Seismic design of precast concrete rocking wall systems with varying hysteretic damping

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- Current seismic design practice for precast concrete rocking walls limits the design efficiency and applications of the wall systems.
- This paper uses shake-table test results from previous research, current design guidelines, and analytical models to develop and validate an improved method for designing precast concrete rocking wall systems that account for their unique energy-dissipation mechanisms.
- The proposed design method is presented along with the results of a parametric study of building designs using the proposed method for different ground motion types and intensities.
- The results indicate that an improved design procedure would lead to more cost-efficient designs for precast concrete rocking wall systems with seismic performance within the allowable limits.

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Precast concrete rocking walls designed with unbonded post-tensioning can provide a reliable, self-centering earthquake-load-resisting system. Single rocking walls (SRWs) are the simplest form of such systems that are connected to the foundation using only unbonded post-tensioning tendons,[1] which remain elastic during design-level earthquakes. Because SRWs have a low energy-dissipation capacity, alternative systems can be designed with supplemental hysteretic energy-dissipating elements.[2–4]

Sritharan et al. developed the precast concrete wall with end columns (PreWEC) system, which is an efficient alternative to cast-in-place concrete (CIP) walls and incorporates easily replaceable steel O connectors to provide supplemental damping. The O connectors are positioned between the precast concrete wall panel and the columns near the end of the wall. **Figure 1** compares details of an SRW with a typical PreWEC system. By increasing the number of O connectors, the amount of hysteretic energy dissipation can be easily altered. **Figure 2** presents the measured force-deformation response of a typical O connector with the specified dimensions shown in the figure. The connector was subjected to asymmetric displacements as this was the expected deformation for O connectors in PreWEC.

The design practice detailed in the American Concrete Institute's (ACI's) *Acceptance Criteria for Special Unbonded Post-Tensioned Precast Structural Walls Based on Validation Testing and Commentary*, ITG-5.1[5] does not permit SRWs in high seismic regions due to their low inherent damping,

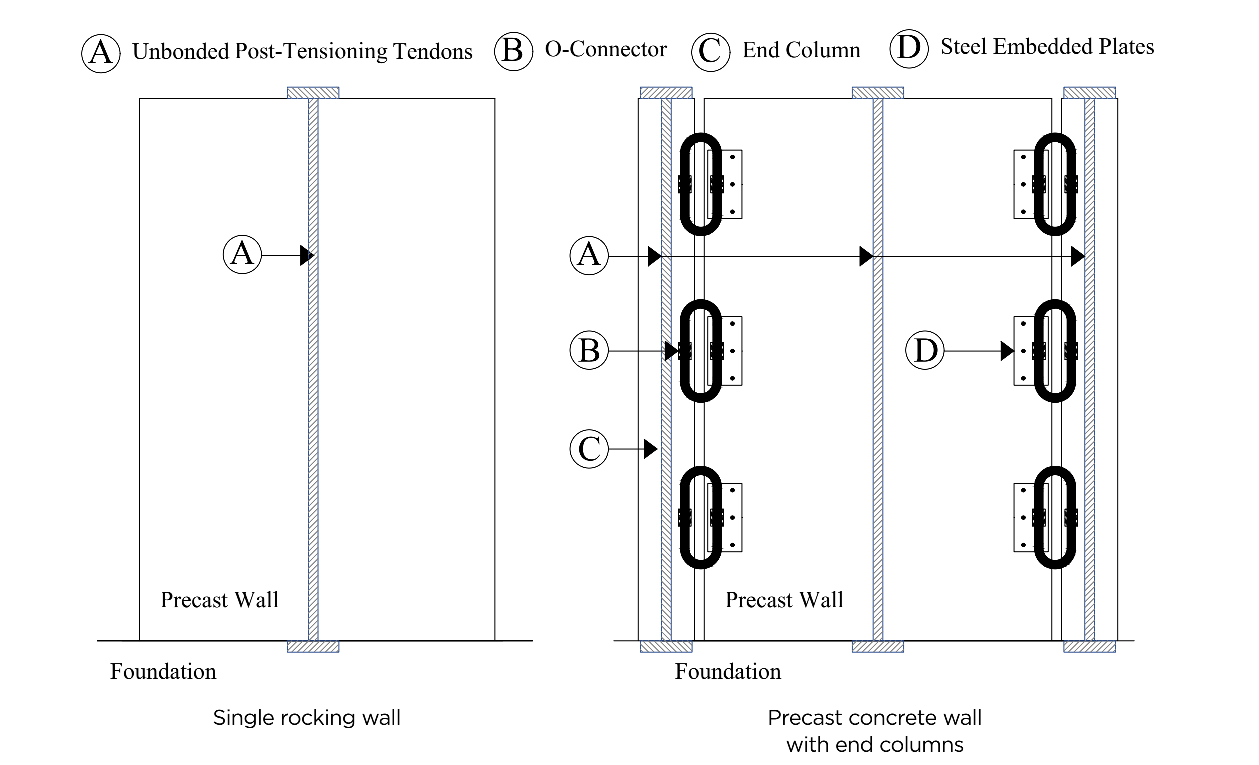


Figure 1. Rocking wall systems.

while the application of PreWECs with a relative hysteretic energy-dissipation ratio exceeding 12.5% (that is, an equivalent viscous damping ratio of about 8%) is permitted. Although the amount of hysteretic damping can be varied in PreWECs, ITG-5.1 does not provide any incentives for designing a system with higher damping ratios. This is because ITG-5.1 follows the force-based design (FBD) approach, which uses a seismic response modification coefficient R as used in ACI's *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11)*[6] and the American Society of Civil Engineers' (ASCE's) *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-10.[7]

In the FBD approach, R is constant for a given structural system; R = 5 is recommended for unbonded post-tensioning walls. In CIP design, the required strength determines

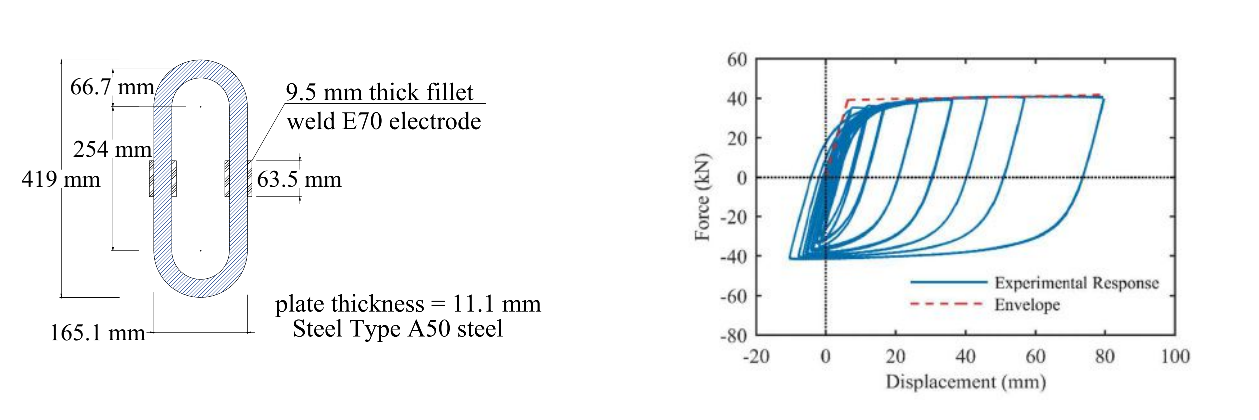
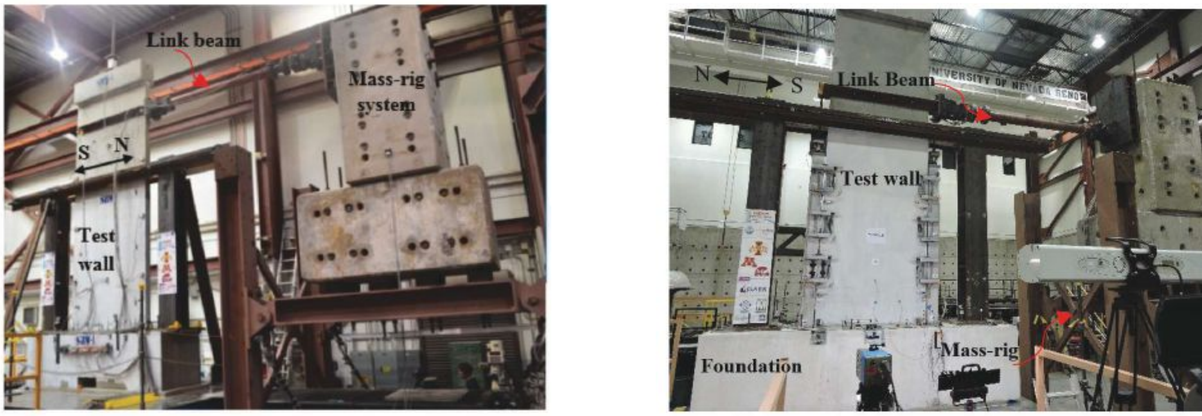


