SEISMIC FRAGILITY ANALYSIS OF BUILDINGS EQUIPPED WITH PROPPED ROCKING WALL SYSTEMS

Afsoon NICKNAM\textsuperscript{1} and André FILIATRAULT\textsuperscript{2}

Abstract: This paper presents a seismic vulnerability assessment study on a newly proposed seismic force resisting system named Propped Rocking Walls (PRW) based on the Methodology presented in FEMA P695. Propped rocking wall is a form of seismic-force resisting system that combines unbonded post-tensioned (PT) concrete walls with passive supplemental damping devices. This study is aimed at verifying the adequacy of a proposed direct displacement-based design procedure for propped rocking walls through evaluating global/local failure capacity of the proposed system. To this end, seismic fragility curves for predefined performance limit states were developed for a PRW prototype system through cumulative distribution functions under the FEMA P695 Far-Field ground motions set. The seismic response of the PRW numerical model used for this evaluation was validated with shake table test results conducted previous to this study. Results from this evaluation confirmed that the specific PRW prototype system considered in this study meets the FEMA P695 criteria regarding the margin of safety against collapse probabilities under MCE ground motions.

Introduction
As illustrated in Figure 1, the PRW system consists of a slender concrete wall post-tensioned using unbonded steel bars and “propped” near its top with multi-story diagonal steel braces equipped with hysteretic dampers. During a major or moderate earthquake, the hysteretic energy dissipators of the steel props provide a significant source of seismic energy dissipation, while the rocking triggered at the wall base/foundation interface avoids the formation of a plastic hinge in the wall panel. Furthermore, residual deformations are minimized due to the self-centering capacity of the rocking wall. As a result, the system can be designed to sustain minimal structural damage and negligible post-earthquake residual drifts.

Seismic Design
As one of the first stages of this research, Nicknam and Filiatrault (2012) developed a seismic Direct Displacement-Based Design (DDBD) procedure (Priestley \textit{et al.} 2007) for buildings equipped with PRW systems. This design procedure was based on a closed-form solution derived for the base shear-roof displacement relationship of the PRW system at its

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{PRW_system.png}
\caption{General layout of a PRW system}
\end{figure}

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maximum response (Nicknam and Filiatrault 2012). Validation studies were then performed based upon nonlinear response history analyses on designed PRW archetype models using the proposed design procedure. The iterative numerical design procedure for a given PRW uses several design criteria to ensure that the desired performance objectives are achieved (Nicknam and Filiatrault 2012). These design criteria are summarized as follows: 1) Full re-centering condition to ensure negligible permanent deformations; 2) Elastic response of the pre-stressed reinforcement under the Maximum Considered Earthquake (MCE) to ensure significant safeguard against collapse and to control residual displacements; 3) Elastic response of the concrete wall under the Design Earthquake (DE); 4) Damage control in hysteretic dampers under the MCE level; and 5) Prevention of sliding of the wall base through adequate design and/or detailing.

**Experimental Study**

Figure 2(a) illustrates the plan view of the prototype building selected for the shake table testing program. As shown, the building is symmetric and assumed to be meeting the requirements of a Seismic Design Category (SDC) D according to ASCE/SEI 7-10 (ASCE 2010). To provide lateral resistance in the north-south direction of the structure, two PRW units are introduced and located symmetrically on the east and west perimeters of the building. Each wall extended the full height of the three-story building. The height of each floor of the prototype building is 152.25 in. The hysteretic energy dissipating devices was selected to be in the form of Buckling-Restrained Braces (BRB), designed and manufactured by StarSeismic, LLC. Figure 2(b) illustrates the three-dimensional configuration of the 1:3 scaled shake table testing model.

