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FRAGILITY ANALYSIS OF NON-STRUCTURAL REINFORCED CONCRETE WALLS

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Abstract: *Lightly reinforced concrete (RC) walls are the popular choice for infill in the moment-resisting framed buildings in Japan. Although not intended for seismic load resistance nor designed, some of these walls resist the seismic load, resulting in damage of some kind. Past earthquakes have shown that even though the structural elements are intact, the non-structural (NS) RC walls can be severely damaged, causing building dysfunctions. Component fragility curves are critical elements in Performance-Based Earthquake Engineering that can be used to target the resilience and functionality of the building. Due to the limited availability of extensive experimental results relating to the damage states of the NS RC walls, numerical simulation of the global and local response is deemed necessary. In this study, a two-dimensional fiber-based macroscopic model capable of representing coupled shear-flexure interaction under cyclic loading conditions is selected to obtain a reliable prediction of the inelastic behavior of NS RC walls. The numerical model is validated and calibrated against the available experimental results obtained from four full-scale NS RC wall specimens characterized with different shear span ratios, axial load levels, reinforcement detailing, and wall geometry tested under reversed cyclic lateral loading. The numerical tool is then used in the damage analysis of the dataset of models representing the non-structural RC walls used in Japan. Four damage limit states are characterized based on the damage progression observed in the experiments. The onset of each damage limit state is defined by strain-based criterion. Using the analysis results, fragility curves describing the probability of experiencing or exceeding a particular damage state as a function of the experienced drift ratio are drawn. The proposed drift-based fragility curves can be used in the performance evaluation and seismic loss assessment in performance-based earthquake engineering of buildings.*

1. Introduction

The preferred option for infill walls in reinforced concrete (RC) buildings in Japan is lightly RC walls. Since they are not designed to withstand either gravity or lateral loads, they are categorized as non-structural components and will be referred to as non-structural (NS) walls. In practice, the NS walls are typically cast monolithically with the surrounding frame and are confined within a single floor. The aftermath of the 2011 Tohoku earthquake and the 2016 Kumamoto earthquake has shown that even if the structural components are intact, damage to NS walls can lead to severe building dysfunctions, impairing functionality and escalating the repair cost (I. Nishiyama et al., 2011; M. Nishiyama et al., 2016). It has been observed that while design efforts have successfully achieved life safety, less emphasis has been placed on addressing the resilience and potential damage states that might develop before the collapse.

In the past two decades, seismic design provisions in several design codes have been incorporating performance-based earthquake engineering (PBEE), where the structures are designed to achieve multiple performance levels when subjected to different levels of seismic hazards. The concept of PBEE has its roots in the realization that the amount of damage, repair cost of structures, and economic loss due to downtime is unacceptably high even though the structures complied with available seismic codes based on traditional design philosophy (Lee & Mosalam, 2006). Vision (1995) report, ATC-40 (1996), FEMA-273 (SEMINAR, 1997), and FEMA 356 (2000) are some early documents of the first-generation PBEE in the US. These procedures theorized the building as being loaded by earthquake-induced lateral forces that result in non-linear response and damage. The structural response parameters are then related to the discrete performance-oriented descriptions. The first-generation PBEE does not directly address the control of economic loss, focuses on assessing the performance of individual building components, rather than the building as a whole, and the element performance evaluation is deterministic (Hamburger, 2004). Considering these shortcomings, a more robust PBEE methodology was developed in the Pacific Earthquake Engineering Research (PEER) Center (Porter, 2003). This next-generation PBEE methodology provides a probabilistic assessment framework to explicitly quantify performance matrices, such as monetary losses, downtime, and casualties, meaningful to decision-makers, stakeholders, and insurers. A key component of the new framework is fragility functions for the building components. These functions relate the probability of exceeding one or more damage thresholds to an engineering demand parameter. Fragility functions enable the transition of mathematical structural response to physical damage terms better understood by the decision-makers and can be subsequently quantified in terms of losses (Shegay, 2019).