Figure 2. Dimensions and corresponding force-displacement behavior of an O connector used in a precast concrete wall with end columns. Note: 1 mm = 0.0394 in.; 1 kN = 0.225 kip.



Single rocking wall

Precast concrete wall with end columns

Figure 3. Experimental setup used for the shake-table testing of wall systems at the Network for Earthquake Engineering Simulation (NEES) shared facility at the University of Nevada, Reno.

the critical reinforcement quantity, which in turn dictates the amount of damping and justifies the use of a constant R. In precast concrete systems with jointed connections, Nakaki et al.[8] and Kurama et al.[9] found that the damping can be varied by changing the strength of the unbonded post-tensioning and other connecting elements (such as O connectors). Therefore, the use of a constant R factor is inappropriate for designing innovative precast concrete systems such as PreWEC.

This challenge can be overcome using the direct-displacement-based design (DDBD) approach,[10] but most building design specifications continue to use the FBD approach, which prevents the efficient design of precast concrete rocking wall systems. This issue was realized in the third phase of the PRESSS (Precast Seismic Structural Systems) program, in which DDBD was used to design a five-story precast concrete test building. The resulting DDBD force required in the wall direction of the PRESSS building was about 46% of that required based on FBD. The R factor for the FBD approach was 4.5,[11] while the design force required in the wall direction of the PRESSS building following DDBD was equivalent to using an R of about 10. As reported by Priestley et al.,[2] the PRESSS building response in the wall direction was excellent, justifying the use of more than twice the R factor recommended in the design specifications.

To investigate the optimal amount of damping in precast concrete rocking wall systems for seismic design and to address the design challenge associated with using a constant R, a shake-table test investigation was conducted on four SRWs and four PreWEC systems using the Network for Earthquake Engineering Simulation (NEES) shake table facility at the University of Nevada at Reno (UNR), as shown in **Fig. 3**.[12–19] These wall systems were designed as lateral-load-resisting elements of a six-story prototype building at $\frac{5}{18}$ scale with a fundamental period of 0.9 seconds.[20,21] The design base shear, moment-to-shear ratio, and amount of hysteretic damping were varied in the test units. The wall systems were subjected to a series of earthquake excitations with varying intensities, and their performance was assessed in terms of the maximum transient drift, maximum absolute acceleration, and residual drift based on multihazard acceptance criteria. Regardless of the amount of hysteretic damping, all SRWs and PreWECs responded satisfactorily to design-level earthquakes. When subjected to maximum considered earthquakes (MCEs), the systems with low damping produced higher drifts and exceeded the permissible values occasionally.

The tests revealed that the amount of damping in the system had less influence on the wall behavior than commonly believed. Instead, what appears to have controlled the response of the wall systems is their ability to respond in a nonlinear fashion. Lower damping in the system produced more displacement cycles with no significant detrimental effect to the system. This finding is significant because it provides flexible options for the seismic design of precast concrete wall systems and suggests that there is no need to design precast concrete wall systems to produce as much hysteretic damping as is expected for CIP walls, which is estimated to be about 20%.[10] The high equivalent damping in CIP walls is caused by the formation of plastic hinges, which often lead to irreparable damage. By contrast, the precast concrete rocking walls studied herein undergo minimal structural damage, which allows them to be used in the design of seismic resilient buildings.

Using the previous experimental and subsequent analytical work on SRWs and PreWEC systems,[1–4,8–9] this paper proposes damping-dependent R factors to be used with the FBD approach to bring flexibility to the seismic design of precast concrete wall systems. The suggested R factors are then examined using six-, nine-, and twelve-story buildings. First, these buildings are designed with SRWs and PreWECs following the proposed approach. Next, they are subjected to nonlinear dynamic analyses using sets of near-field (NF) and far-field (FF) ground motions, each containing design-basis earthquakes (DBEs) and MCEs. A multiple-level, performance-based seismic evaluation then follows to validate the proposed R factors. Finally, a cost index is developed to select the more economical rocking wall option (that is, SRW or PreWEC) for designing low- to midrise buildings in high seismic regions.

Equivalent damping

SRWs with unbonded post-tensioning possess some inherent viscous damping, impact damping when the wall panel rocks on top of the foundation, and hysteretic damping due to material nonlinearity of concrete at wall toes. Including all of these energy dissipation components, Nazari et al.[20] recommend that SRWs can be assumed to have a total equivalent viscous damping of about 6%. By integrating O connectors and increasing the amount of hysteretic damping, PreWECs, as detailed in Fig. 1, can be designed to have much higher damping than SRWs. Therefore, PreWECs can be designed to easily attain an equivalent viscous damping in the range of 9% to 20%,[21] which enables PreWECs to be used for seismic design according to ITG-5.1.[5]

Current design approach

Precast concrete walls with unbonded post-tensioning can be used in seismic regions if they comply with ITG-5.1,[5] which specifies that all walls with unbonded post-tensioning must include supplementary hysteretic energy-dissipating elements with a capacity to provide a minimum equivalent viscous damping ratio of about 8%. This requirement, as noted, prevents the seismic application of SRWs. Specific design provisions for precast concrete rocking wall systems satisfying ITG-5.1 are presented in ACI's *Requirements for Design of a Special Unbonded Post-tensioned Precast Shear Wall Satisfying ACI ITG-5.1 (ACI ITG-5.2-09) and Commentary*.[22] Accordingly, the following steps can be used to design PreWEC systems:

- Step C-1, estimate the required flexural strength: Using an R of 5, estimate the required strength of the PreWEC at the base using the elastic response spectrum specified by ASCE 7-10.[7] Follow the classical FBD method (for example, ASCE 7-10[7]) for walls and use the elastic response spectrum corresponding to design-level earthquakes. Apply a strength reduction factor ϕ of 0.9 to estimate the nominal capacity.