![Diagram of experimental setup](image)

A subset of 10 of the 44 historical motions of the FEMA P695 far field Ground Motion (GM) set (FEMA 2009) was selected for this experimental investigation (Nicknam 2014). For the seismic tests, five incremental shaking intensities were defined including DE, where the
A subset of GMs were scaled to the Design Earthquake (10% probability of exceedance in 50 years), and MCE, where the subset of GMs are scaled to the Maximum Considered Earthquake (2% probability of exceedance in 50 years). In general, the seismic test results confirmed the intended behavior and reliability of the proposed controlled rocking wall system. In particular, the tests demonstrated the self-centering and damage control capabilities of the PRW system as well as the reliability of the PT threadbars for post-tensioning even up to relatively high levels. No failure in the buckling-restrained braces was reported due to sufficient axial strain capacity provided for these components. Both braces exhibited stable hysteretic behavior over the entire tests for all displacement amplitudes and achieved a cumulative plastic axial ductility capacity significantly higher than what is required per AISC 341-10 (AISC 2010). The capacity design procedure applied to the concrete wall was adequate to provide sufficient margin against concrete shear failure due to the rocking elastic response of the propped rocking wall even at the MCE intensity level. The initial fundamental period of the PRW test specimen, however, was measured at approximately 0.5 sec which was significantly longer than the computed fundamental period from the pre-test numerical analysis (0.18 sec). This difference was influenced by the foundation compliance at the wall/base plate interface and by the rotational compliance of the shake table. Despite of this increased flexibility at the base of the PRW test specimen which caused reduction of seismic forces, sufficient inelastic response of hysteretic dampers and rocking of the wall were still triggered during the seismic tests and made it possible to evaluate the performance of propped rocking walls as a seismic force-resisting system.

**Numerical Study**

An object-oriented framework, Open Systems for Earthquake Engineering Simulation (OpenSees), was used to create the numerical model and perform nonlinear dynamic analyses described in this section. The concrete wall was modeled using a Force-Based Beam-Column Element with a fiber section. In order to model the gap opening at wall base/foundation interface, a series of 11 axial springs was inserted at the base of the wall, using zero-length elements with Elastic-Perfectly Plastic Gap Material. To model unbonded PT reinforcement, Corotational Truss Elements were considered and post-tensioning forces were introduced to these elements using the initial strain condition. The upper end of the PT elements was kinematically (lateral, vertical, and rotational degrees-of-freedom) constrained to the top of the wall to model the anchorage between the post-tensioning reinforcement and the concrete wall. The foundation compliance, reported during the seismic tests, was taken into account by introducing supplemental flexible springs at locations where gap elements as well as PT bars were attached to the base (using zero-length linear springs). Finally, the buckling-restrained braces were modeled using Corotational Truss Elements. Inherent damping properties of the analytical model was idealized using Rayleigh damping with damping ratios of 1% and 5% of critical in the fundamental and second mode of the structure, respectively. Figure 3 illustrates the comparison between the experimental and numerical results of the test model in terms of the hysteretic response of the BRBs under the Landers 1992 earthquake ground motion at the MCE level of intensity. As shown, there is a good agreement between the simulated and measured responses.
Figure 3. Comparison of numerical and experimental results for hysteretic response of (a) west and (b) east BRBs under Landers 1992 earthquake at MCE level of intensity.

Figure 4 reveals the good agreement of the test data and numerical simulation in terms of floor relative displacement response-histories under the same seismic excitation. Similar patterns could be observed for the floor absolute acceleration response-histories (see Figure 5). The good agreement between numerical and experimental results further confirmed the validation of the developed numerical model (Nicknam 2014).

Figure 4. Comparison of numerical and experimental results for floor relative displacement response under Landers 1992 earthquake at MCE level of intensity; (a) Third, (b) Second and (c) First Floor.

Figure 5. Comparison of numerical and experimental results for floor absolute acceleration response under Landers 1992 earthquake at MCE level of intensity; (a) Third, (b) Second and (c) First Floor.