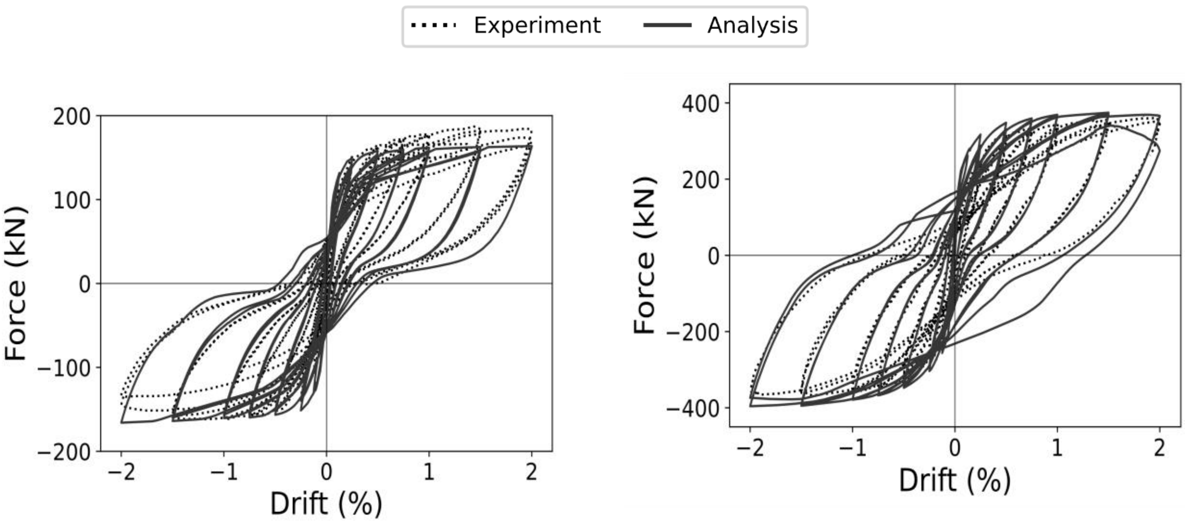
Numerous studies have investigated the collapse fragility of building components primarily based on incremental dynamic analysis as per FEMA P-695 (ATC, 2009) methodology. Few studies focus on the fragility of RC wall damage states that precede collapse (Birely, 2012; Brown, 2008; Gulec & Whittaker, 2009; Shegay, 2019). To the best of authors' knowledge, there has been no research on the component fragility of the RC NS walls. This paper aims to contribute to the PBEE methodology and loss estimation by addressing the development of fragility curves for NS walls, which are prevalent in construction practices in Japan. All the damage information of the component leading to a particular damage state is crucial in defining fragility functions. Since the extensive experimental results and the damage data of NS walls are limited, it is necessary to rely on numerical simulations to produce the data required for developing fragility functions. In this paper Shear-Flexure Interaction-Multiple Vertical Line Element Model (available in OpenSees (Mazzoni et al., 2006) as SFI MVLEM) is used to analyze samples. This model has been validated for simulating the local and global responses of NS walls typically used in Japan. The following sections explain the numerical model description and validation, repair method and damage levels, summarize the data used to establish the fragility curves, and present the statistical analysis of the data and the fragility functions.

2. Numerical model description and validation

Given that NS walls typically exhibit a low shear span-to-wall length ratio and are characterized by a low reinforcement ratio, an interaction exists between the shear and flexural response. One method for analyzing such walls involves using the cyclic shear-flexure interaction model for RC walls, referred to as Shear-Flexure Interaction-Multiple Vertical Line Element Model (SFI MVLEM) (Kolozvari, 2013). It uses the two-dimensional fiber-based macroscopic representation of RC panel to couple the axial and shear responses and has been widely used and validated through numerous experimental results (Kolozvari et al. 2015, Faraone et al. 2022). This paper adopts SFI MVLEM for analysis.

The non-linear SFI MVLEM consists of RC panels subjected to in-plane actions integrated into a two-dimensional macroscopic fiber model formulated by Orakcal et al. (2004). The behavior of the RC panel is described by the modified formulation of the Fixed-Strut-Angle-Model (FSAM) (Orakcal et al., 2019), which includes dowel action of reinforcement and shear aggregate interlock. The concrete behavior under biaxial loading is modeled with a concrete model (ConcreteCM) that modifies the Chang and Mander formulation (1994) to include compression softening, hysteretic biaxial damage, and tension stiffening effects. The uniaxial stress-strain relationship for the reinforcing steel is represented by the non-linear hysteretic model of Menegotto and Pinto (1973) extended by Filippou et al. (1983) (SteelMPF) to include strain hardening.

Two full-scale NS RC wall specimens, tested by Yuniarsyah et al. (2018) were used for the numerical model validation. Both the specimens were tested under constant axial load and reversed cyclic lateral load conditions. The specimens were modeled with a vertical stack of eight SFI MVLEM elements, with seven RC panels along the wall length. The analysis parameters were not finely calibrated to fit the experimental result. The numerical predictions are compared to the test results in terms of lateral load versus drift ratio, as shown in Figure 1. The lateral load capacity and the lateral stiffness of the specimens are captured reasonably well for most of the lateral drift levels. The degradation in lateral stiffness of both specimens at the negative drift of 2%, is due to buckling of the longitudinal reinforcement. The numerical response does not reflect this behavior because the model does not consider reinforcement bar buckling.