- Step C-2, evaluate the area and initial stress of post-tensioning tendons in the wall panel: Determine the required area of post-tensioning tendons using moment equilibrium of forces acting at the base of the PreWEC, assuming the plastic capacity of the energy-dissipating elements (Fig. 2) and 95% of the yield strength of post-tensioning tendons at the design drift. To maintain the self-centering capability of the system up to the design drift, choose the initial stress for post-tensioning tendons such that they remain elastic until the chosen target drift. Estimate the elongation of the tendons by calculating the neutral axis depth at the base of the wall panel.[23,24]

- Step C-3, evaluate the area and initial stress of post-tensioning tendons in the end columns: Design the area of post-tensioning tendons connecting the end columns to the foundation to resist the ultimate connector force per joint, with a safety factor varying between 1.25 and 1.5.[23] The maximum permissible initial prestress in the end columns is $0.8f_{pu}$, per ACI 318-11,[6] where f_{pu} is the ultimate strength of the post-tensioning tendons.

- Step C-4, provide additional detailing: Design the confined boundary elements at the wall corners to satisfy the provisions of ACI 318-11.[6] Detail the walls to provide the minimum amount of shear stirrups and longitudinal reinforcement required by ACI 318-11. Additional guidance for the confinement reinforcement may be found in Aaleti[23] and Aaleti and Sritharan.[24]

Proposed design modifications

Appropriate modifications to the current design approach are presented in the following sections. They were determined using ITG-5 documents[5,22] as the basis along with the test observations reported in Nazari et al.[20] and Nazari and Sritharan.[21] The proposed modifications will facilitate the design of SRWs and PreWECs with varying amounts of hysteretic damping in high seismic regions. Proposed equations to estimate equivalent damping of the walls and revised damping-dependent R factors are presented first, followed by the improved design steps.

Recommended damping-dependent R factors

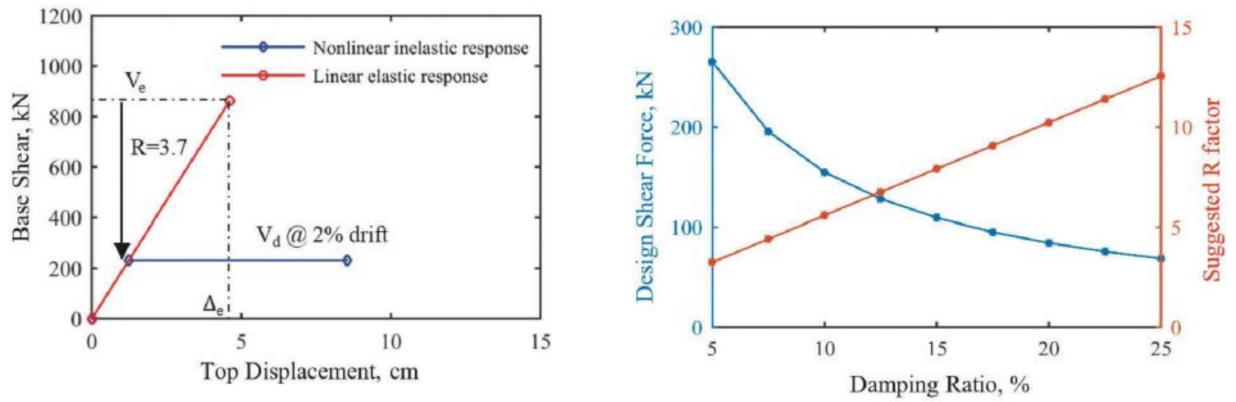
As previously noted, ITG-5.1[5] recommends an R of 5, which was originally proposed for CIP walls. Using the same value for precast concrete wall systems is irrational because the two systems have different ductility and energy-dissipation capacities. Furthermore, the amount of hysteretic damping can be easily altered in precast concrete wall systems. Therefore, new R factors have been developed for such systems as a function of total equivalent damping ratio in the system following the procedure described in ASCE 7-10 and the International Code Council's *2012 International Building Code*.[7,25] While developing the proposed R factors, it was determined that different R factors are necessary for designing buildings to withstand FF and NF earthquakes, as should be expected for any structural system. This led to Eq. (1) and (2) for obtaining appropriate R factors for SRWs and PreWEC systems.[21]

For FF earthquakes,

$$R = 0.46 \times \xi_{eq} + 0.93 \tag{1}$$

For NF earthquakes,

$$R = 0.15 \times \xi_{eq} + 2.65 \tag{2}$$



Single rocking wall

Precast concrete wall with end columns

Figure 4. R factors suggested for force-based design of precast concrete rocking wall systems—FF records.
Source: Data for right graph from Nazari and Sritharan (2018).
Note: FF = far-field motion; R = response modification coefficient; V_d = design-level shear resistance; V_e = base shear force corresponding to the elastic response of the system; Δ_e = elastic response displacement. 1 cm = 0.394 in.; 1 kN = 0.225 kip.

where

ξ_{eq} = total equivalent damping ratio of the system expressed in percentage

ξ_{eq} could vary from about 6% for SRWs to larger values for PreWECs, depending on the amount of supplemental damping provided by the connectors. The preceding equations were derived by estimating the ratio of the base shear forces corresponding to the elastic response of the system V_e and the design-level shear resistance V_d using FBD (that is, ASCE 7-10[7] with R = 1) and DDBD (for a selected damping ratio of ξ_{eq} at design drift of 2%), respectively, as shown in **Fig. 4** for the SRWs with ξ_{eq} of 6% for FF earthquakes. In this figure, the elastic response displacement Δ_e of 460 mm (1.8 in.) is related to an elastic period of 0.9 seconds, as estimated for the prototype system.[20] Ignoring the overstrength due to negligible second slope of the rocking system, the nonlinear response was found from the DDBD approach using the corresponding damping ratio of SRWs. The same methodology was adopted to define the design shear forces of PreWECs with varying ξ_{eq}. Figure 4 also presents the suggested R factors for PreWEC units with total damping ratios varying between 5% and 25% for FF earthquakes.[21]

As outlined through the shake-table study,[20,21] ξ_{eq} at design drift level consisted of the following:

- 4.2% damping due to the material nonlinearity of concrete at wall toes and inherent viscous damping of the system
- 1% and 1.5% damping due to impacts for PreWECs and SRWs, respectively
- additional equivalent damping of PreWEC systems due to hysteretic action of O connectors

Eq. (3) is suggested to determine the amount of equivalent damping for the hysteretic action of connectors $\xi_{conn.D\%}$ as a function of the number and response characteristics of connectors, as well as shear capacity of the rocking wall system.[21]

$$\xi_{conn.D\%} = \frac{N_{conn.} \times F_{c,ave} \times \left(l_{con} - \dfrac{\Delta_{c,y}}{D\%} \right)}{\pi \times V_{D\%} \times H_s} \tag{3}$$

where

$N_{conn.}$ = total number of connectors

$F_{c,ave}$ = 0.5 × ($F_{c,y}$ + $F_{c,D\%}$), where $F_{c,y}$ is the yield strength of the connector and $F_{c,D\%}$ is the force in the connector when the PreWEC is subjected to design drift $D\%$ (assuming that an elastoplastic response of the connector will lead to $F_{c,ave} = F_{c,y}$ [Fig. 2])

l_{con} = the distance to the center of the connector leg attached to the uplifting end of the wall panel measured from the neutral axis; the neutral axis depth of the panel can be estimated from the recommendations of Aaleti and Sritharan[24]