Seismic Collapse Fragility Analysis of Prototype Propped Rocking Wall System

In this section, the seismic performance of the PRW prototype system, supported on an ideal rigid foundation, is evaluated beyond Maximum Considered Earthquake (MCE) level of intensity using a seismic collapse fragility framework. This study is based on the
Methodology presented in FEMA P695 (FEMA 2009) and is aimed at verifying the adequacy of the proposed DDBD procedure for propped rocking walls through evaluating global/local failure (defined later in the section) capacity of the proposed system. For this purpose, nonlinear dynamic analyses were conducted using the FEMA P695 Far-Field Ground Motions (GM) set to calculate the median value of the collapse ground motion intensity ($\bar{S}_{CT}$) and the Collapse Margin Ratio (CMR) for the prototype building. The FEMA P695 Far-Field record set includes 22 historical records (44 individual components) selected worldwide using the criteria described in Section A.7 of the FEMA P695 document. In accordance with the Methodology, the median collapse capacity is defined herein as the ground motion intensity in which half of the records in the set cause a global or local collapse in the proposed seismic force-resisting system. Although not required, a full incremental dynamic analysis was carried out. It should be noted that the term ‘system’ used herein is associated with the single PRW (archetype) building considered in the study. Therefore, the collapse capacity evaluation conducted in this section is for this single archetype and cannot be considered as an evaluation of the PRW system as a seismic force resisting-system, for which a suite of archetype systems spanning the design space would need to be designed and evaluated. This evaluation defines the seismic intensity measure as the median spectral acceleration of the 44 ground motions at the fundamental period of the PRW system under consideration. Per FEMA P695, this fundamental period is defined by:

$$T = C_u T_a = C_u C_h h_n \geq 0.25 \text{ sec}$$

where $h_n$ is the building height in ft, the values of the coefficient $C_u$ are given in Table 12.8-1 of ASCE/SEI 7-10, and values of period parameters $C_h$ and $x$ are given in Table 12.8-2 of ASCE/SEI 7-10. The fundamental period of the prototype PRW system under study was calculated as follows:

$$T = C_u C_h h_n = 1.4 \times 0.02 \times (12.7 \times 3)^{0.75} = 0.43 \text{ sec}$$

Collapse Criteria

With reference to FEMA P695 Methodology, structural collapse under each ground motion can be assessed indirectly through non-simulated component collapse limit state criteria. Non-simulated limit state checks are similar to the assessment approach of ASCE/SEI 41-06 (ASCE 2006), in which component acceptance criteria are used to evaluate specific performance targets based on demand quantities extracted from the analyses. Even though this approach is more of an approximation to the actual behavior of the system, which increases the uncertainty in analytical results and tends to provide conservative estimates of collapse limit states, it is a practical approach that provides a consistent method for evaluating the effects of deterioration and collapse mechanisms that are otherwise difficult (or impossible) to incorporate directly in the analytical model. Table 1 summarizes the non-simulated collapse criteria considered in this study. As can be seen from this table, two types of collapse are defined for the PRW system: 1) Collapse at the component level (i.e. Local) and 2) Collapse at the system level (i.e. Global).

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT</td>
<td>Local</td>
<td>Complete loss of post-tensioning force in PT bars.</td>
</tr>
<tr>
<td>BRB</td>
<td>Local</td>
<td>Excessive axial strain in BRBs steel core (≥ 4%) which is assumed to result in fracture of element and therefore loss of seismic energy dissipating devices and further instability of the system.</td>
</tr>
<tr>
<td>S</td>
<td>Local</td>
<td>Shear failure in concrete wall.</td>
</tr>
<tr>
<td>D</td>
<td>Global</td>
<td>Large peak inter-story drift ratios (≥ 4%) which is typically assumed to be associated with most likely unrepairable damage to other parts of the building than the seismic force-resisting system itself.</td>
</tr>
<tr>
<td>RD</td>
<td>Global</td>
<td>Large residual inter-story drift ratios (≥ 0.02) which is assumed to correspond to</td>
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</table>
significant damage in the system.