(a) NSW4 (b) NSW5

Figure 1. Experimental and Analytical Lateral load vs. Drift ratio

3. Damage levels and repair methods

An essential element of the PBEE methodology is establishing the damage states and their associated repair method (RM). The damage states set limits for the extent of damage that a building component can endure under lateral loads. The 2004 AIJ Draft Guidelines (AIJ, 2004) defined four member damage levels and linked them to four limit states based on steel bars and concrete conditions. However, these definitions are specific to structural members, and the associated RM remains ambiguous. Similarly, the Damage Evaluation Guideline (JBDPA, 2014) categorized structural member damage into five classes based on damage descriptions, which differ from the AIJ limit states. The practitioners, code committees, and researchers lack consensus regarding the definition of damage state, their quantitative assessment parameters, and the associated RM. Therefore, four damage levels are characterized using visual indicators of damage, such as crack width, concrete spalling, crushing, and reinforcement buckling, based on the progression of damage reported in Yuniarsyah et al. (2018). The damage levels were then linked to the RM needed to reinstate the component. Information on each repair method is discussed in the following bullet points.

- RM-1: RM-1 is associated with cosmetic repair. It is characterized by the appearance of hairline cracks with small crack widths. Repair of surface finishes may be required to restore the aesthetic appearance of the NS wall.
- RM-2: It involves the injection of epoxy resin into cracks. The residual crack width (W'_{cr}) is less than 0.4mm. If these cracks are left untreated beyond this damage state, there is a risk of corrosion (Schlicke et al., 2021).
- RM-3: RM-3 is associated with minor concrete patching and epoxy injection of the cracks. The spalling of the cover concrete becomes noticeable, and the compression cracks also start to appear.
- RM-4: This RM corresponds to the buckling of reinforcement and crushing of concrete. It involves replacing the buckled reinforcement and damaged concrete in the crushed zone.

Experiment results from the five cyclic tests (Lu et al., 2017; Yuniarsyah, 2018) on the wall specimens were used to quantify and validate the definitions of RMs. For the quantification of RMs, the material strain values were obtained from each wall analysis at the drift levels that corresponded to the damage level and necessary RM observed in the experiments. Two material strain values, one representing concrete compressive strain, and the other representing longitudinal reinforcement tensile strain was assigned for each RM. It was assumed that the minimum value of the drift ratio, corresponding to the two strain levels, triggers that RM. Examples of the four damage levels are shown in Figure 2.