$\Delta_{c,y}$ = yield displacement of connectors

$D\%$ = design drift ratio

$V_{D\%}$ = shear resistance of the wall system at the design drift ratio

H_s = seismic height

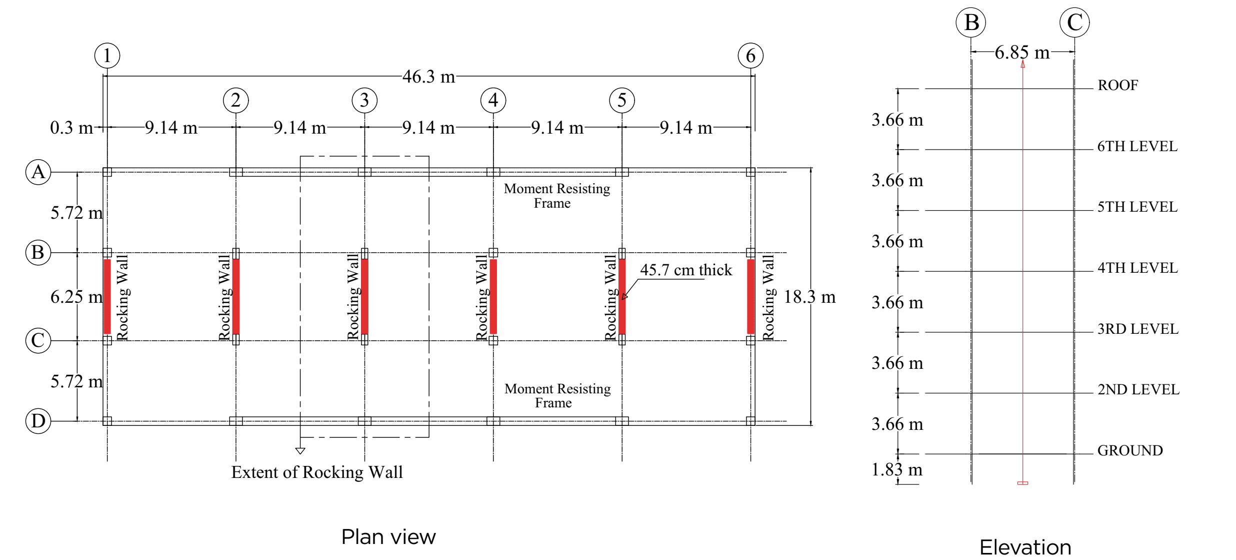


Figure 5. Plan view of buildings and elevation of a six-story building. Note: 1 cm = 0.394 in.; 1 m = 3.28 ft.

Any given range of $\xi_{conn.D\%}$ could be used to design PreWECs and the corresponding R factors. This allows for more flexibility in the design of precast concrete rocking wall systems. Equation (1) and Fig. 4 show that SRWs could be designed with an R of 3.7 for FF records when the system has a damping ratio of 6%. Although design forces of SRWs are expected to be greater in this case than with a PreWEC option with ξ_{eq} of 10%, the SRW option may be preferred for constructibility reasons. The damping-dependent R concept is ignored in current design guidelines, including ITG-5.[5,22]

Improved design steps

Modified steps to design both SRWs and PreWECs, considering Eq. (1) to (3), are presented as follows:

- Step M-1, select a damping ratio for the system: Use a damping ratio of 6% for SRWs.[20] Considering a minimum R factor of 4 and to preserve the self-centering capability of the system, a damping ratio in the range of 9% to 20%[21] is suggested for the design of PreWECs, which will be provided with a specific number of O connectors with predefined force-deformation properties.[23]

- Step M-2, estimate R and the required flexural strength: After selecting the equivalent damping ratio for the rocking wall system, use either Eq. (1) or Eq. (2) to determine R. Estimate the required base shear resistance following the FBD method with the calculated R factor. Design the wall system such that

$$M_n \geq \frac{M_{design}}{\phi}$$

where

M_n = nominal moment capacity of the wall system at design drift

M_{design} = required base moment resistance of the wall system at the design drift

ϕ = flexural strength reduction factor

- Step M-3, determine the number of O connectors per joint: First, establish an appropriate force-deformation response of the connector (Fig. 2). Then, define the required number of O connectors using Eq. (3).

- Step M-4, estimate the area and initial stress of post-tensioning tendons: Follow the approach described in the current design approach section, step C-2 for the wall panel. For PreWECs, design the post-tensioning in the end columns as well according to step C-3.

- Step M-5, provide additional detailing: Use the approach described in step C-4.

The preceding improved design procedure can also be used to ensure satisfactory performance of rocking wall systems subjected to MCE events. To complete this design, a macro-based

Excel program has been developed for the analysis and design of the rocking wall systems and is available at http://sri.cce.iastate.edu/NEES-Rocking-Wall/.

Parametric study

Using an experimentally verified analytical modeling technique developed in OpenSees (http://opensees.berkeley.edu), a parametric study is presented in the following sections to verify the accuracy of the proposed R factors for designing precast concrete rocking wall systems.

Prototype buildings

Six-, nine-, and twelve-story office buildings were designed in San Diego, Calif., which is seismic zone 4 and site class C. The typical plan view of the case study buildings is shown in **Fig. 5**. Precast concrete rocking wall systems (SRW or PreWEC) were used as seismic-resisting elements for these buildings in the transverse direction. The number of wall systems ranged from three to six depending on the value of R. A typical elevation of the wall system is also shown in Fig. 5 for a six-story case study building. The effective weight per unit area of each story was taken as 8.38 kPa (175 lb/ft^2). The total lengths of the SRW and PreWEC wall panels were 6.85 and 5.33 m (22.5 and 17.5 ft), respectively, with a 760 × 760 mm (30 × 30 in.) column connected to either end of the PreWEC wall panel. The unbonded length of the post-tensioning tendons was 1.8 m (5.9 ft) longer than the height of the wall panels.

All precast concrete rocking wall systems were designed with target design drifts corresponding to DBE and MCE events. For each event type, two designs were completed, one for FF and the other for NF ground motions, using the appropriate value for R from Eq. (1) and (2). The two target drifts and two ground motion types led to four designs for each building. Design details are summarized in **Tables 1–4.**

Building designations reported in Table 1 indicate the number of stories and type of wall system. (For example, SRW6 is a six-story building with SRWs as the lateral-load-resisting elements.) Table 1 shows that SRWs with a lower energy dissipation capacity than PreWECs resulted in larger design forces for the buildings. Depending on the calculated design force, buildings were designed with four to six SRWs. The area of post-tensioning tendons and the initial prestress in the SRWs varied from 8800 mm^2 (13.64 in.^{2}) and $0.65f_{pu}$ to 20,900 mm^2 (32.4 in.^{2}) and $0.82f_{pu}$, where f_{pu} is the tensile strength of the post-tensioning tendon, which was 1862 MPa (270 ksi). The variation in the initial post-tensioning stress was necessary because it was dictated by the required moment resistance and the need to keep the tendon stress elastic at the design drift to ensure self-centering behavior. As explained in this table, the maximum number of SRWs (six) was not adequate to design the 12-story building for MCE events.

PreWEC systems with ξ_{eq} of 13% and 18% were also used for resisting seismic loads of the multistory buildings (Tables 2–4). In this case, a designation of either PreWEC*n*-13 or PreWEC*n*-18 was used, depending on the value of ξ_{eq} (13% or 18%), with *n* representing the number of stories in the building. Table 2 shows that the PreWEC systems were designed with fewer post-tensioning tendons compared with the SRWs because of the following:

- Higher damping in PreWECs led to lower design base shear forces.

- Part of the moment resistance in PreWECs is provided by the forces developed in the O connectors.