Analyses Results
Figure 6 presents results of the incremental dynamic analyses conducted on the PRW prototype system. In this figure, the vertical axis represents the ratio between median values of spectral acceleration of all ground motion records ($S_T$) and the maximum considered earthquake intensity ($S_{MT}$) both at the fundamental period of the system, and the horizontal axis represents values of peak inter-story drift ratio recorded in each analysis. As a result, a ratio of unity on the vertical axis of this figure corresponds to a collapse probability of 50%. Each point in these figures corresponds to the results of one nonlinear dynamic analysis of the structural model subjected to one ground motion record that is scaled to one intensity level. Figure 6 connects results for a given ground motion scaled to increasing spectral intensities. Differences between the IDA curves reflect differences in the response of the same system when subjected to different ground motions with different characteristics. Collapse modes under each ground motion are further tabulated in Figure 7. The dominant collapse mode across the entire ground motion ensemble (i.e. the point at which the analysis under each of corresponding ground motions was stopped due to the occurred collapse in the system) is the complete loss of post-tensioning force in the PT bars (PT). Only four of the ground motions (Motions 19, 27, 30 and 42) caused shear failure in the concrete wall prior to the complete loss of post-tensioning force in the PT bars. As can be seen from Figure 7, in most cases, the PT collapse mode is associated with excessive axial strain in the BRBs steel core. This could further indicate that the displacement profile of the PRW system was predominantly in accordance with its first mode of vibration around the peak response under those seismic excitations.

Evaluation of Collapse Margin Ratio
The collapse margin ratio (CMR) is defined as the ratio of $\hat{S}_{CT}$ to the maximum considered earthquake intensity ($S_{MT}$), which is obtained from the response spectrum of MCE ground motions at the fundamental period of system under consideration, as shown by Equation (2).

$$CMR = \frac{\hat{S}_{CT}}{S_{MT}}$$  \hspace{1cm} (2)

The MCE intensity for the PRW prototype system lies within the constant acceleration plateau of the ASCE/SEI 7-10 response spectrum for Seismic Design Category (SDC) D_max, and therefore $S_{MT} = 1.5$ g. Using collapse data from IDA results, a collapse fragility curve can be defined through a cumulative distribution function (CDF) which indicates the probability that a component or the system, as a whole, will exceed a certain performance limit state, given a specific level of demand. Figure 8(a) shows the cumulative distribution plot obtained by least squares fitting a lognormal probability distribution to the collapse data points of Figure 6. The lognormal distribution is defined by two parameters: the median and standard deviation in the log plane ($\beta$). The horizontal axis in Figure 8(a) represents the ratio between median values of spectral acceleration of all ground motion records ($S_T$) and the maximum considered earthquake intensity ($S_{MT}$) both at the fundamental period of the system. As a result, the median of the fitted lognormal distribution represents the collapse margin ratio and is computed as $CMR=2.11$. The dispersion in results is due only to the record-to-record (RTR) variability (uncertainty) and is calculated as $\beta_{RTR} = 0.31$. Figure 8(b) shows the fragility curve corresponding to the 1% peak inter-story drift ratio limit state. Accordingly, the median of the fitted lognormal distribution (indicated in the figure as $CMR_{\delta=1%}$) represents the seismic intensity, as a ratio of the maximum considered earthquake intensity, at which half of ground motions cause the PRW system to develop peak inter-story drift ratios of greater than or equal to 1%. The median and lognormal standard deviation of the collapse fragility curve of Figure 8(b) are obtained as 0.76 and 0.41, respectively. Given that the PRW prototype system was designed for 1% peak inter-story drift ratio under the design earthquake (defined
as 2/3 of the maximum considered earthquake), a median spectral acceleration of 0.76\(S_{MT}\) at which half of the ground motions cause minimum 1% peak inter-story drift is found reasonable.

Figure 6. Results of incremental dynamic analyses on the PRW prototype system under the FEMA P695 Far-Field record set

<table>
<thead>
<tr>
<th>GM</th>
<th>Collapse intensity*</th>
<th>PT</th>
<th>BRR</th>
<th>N</th>
<th>D</th>
<th>ED</th>
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<td>x</td>
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</table>

* In terms of \(S/S_{MT}\)

Figure 7. Non-simulated collapse modes under each ground motion used in incremental dynamic analyses of the PRW prototype system
Figure 8. (a) Collapse fragility curve for the PRW prototype system, and (b) One percent inter-story drift ($\delta=1\%$) fragility curve of the PRW prototype system using FEMA P695 Far-Field record set