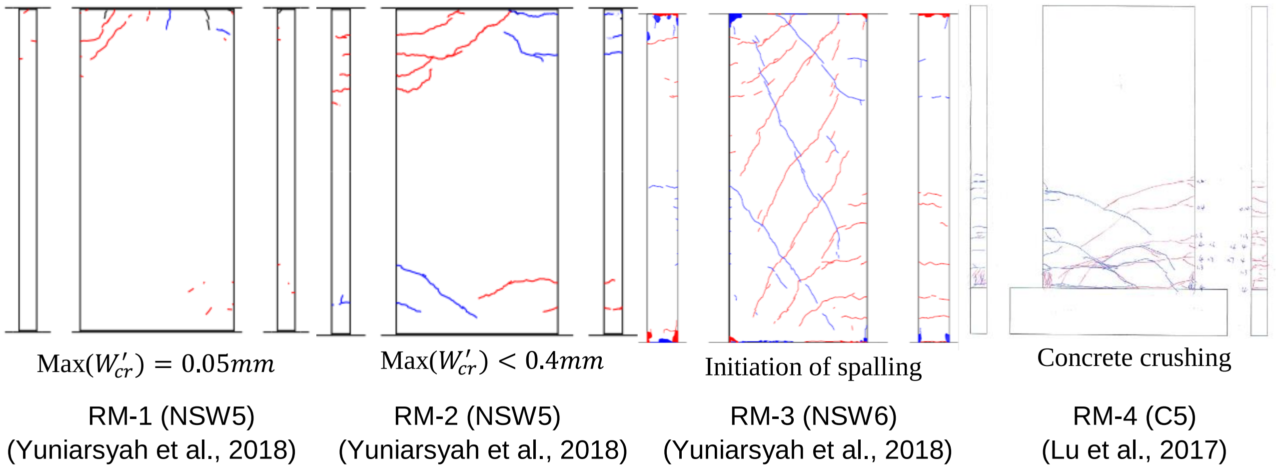


Figure 2. Damage condition corresponding to different RM

4. Summary of data used

The development of fragility curves needs a large number of damage data for the representative NS walls. Fictitious wall models were created and simulated under the quasi-static cyclic loading protocol. The design practice of NS walls in Japan does not typically require seismic analysis, and the infills are chosen without thorough consideration. The construction recommendation guideline for non-structural elements (AIJ, 2003) proposes using 120mm thick walls with a single curtain of D10 reinforcement at 300mm intervals for internal partitions and 150mm thick walls with double curtain reinforcement at 450mm intervals for the exterior partitions. However, the practitioners often deviate from these recommendations in practice, opting for thicker walls up to 180mm for the exterior partitions and reducing the spacing of reinforcements. As for the connection of NS walls with the floors at the top and bottom, two approaches are commonly used. The first involves a rigid connection with beams at both ends of the wall, while the second utilizes a rigid connection at one end with a seismic slit at the other. In the latter scenario, standard practice includes the use of tie-bars to prevent out-of-plane deformation, along with filler material for insulation.

For the above reasons, the wall samples were created by considering reinforcement ratio and wall thickness as design variables. The reinforcement ratio varied from 0.2% to 0.45%, and the wall thickness ranged from 120 mm to 180 mm. Given the typical absence of confinement in the end zones of NS walls in practical applications, it is important to note that the reinforcement ratio referred to in this context is the distributed reinforcement in the wall panel. Additionally, two more parameters were considered: the shear span to wall length ratio and the axial load ratio. The shear span is determined by the ratio of base moment to base shear and is influenced by the boundary conditions of the wall. For example, in the case of the NS wall with an adequate seismic slit, allowing one end of the wall to sway freely, the shear span to wall length ratio is equal to the aspect ratio. However, for NS walls that are rigidly tied to beams at both the top and bottom, the shear span to wall length ratio is less than its aspect ratio. In cases where the seismic slit is not adequate, such that the NS wall will interact with the surrounding beam under significant lateral load, there will be an increase in axial load and a concurrent decrease in shear span to wall length ratio (Yuniarsyah, 2018). The aspect ratio for all wall samples was set at two and the shear span to wall length ratio was adjusted within a range of 1 to 2, and the axial load ratio was raised to a maximum of 15%. By generating the wall models with different combinations of these four variables at different intervals, a total of 678 wall simulations were carried out. The analysis results were processed to obtain the drift ratios at each RMs.

5. Probability distributions and fragility functions

Three common probability distributions that necessitate a positive value of the random variable (drift ratio in this case) were used to fit the damage data. These distributions include the Lognormal, Gamma, and Weibull distributions. The distribution parameters were computed using the method of maximum likelihood, which seeks to optimize the likelihood functions (Benjamin & Cornell, 2014). The parameters or the fragility functions derived from each of the three distributions are presented in Table 1. To assess the goodness of fit, a two-sample Kolmogorov-Smirnov test (K-S test) (Massey, 1951) was conducted for all distributions, and the results are also included in Table 1. The K-S test statistic (D) represents the maximum absolute difference among n (sample size) discrepancies between the empirical cumulative distribution function (CDF) and the CDF associated with the hypothesized distribution. Parameter D serves as the criterion for accepting or rejecting the null hypothesis (H_0). The null hypothesis (H_0) for the K-S test is that there is no significant difference between the empirical distribution of the sample and the hypothesized probability distribution. At a specified significance level (α), H_0 is rejected if $D > D_{critical}$, where $D_{critical}$ is the critical value dependent on the sample size. For the considered sample size and a 5% significance level, the calculated $D_{critical}$ value is 0.074. The "H_0" column in the table indicates the decision regarding the null hypothesis. "A" stands for 'H_0 Accepted', signifying that the sample is consistent with the hypothesized distribution. Conversely, "R" represents 'H_0 Rejected', indicating that the sample does not conform to the hypothesized distribution.

Table 1. Distribution parameters and K-S test statistics

Repair Method	Lognormal distribution				Gamma distribution				Weibull distribution			
	Distribution parameters		K-S Test results		Distribution parameters		K-S Test results		Distribution parameters		K-S Test results	
	θ	β	D	H_o	k	λ	D	H_o	k	λ	D	H_o
RM-1	0.131	0.123	0.032	A	64.960	492.486	0.022	A	7.321	0.140	0.015	A
RM-2	0.373	0.141	0.140	R	54.198	143.835	0.094	R	9.337	0.397	0.022	A
RM-3	0.631	0.118	0.174	R	77.067	121.315	0.125	R	12.382	0.663	0.071	A
RM-4	0.974	0.275	0.600	R	13.128	12.966	0.600	R	3.671	1.122	0.202	R