The area of post-tensioning tendons and the initial prestress in the PreWECs varied from 1200 mm^2 (1.86 in.^{2}) and $0.68f_{pu}$ to 24,800 mm^2 (38.44 in.^{2}) and $0.86f_{pu}$. The latter values were required for the design of the 12-story building for NF-MCE events. Table 3 presents the design details of end columns and O connectors for all of the PreWEC systems. Two different

Table 1. Design of the case study buildings with single rocking walls

Building ID	R	Number of SRWs	Design force per wall $V_{D\%}$, kN				Area of post-tensioning tendons A_p, mm^2/initial post-tensioning stress f_{pi}, MPa, in the wall panels			
	FF/NF	DBE/MCE	FF-DBE	NF-DBE	FF-MCE	NF-MCE	FF-DBE	NF-DBE	FF-MCE	NF-MCE
SRW6*	3.7/3.6	4/6	2595	2694	2595	2694	8800/$0.65f_{pu}$	9200/$0.66f_{pu}$	9900/$0.65f_{pu}$	10,500/$0.66f_{pu}$
SRW9	3.7/3.6	4/6	2975	3094	2975	3094	17,900/$0.78f_{pu}$	19,000/$0.78f_{pu}$	19,900/$0.78f_{pu}$	20,900/$0.78f_{pu}$
SRW12	3.7/3.6	6/n.d.†	2130	2214	n.d.	n.d.	18,600/$0.81f_{pu}$	19,600/$0.82f_{pu}$	n.d.	n.d.

Note: The dead load carried by the walls is considered to be an additional post-tensioning force and varies for different heights and number of walls in the building. DBE = design-basis earthquake; f_{pu} = ultimate strength of the post-tensioning tendons; FF = far-field ground motion; ID = identification; MCE = maximum considered earthquake; n.d. = no data; NF = near-field ground motion; R = response modification coefficient; SRW = single rocking wall. 1 mm^2 = 0.00155 in.^{2} ; 1 kN = 0.225 kip; 1 MPa = 0.145 ksi.

* SRW6 is a six-story building with SRWs as the lateral load-resisting elements.

† The maximum number of SRWs (six) was not adequate to design the 12-story building for MCE events.

Table 2. Design of the case study buildings with precast concrete wall with end columns—wall panel design

Building ID	R	Number of PreWECs		Design force per wall $V_{D\%}$, kN				Area of post-tensioning tendons A_p, mm²/initial post-tensioning stress f_{pi}, MPa, in the wall panels			
	FF/NF	DBE/MCE		FF-DBE	NF-DBE	FF-MCE	NF-MCE	FF-DBE	NF-DBE	FF-MCE	NF-MCE
		FF	NF								
PreWEC6-13*	7.0/4.7	3/4	4/6	1723	1939	1944	1939	4800/0.7f_{pu}	5800/0.71f_{pu}	5900/0.71f_{pu}	7100/0.71f_{pu}
PreWEC9-13	7.0/4.7	3/5	5/6	1977	1774	1779	2219	10,000/0.8f_{pu}	8400/0.79f_{pu}	8400/0.79f_{pu}	14,100/0.81f_{pu}
PreWEC12-13	7.0/4.7	4/5	5/6	1596	1910	1915	2386	10,500/0.83f_{pu}	14,800/0.85f_{pu}	15,000/0.85f_{pu}	24,800/0.86f_{pu}
PreWEC6-18*	9.3/5.4	3/4	4/6	1252	1599	1404	1599	1200/0.68f_{pu}	2800/0.69f_{pu}	2100/0.68f_{pu}	4100/0.69f_{pu}
PreWEC9-18	9.3/5.4	3/5	5/6	1433	1465	1289	1839	3900/0.77f_{pu}	3900/0.77f_{pu}	2900/0.77f_{pu}	8300/0.78f_{pu}
PreWEC12-18	9.3/5.4	3/5	5/6	1541	1580	1384	1971	6700/0.82f_{pu}	7000/0.82f_{pu}	5320/0.82f_{pu}	13,300/0.83f_{pu}

Note: The dead load carried by the walls is considered as an additional post-tensioning force and varies for different heights and number of walls in the building. DBE = design-basis earthquake; f_{pu} = ultimate strength of the post-tensioning tendons; FF = far-field ground motion; ID = identification; MCE = maximum considered earthquake; NF = near-field ground motion; R = response modification coefficient; PreWEC = precast concrete wall with end columns. 1 mm² = 0.00155 in.²; 1 kN = 0.225 kip; 1 MPa = 0.145 ksi.

* The *n*-story building designed with PreWECs was named either PreWECn-13 or PreWECn-18, depending on the provided damping.

Table 3. Design of the case study buildings with precast concrete wall with end columns: Connectors and end column design

Building ID	Type and number of connectors per joint				Area of post-tensioning tendons $A_{p,col}$, mm²/initial post-tensioning stress $f_{pi,col}$, MPa, in the end columns			
	FF-DBE	NF-DBE	FF-MCE	NF-MCE	FF-DBE	NF-DBE	FF-MCE	NF-MCE
PreWEC6-13	A-22	A-24	A-24	A-24	18,200/0.8f_{pu}	19,600/0.8f_{pu}	19,600/0.8f_{pu}	19,600/0.8f_{pu}
PreWEC9-13	A-38	A-34	A-34	A-42	30,800/0.8f_{pu}	28,000/0.8f_{pu}	28,000/0.8f_{pu}	34,000/0.8f_{pu}
PreWEC12-13	A-40	A-48	A-48	A-60	32,000/0.8f_{pu}	39,200/0.8f_{pu}	39,200/0.8f_{pu}	47,600/0.8f_{pu}
PreWEC6-18	B-24	B-32	B-28	B-32	19,600/0.8f_{pu}	25,200/0.8f_{pu}	22,400/0.8f_{pu}	25,200/0.8f_{pu}
PreWEC9-18	B-42	B-44	B-38	B-54	33,600/0.8f_{pu}	35,000/0.8f_{pu}	30,800/0.8f_{pu}	43,400/0.8f_{pu}
PreWEC12-18	B-60	B-62	B-54	B-78	47,600/0.8f_{pu}	50,400/0.8f_{pu}	43,400/0.8f_{pu}	63,000/0.8f_{pu}

Note: DBE = design-basis earthquake; f_{pu} = ultimate strength of the post-tensioning tendons; FF = far-field ground motion; ID = identification; MCE = maximum considered earthquake; NF = near-field ground motion; PreWEC = precast concrete wall with end columns. 1 mm² = 0.00155 in.²; 1 MPa = 0.145 ksi.

Table 4. Design of the case study buildings with precast concrete wall with end columns: Force deformation properties of O connectors

Connector ID	Force, kN		Deformation, mm	
	Yield	Ultimate	Yield	Ultimate
A	75.6	80.1	6.4	102
B	75.6	80.1	2.5	102

Note: ID = identification. 1 mm = 0.0394 in.; 1 kN = 0.225 kip.