**Evaluation of Adjusted Collapse Margin Ratio**

According to FEMA P695, the collapse capacity, and the calculation of the collapse margin ratio, can be significantly influenced by the frequency content (spectral shape) of the ground motion. As a consequence, to account for the effects of spectral shape, the collapse margin ratio, $CMR$, is modified to obtain an adjusted collapse margin ratio, $ACMR$, as:

$$ACMR = SSF \times CMR$$

where $SSF$ represents the spectral shape factor which is a function of the fundamental period, $T$, the period-based ductility, $\mu_T$, and the applicable seismic design category. Values of $SSF$ are provided in Table 7-1b of FEMA P695 for SDC $D_{max}$. The period-based ductility for a given archetype system, $\mu_T$, is defined as the ratio of the ultimate roof drift displacement to the effective yield roof drift displacement (Equation (4)).

$$\mu_T = \frac{\delta_u}{\delta_{y\text{eff}}}$$

In accordance with the Methodology, results from nonlinear static analysis are used to calculate the period-based ductility ($\mu_T$) defined by Equation (4). The vertical distribution of the lateral force used in the nonlinear static analysis was proportional to the fundamental mode shape of the numerical models. Figure 9 presents the base shear versus the roof drift ratio (pushover curve) for the PRW prototype system under study. It should be noted that since pushover analyses are intended to verify the models and provide a conservative bound on the system over-strength factor, checks for non-simulated collapse modes are not incorporated directly. Consequently, non-simulated collapse modes are considered when evaluating ultimate roof drift displacement, $\delta_u$, as described below. In this case, the ultimate roof displacement ($\delta_u$) is determined as the displacement associated with the onset of complete loss of post-tensioning force in PT bars. In other words, $\delta_u$ is the minimum lateral displacement experienced by the PRW system for which the PT bars will lose all the post-tensioning force once the system returns to its initial position. Values of maximum base shear, $V_{\text{max}}$, and ultimate roof displacement, $\delta_u$, are indicated in Figure 9. The effective yield roof drift displacement is given by Equation (5):

$$\delta_{y\text{eff}} = C_o \frac{V_{\text{max}}}{W} \left[ \frac{g}{4\pi^2} \right] \left( \max\{T,T_1\} \right)^2$$

where $C_o$ relates fundamental-mode (SDOF) displacement to roof displacement, $V_{\text{max}}/W$ is the maximum base shear normalized by building weight, $g$ is the acceleration of constant, $T$ is the fundamental period defined by Equation (1) and $T_1$ is the fundamental period of the archetype model computed using eigenvalue analysis.

The coefficient $C_o$ is based on Equation C3-4 of ASCE/SEI 41-06, as follows:
where \( m_x \) is the mass at level \( x \); and \( \phi_{1,x} \) (\( \phi_{1,r} \)) is the ordinate of the fundamental mode at level \( x \) (roof), and \( N \) is the number of levels. In accordance with the PRW system fundamental mode ordinates, the coefficient \( C_o \) is calculated as follows:

\[
C_o = \frac{\sum_{x=1}^{N} m_x \phi_{1,x}}{\sum_{x=1}^{N} m_x \phi_{1,r}^2} \tag{6}
\]

Given \( V_{\text{max}} = 370 \text{ kips} \) (see Figure 9) and \( W = 3 \times 201 = 603 \text{ kips} \), the effective yield roof drift displacement is obtained as follows:

\[
\delta_{\text{y,eff}} = 1.29 \times \left( \frac{0.16 + 0.53 + 1.00}{0.16^2 + 0.53^2 + 1.00^2} \right) \times \frac{370}{603} \left[ \frac{g}{4\pi^2} \right] \left( \max \left( 0.43, 0.39 \right) \right)^2 = 1.43 \text{ in}
\]

Therefore, using Equation (4), the period-based ductility, \( \mu_T \), is calculated as:

\[
\mu_T = \frac{15.5}{1.43} = 10.8
\]

With reference to Table 7-1b of FEMA P695, for values of \( \mu_T \geq 8 \) and \( T \leq 0.5 \text{ sec} \), the spectral shape factor is determined as \( \text{SSF}=1.33 \).