θ, β: median and dispersion of lognormal distribution

k, λ: shape and scale of Gamma and Weibull distribution

In Table 1, the cells shaded in blue represent the smallest D for the K-S test conducted on the respective RMs. For all the four RMs, Weibull distribution yields the smallest values of D, indicating a better fit compared to other distributions considered in this analysis. All the distributions fail the K-S goodness-of-fit test for RM-4. The probability density function (PDF) for the Weibull distribution is shown in Equation (1), and Equation (2) gives the mean (μ) and the standard deviation (σ) for the Weibull distributed variable.

$$f(x;\lambda,k) = \frac{k}{\lambda}\left(\frac{x}{\lambda}\right)^{k-1} \exp\left[-\left(\frac{x}{\lambda}\right)^{k}\right] \quad x \geq 0; k,\lambda > 0 \tag{1}$$

$$\mu = \lambda\Gamma\left(1+\frac{1}{k}\right);\ \sigma = \lambda\sqrt{\Gamma\left(1+\frac{2}{k}\right) - \Gamma^2\left(1+\frac{1}{k}\right)} \tag{2}$$

where, k defines the shape of the distribution, λ is a scale parameter, and Γ is the gamma function. The values of the distribution parameters are given in Table 1 and the mean (μ) and standard deviation (σ) of the drift ratios (%) at each RMs are presented in Table 2.

Table 2. Mean (μ) and standard deviation (σ) of the distribution

Repair Method	μ (%)	σ (%)

RM-1	0.131	0.021
RM-2	0.376	0.048
RM-3	0.636	0.062
RM-4	1.003	0.304

It should be noted that the dispersion of the CDF, as defined by the distribution parameters, does not account for uncertainties beyond those arising from the variability in the data itself. These uncertainties are attributed to the simulation of wall models with varying design parameters (reinforcement ratio, wall thickness, axial load ratio, and shear span to wall length ratio) and do not encompass additional sources of uncertainty. Adjustments should be made to account for other uncertainties. Figure 2 presents the empirical and theoretical fragility functions for the NS walls considered in the analysis.

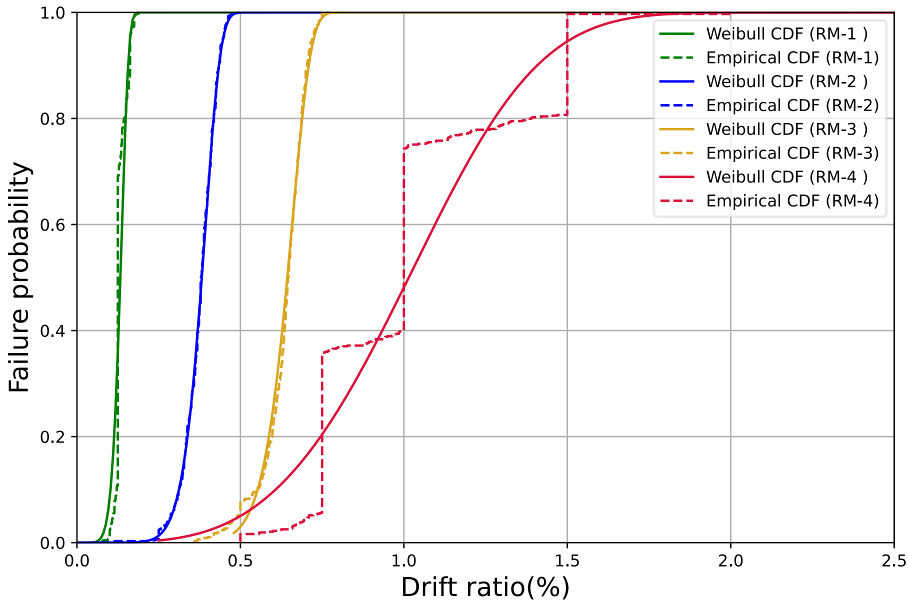


Figure 3. Fragility functions for NS walls

6. Conclusions

This study developed a series of fragility functions for non-structural reinforced concrete walls commonly used as infills in Japanese construction by using numerical analysis to generate the damage data. The fragility functions quantify the likelihood of exceeding one or more damage levels at a given demand parameter. The damage thresholds in this paper are linked with the repair method needed to reinstate the component. Fragility functions are presented for each repair method using Weibull distribution and drift ratio as demand parameters. Since the fragility functions are given for repair method, it can be used for potential loss estimation of the buildings, and also in the performance assessment of the buildings incorporating these walls.

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