Table 5. List of input motions used for analysis of the case study buildings

Earthquake				Scale factor for DBE		Scale factor for MCE	
Name (year, station)	Component 1	Component 2	PGA_{max}, g	Component 1	Component 2	Component 1	Component 2
Far-field ground motions							
Hector Mine, Calif. (1999, Hector)	HEC000	HEC090	0.34	1.70	1.30	2.50	1.80
Kobe, Japan (1995, Nishi-Akashi)	NIS000	NIS090	0.51	1.33	1.20	2.00	2.00
Kocaeli, Turkey (1999, Arcelik)	ARC000	ARC090	0.22	4.50	n.d.*	6.00	n.d.*
Manjil, Iran (1990, Abbar)	ABBAR--L	ABBAR--T	0.51	1.30	0.85	2.00	1.40
Chi-Chi, Taiwan (1999, TCU045)	TCU045-E	TCU045-N	0.51	2.00	1.50	2.50	2.10
Friuli, Italy (1976, Tolmezzo)	A-TMZ000	A-TMZ270	0.35	2.90	n.d.*	4.00	n.d.*
Near-field ground motions							
Loma Prieta, Calif. (1989, Saratoga–Aloha)	STG_038	STG_128	0.38	1.00	1.60	1.50	2.30
Cape Mendocino, Calif. (1992, Petrolia)	PET_260	PET_350	0.63	n.d.*	0.60	n.d.*	0.90
Landers, Calif. (1992, Lucerne)	LCN_239	LCN_329	0.79	0.75	1.70	1.20	2.50
Northridge-01, Calif. (1994, Sylmar)	SYL_032	SYL_122	0.73	0.75	0.55	1.00	0.70
Chi-Chi, Taiwan (1999, TCU102)	TCU102_278	TCU102_008	0.29	0.65	0.65	1.20	1.10
Gazli, USSR (1976, Karakyr)	GAZ_177	GAZ_267	0.71	0.85	0.90	1.10	1.40
Nahanni, Canada (1985, Site 1)	S1_070	S1_160	1.18	1.60	1.65	2.30	2.50
Nahanni, Canada(1985, Site 2)	S2_070	S2_160	0.45	3.00	2.00	4.50	2.90
Loma Prieta, Calif. (1989, BRAN)	BRN_038	BRN_128	0.64	1.15	n.d.*	1.50	n.d.*
Loma Prieta, Calif. (1989, Corralitos)	CLS_038	CLS_128	0.51	1.30	n.d.*	2.00	n.d.*
Cape Mendocino, Calif. (1992, Cape Men.)	CPM_260	CPM_350	1.43	0.70	1.45	1.00	2.30
Northridge-01, Calif. (1994, LA)	0637_032	0637_122	0.73	n.d.*	0.70	n.d.*	1.10
Chi-Chi, Taiwan (1999, TCU067)	TCU067_285	TCU067_015	0.56	0.75	0.75	1.10	1.20
Chi-Chi, Taiwan (1999, TCU084)	TCU084_271	TCU084_001	1.16	0.80	n.d.*	1.20	n.d.*
Denali, Alaska (2002, TAPS Station 10)	ps10_199	ps10_289	0.33	0.55	0.90	0.80	1.30

Source: Data from Federal Emergency Management Agency (2009).

Note: DBE = design-basis earthquake; g = acceleration due to gravity; MCE = maximum considered earthquake; n.d. = no data; PGA_{max} = peak ground acceleration.

* Scaled records were not used in the analysis.

types of connectors, designated as connectors A and B, were used for PreWEC*n*-13 and PreWEC*n*-18, respectively, and their force-deformation properties are noted in Table 4. Connector B was designed with a smaller yielding displacement to provide a larger hysteretic damping ratio for PreWEC*n*-18 buildings.

Selection and scaling of ground motions

Each building design chosen for the parametric study was subjected to a total of 35 ground motion records corresponding to site class C selected from the Federal Emergency Management Agency's *Quantification of Building Seismic Performance Factors*, FEMA P695.[26] **Table 5** summarizes the record set that includes 10 FF and 25 NF records. The records were scaled to appropriately represent either EQ-III or EQ-IV seismic hazard levels, representing DBE and MCE events. As presented in the Structural Engineers Association of California's *Recommended Lateral Force Requirements and Commentary*[27] and consis-

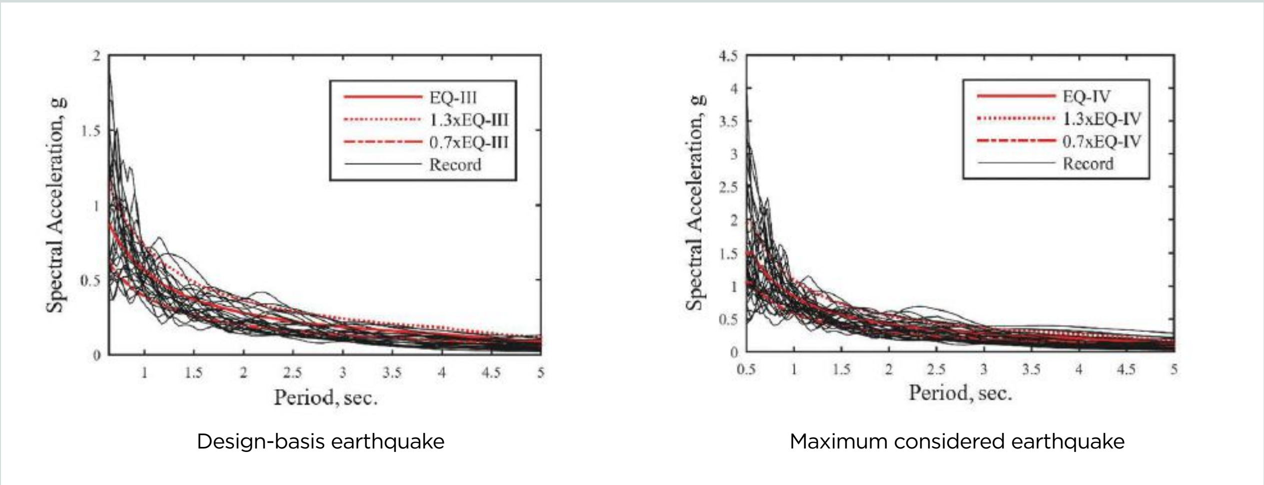


Figure 6. Spectral acceleration of scaled near-field and far-field ground motion records selected for analysis. Note: EQ-III = seismic hazard level III, representing design-basis earthquake events; EQ-IV = seismic hazard level IV, representing maximum considered earthquake events; *g* = acceleration of gravity.