**Total System Collapse Uncertainty**

Acceptable values of adjusted collapse margin ratio for 10% and 20% probability of collapse for MCE ground motions (ACMR\(_{10\%}\) and ACMR\(_{20\%}\), respectively) are based on the total system collapse uncertainty (\( \beta_{\text{TOT}} \)) and established values of acceptable probabilities of collapse. The total system collapse uncertainty is calculated combining four sources of uncertainties (\( \beta_{\text{RTR}}, \beta_{\text{MDL}}, \beta_{\text{DR}} \) and \( \beta_{\text{TD}} \)). The quality of the structural system design requirements is rated (B) Good (i.e. \( \beta_{\text{DR}} = 0.2 \)) based on following rationales: 1) The prototype PRW system was designed according to the ACI 318-11 (ACI 2011) and AISC 341-10 (AISC 2010) design provisions. Such design provisions represent many years of development and include lessons learned from a number of major earthquakes (FEMA 2009); 2) The DDBD requirements were reasonably extensive and provided reasonable safeguards against unanticipated collapse modes; 3) Design requirements establish a suggested hierarchy of component yielding and collapse; 4) There is substantiating evidence
(experimental data and numerical verifications) with a high level of confidence that the properties, criteria, and equations provided in the design requirements will result in component designs that perform as intended. The quality of test data is rated (C) Fair (i.e. $\beta_{TD} = 0.35$) based on following rationales: 1) Experimental evidence is sufficient so that nearly all important behavior aspects from material level to system level are well understood and results can be used to quantify important parameters that affect design requirements and analytical modeling. Nevertheless, only a single configuration of the PRW system was tested; 2) Moderately reliable experimental information is produced on important parameters that affect design requirements and analytical modeling; 3) Comparable tests from other testing programs on unbonded rocking concrete walls with passive supplemental damping devices do not contradict results from the system-specific testing program; 4) Test results are supported by basic principles of mechanics. The nonlinear structural modeling quality is rated (B) Good (i.e. $\beta_{MDL} = 0.2$) based on following rationales: 1) Nonlinear models capture some of nonlinear deterioration and response mechanisms leading to system/component collapse and therefore, component-based limit state checks (i.e. non-simulated collapse modes) are necessary to assess collapse; 2) There is reasonable confidence that the response captured by the numerical model is indicative of the primary structural behavior characteristics that affect system/component collapse. Based on these individual values of uncertainty and given $\beta_{RTR} = 0.31$, the total system uncertainty is determined as the Square Root of the Sum of the Squares (SRSS) of individual values of uncertainty to be $\beta_{TOT} = 0.55$. Table 7-3 of FEMA P695 provides acceptable values of the adjusted collapse margin ratio based on the total system collapse uncertainty and values of acceptable collapse probability. For this study, the acceptable values of adjusted collapse margin ratio for a system uncertainty $\beta_{TOT} = 0.55$ are $ACMR_{10\%} = 2.02$ and $ACMR_{20\%} = 1.59$. Table 2 summarizes the collapse results for the structural system under study along with other quantitative values needed for the collapse assessment discussed in the following section. As can be observed from the acceptable values of adjusted collapse margin ratio in this table, the PRW system archetype considered in this study exhibit enough safeguards against the predefined modes of collapse and therefore pass the acceptance criteria.

<table>
<thead>
<tr>
<th>IDA Results</th>
<th>Collapse Uncertainty</th>
</tr>
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<tbody>
<tr>
<td>$S_{MT}$ (g)</td>
<td>$S_{CT}$ (g)</td>
</tr>
<tr>
<td>1.5</td>
<td>3.15</td>
</tr>
</tbody>
</table>

Conclusions
In this paper, the seismic performance of a propped rocking wall prototype system under consideration is evaluated beyond Maximum Considered Earthquake (MCE) level of intensity using the Methodology presented in FEMA P695. The objective of this study was to assess the capacity of the proposed archetype system for a set of predefined collapse modes, using a numerical model which was validated with the shake table experimental results. Results from this study confirm that the proposed PRW system reveals enough margin of safety against collapse probabilities of 10% and 20% under MCE ground motions. It should be noted that the collapse capacity evaluation presented in this paper was conducted for a single PRW archetype and is not considered to be an evaluation of the full PRW system, which would require the evaluation of an ensemble of archetypes spanning the design space.

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