PBEE. However, it should be noted that the results are based on just six experiments that do not represent the whole population. Fragility analysis using the extensive experimental database is needed to derive the well-founded fragility parameters.

6. CONCLUSION

Drift-based fragility parameters for non-structural RC walls are computed based on observations of quasi-static cyclic loading test on six walls. The parameters are calculated for three limit states preceding life safety limit state defined by the 2004 AIJ Guidelines. With the clear description of limit states, supplemented with loss and repair functions, experimental fragility curves constructed with a database representing the population can be considered as an important tool in performance evaluation, loss estimation, and overall performance-based design.

REFERENCES

[1] S. H. Basha and H. B. Kaushik, "Behavior and failure mechanisms of masonry-infilled RC frames (in low-rise buildings) subject to lateral loading," *Eng. Struct.*, Vol. 111, pp. 233–245, 2016, doi: 10.1016/j.engstruct.2015.12.034.

[2] I. Nishiyama, I. Okawa, H. Fukuyama, and Y. Okuda, "Building damage by the 2011 off the Pacific coast ofTohoku earthquake and coping activities by NILIM and BRI collaborated with the administration," Vol. 25, No. 2, pp. 134–138, 2011.

[3] V. E. Saouma and M. A. Hariri-Ardebili, *Aging, Shaking, and Cracking of Infrastructures*. Springer Nature, 2021.

[4] A. Shegay, "Seismic Performance of Reinforced Concrete Walls Designed For Ductility," PhD Dissertation, ResearchSpace@Auckland, 2019.

[5] Applied Technology Council and United States. Federal Emergency Management Agency, *Quantification of Building Seismic Performance Factors*. US Department of Homeland Security, FEMA, 2009.

[6] A. C. Birely, "Seismic performance of slender reinforced concrete structural walls," PhD Dissertation,University of Washington, 2012.

[7] C. K. Gulec and A. S. Whittaker, *Performance-Based Assessment and Design of Squat Reinforced Concrete Shear Walls*. State University of New York at Buffalo, MCEER Report, 2009.

[8] P. C. Brown, "Probabilistic Earthquake Damage Prediction for Reinforced Concrete Building Components," Doctoral dissertation, University of Washington, 2008.

[9] H.-M. MR, *Multi-hazard loss estimation methodology: Earthquake model*. Department of Homeland Security, FEMA, Washington, DC, 2012.

[10] Applied Technology Council and United States. Federal Emergency Management Agency, *Seismic Performance Assessment of Buildings, Volume 1 - Methodology*. US Department of Homeland Security, FEMA, 2018.

[11] AIJ, "Guidelines for performance evaluation of earthquake resistant reinforced concrete buildings (Draft)," *(In Japanese)*, 2004.

[12] E. Yuniarsyah, S. Kono, M. Tani, R. Taleb, K. Sugimoto, and T. Mukai, "Damage evaluation of lightly reinforced concrete walls in moment resisting frames under seismic loading," *Eng. Struct.*, Vol. 132, pp. 349–371, 2017, doi: 10.1016/j.engstruct.2016.11.047.

[13] M. Tani, E. Yuniarsyah, T. Mukai, and S. Kono, "Full scale experiment of RC non-structural walls focused on damage reduction and seismic behavior improvement," Proceedings of the Japan Concrete Institute, 37(2) 901–906 (in Japanese), 2015.

[14] T. Obara, S. Kono, D. Mukai, "Damage Evaluation and Limit States of Rocking Concrete Walls with Energy Dissipaters", Journal of Earthquake Engineering, 1-27, 2020, doi: 10.1080/13632469.2020.1835748

[15] Architectural Institute of Japan, *Draft of guidelines for structural design and construction of prestressed concrete building based on performance evaluation concept*. 2015.

[16] M.Egawa et al., "Each Limit State of Reinforced Concrete Lightly Walls Based on Damage Evaluation Part1: Experimental results and damage evaluation," Summary of Technical Papers of Annual Meeting, Vol. IV, 465-468, Architectural Institute of Japan, 2018.

[17] D. Bonowitz *et al.*, "ATC-58 Project Task Report Engineering Demand Parameters for Nonstructural Components," 2004.

[18] F. P-58-1, *Seismic Performance Assessment of Buildings*, Vol. 1, No. December. 2018.

[19] K. Porter, R. Kennedy, and R. Bachman, "Creating fragility functions for performance-based earthquake engineering," *Earthq. Spectra*, Vol. 23, No. 2, pp. 471–489, 2007, doi: 10.1193/1.2720892.

[20] S. M. Ross, "Peirce's criterion for the elimination of suspect experimental data," *J. Eng. Technol.*, Vol. 20, No. 2, pp. 38–41, 2003.

[21] H. W. Lilliefors, "On the Kolmogorov-Smirnov Test for Normality with Mean and Variance Unknown," *J. Am. Stat.*, Vol. 62, No. 318, pp. 399–402, 1967.

[22] K. Porter, "An Overview of PEER's Performance-Based Earthquake Engineering Methodology," *Proc. ninth Int. Conf. Appl. Stat. Probab. Civ. Eng.*, Vol. 52, No. 12, pp. 3729–3735, 2009.