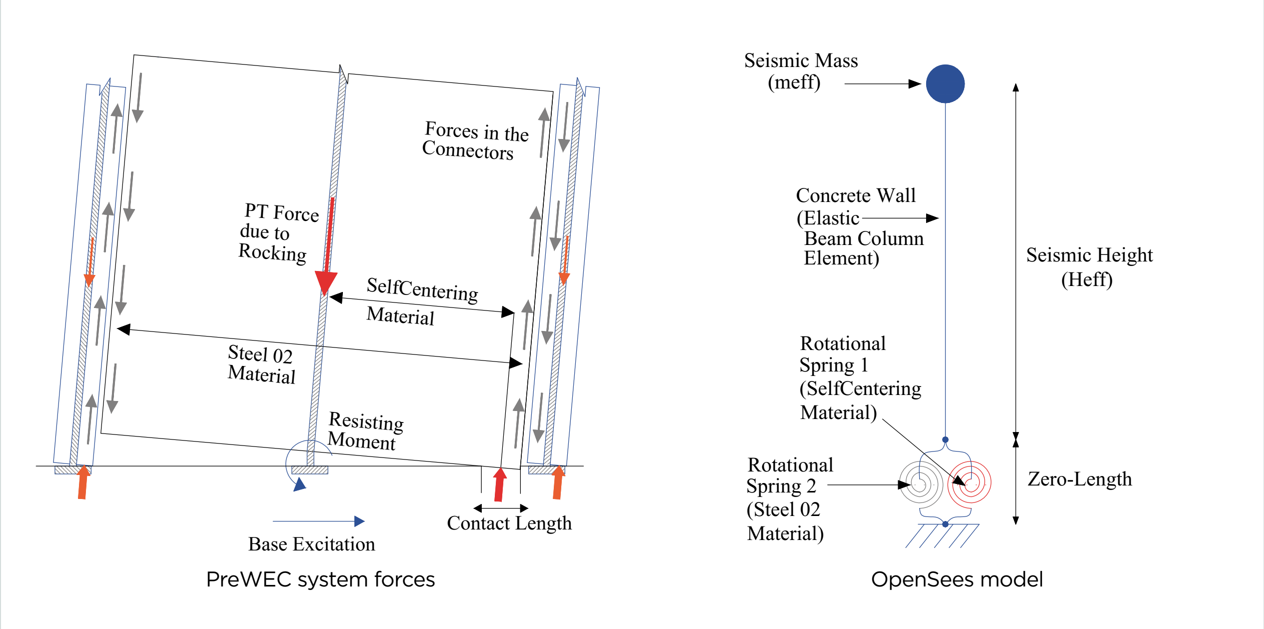


Figure 7. Forces contributing to base moment resistance in a PreWEC system and the corresponding OpenSees model.
Source: Reproduced by permission from Nazari and Sritharan (2018).
Note: PreWEC = precast concrete wall with end columns; PT = post-tensioning.

tent with the requirements of ASCE 7-10,[7] the seismic hazard levels are defined by 5% damped elastic acceleration response spectra with peak ground accelerations of 1g and 1.5g for DBE and MCE events, respectively, for the selected location of the buildings. The scale factors (Table 5) were determined for the ground motions such that the root mean square deviation of their spectral ordinates remains within ±30% of the mean of the targeted hazard spectrum over the rocking period range of the buildings (0.5 to 5 seconds). More details about the scaling procedure are presented in Nazari et al.[20] **Figure 6** presents the acceleration response spectra of the scaled records, including FF and NF, representing DBE and MCE events.

Analytical model

All buildings in the parametric study were modeled in OpenSees following an approach that has been validated using experimental data.[20,21] For this investigation, only one wall from the building was chosen and subjected to in-plane seismic loading. The wall system was represented by a single degree of freedom (SDOF) model, as this is appropriate for rocking wall systems because the response of these systems is predominantly controlled by a linear rocking mode. To obtain accurate results, the model needs to be characterized with the effective mass m_{eff} and effective height H_{eff} of the equivalent SDOF model. The following example is for a six-story building with four SRWs designed for a target drift of 2% to obtain satisfactory results under DBE. In the following equations, m_i and Δ_i represent the mass and displacement of each floor in the n-story building.

$$H_{eff} = \frac{\dfrac{\sum_{i=1}^{n}\left(m_i\Delta_i^2\right)}{\sum_{i=1}^{n}\left(m_i\Delta_i\right)}}{0.02}$$

$$= 0.722 \times H_n = 0.722 \times 6 \times 3.66$$

$$= 15.86 \text{ m (52 ft)}$$

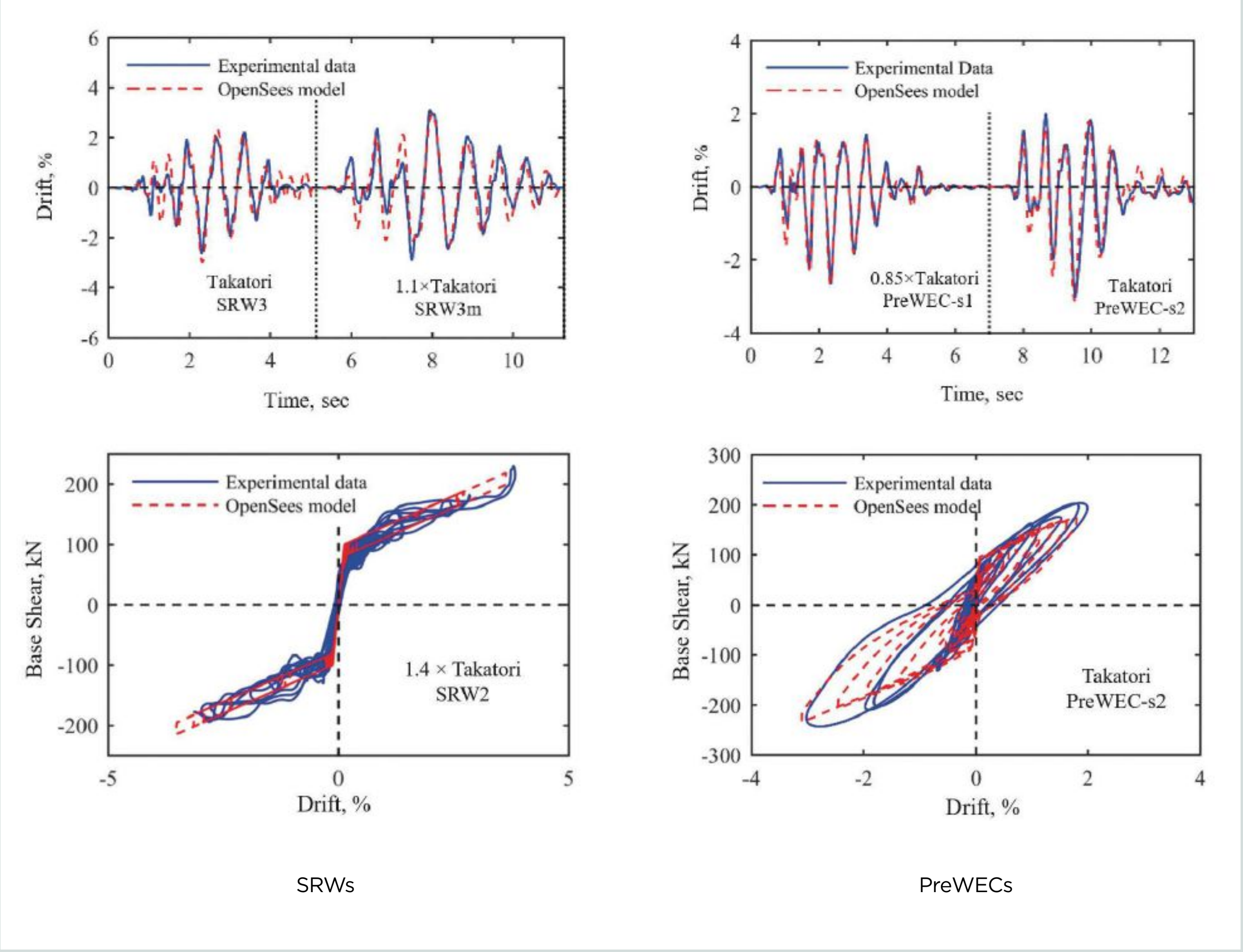


Figure 8. Experimental verification of the OpenSees models of SRWs and PreWECs.
Source: Data for left graphs (top and bottom) from Nazari, Sritharan, and Aaleti (2016); data for right graphs (top and bottom) from Nazari and Sritharan (2018). Note: 1 kN = 0.225 kip.
Note: PreWEC = precast concrete wall with end columns; SRW = single rocking wall.

$$m_{eff/Wall} = \frac{\dfrac{\sum_{i=1}^{n}\left(m_i \Delta_i\right)}{0.02 \times H_{eff}}}{4}$$

$$= 4.85 \times \frac{m_i}{4} = 4.85 \times 847.3 \times \frac{8.38}{4}$$

$$= 8609 \text{ kN (1935 kip)}$$

Key details of the OpenSees model of a PreWEC system are presented in **Fig. 7**. Figure 7 shows that the effective seismic mass m_{eff} is placed at the top of an elastic beam-column element with an effective height H_{eff} representing the precast concrete wall. The beam-column element is attached to the foundation using a zero-length rotational spring system consisting of the following:

- a SelfCentering material model, which captures the wall base moment resistance resulting from the post-tensioning and the associated self-centering behavior and the limited damping due to concrete nonlinearity
- a Steel02 material model, which characterizes the moment resistance and hysteretic energy dissipation of the O connectors

These two rotational springs act in parallel and capture the total response of PreWEC systems. The same modeling concept is used for all SRWs except that the second spring modeled with Steel02 is eliminated. More information about the analytical models, including material models, is available in Nazari et al.[20] and Nazari and Sritharan.[21]

The lateral drift time history responses of two SRW and two PreWEC test units from the shake-table investigations[12–21] were compared with the analytical results from the OpenSees models. **Figure 8** presents this comparison and confirms the accuracy of the OpenSees models in satisfactorily capturing the responses, including the peak lateral drifts. The same observations were made when plotting the lateral load-drift hysteretic response of the wall systems, as also shown in Fig. 8 for one SRW and one PreWEC test unit.

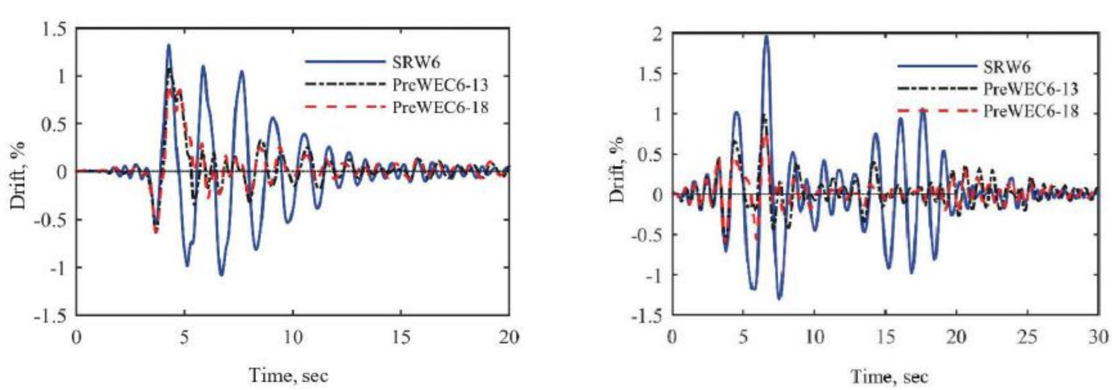
Dynamic analysis results

Dynamic analyses of all precast concrete rocking wall system designs (Tables 1–4) were conducted using OpenSees models and the ground motions listed in Table 5 to determine the maximum responses for the case study buildings. For all analyses, the Newmark constant average acceleration solution method was used with an integration time step of 0.005 to 0.02 seconds; the selected time step corresponded to the time step available for the respective ground motions. Using the tangent (current) stiffness proportional Rayleigh damping in the models, 2% and 3% elastic viscous damping ratios were included in the analysis of PreWECs and SRWs, respectively, to represent the estimated damping of the walls due to impacts.[20,21] Due to space limitations, only the DBE analysis results of the buildings are presented in this paper.

Lateral drift

Figure 9 compares the lateral drift time history response of buildings designed with SRWs and PreWECs for two DBE records and demonstrates that the main benefit of the additional hysteretic damping in PreWECs compared with SRWs is faster decay of dynamic response, but the difference between 13% and 18% damping on the response of the PreWEC systems is not significant.

The maximum drift ratio d_{max} was computed for all buildings as the ratio of the resulting maximum lateral drift to the acceptable



FRIULI/A-TMZ000 record (FF-DBE)

CHICHI/TCU067_285 record (NF-DBE)

Figure 9. Impact of additional hysteretic damping on the drift time history response for two earthquake records. Note: DBE = design-basis earthquake; FF = far-field ground motion; NF = near-field ground motion; PreWEC = precast concrete wall with end columns; SRW = single rocking wall.

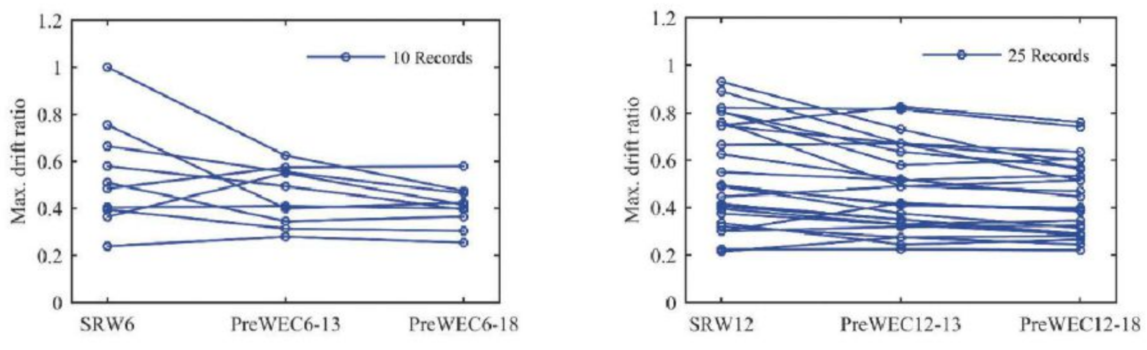
performance limits. The permissible drifts of 2% and 3% were used for the DBE and MCE input motions, as recommended by Rahman and Sritharan.[28] **Figure 10** plots d_{max} for six- and twelve-story buildings for both the FF-DBE and NF-DBE events. The proposed design approach led to satisfactory performance of the precast concrete rocking wall systems in terms of the maximum drift by producing d_{max} less than or equal to 1 (Fig. 10). Similar results were observed for all other buildings, with d_{max} ranging from 0.22 to 1, and therefore the remainder of the paper focuses on the average of the maximum responses obtained for each building under DBE events, under MCE events, or for the combined sets of DBE and MCE earthquakes.

Figure 11 shows the correlation between the average of the maximum drift ratios d_{ave} of the buildings for the FF-DBE records as a function of building height. For example, d_{ave} for SRW6 is the average of d_{max} of the building obtained for the 10 FF records representing DBE events (that is, the 10 data points shown on the left side of Fig. 10). Figure 11 also presents the variation in d_{ave} of the buildings for the different types of motions (NF and FF) and intensities (DBE and MCE). Accordingly, relatively greater drift responses were observed during NF records compared with FF records, though all buildings satisfied the performance criteria. Figure 11 shows that the sensitivity of d_{ave} to the height of the building as well as type and intensity of the applied ground motion is relatively small. Figure 11 also illustrates the variation of d_{ave} as a function of the damping ratios of the buildings. Each data point in this figure is d_{ave} of the building with *n* stories, where *n* was 6, 9, or 12, during ground motions with a specific type and intensity (for example, FF and DBE). The maximum drift response of the structures generally decreased with an increase in the damping ratio from 6% for SRWs to higher values for PreWEC systems (Fig. 11). Increasing the damping ratio from 13% to 18% did not cause a significant reduction in the peak responses, again emphasizing that precast concrete rocking wall systems do not have to be designed with damping ratios similar to those expected for CIP walls. As shown in Fig. 11, the PreWECs had a d_{ave} of less than 0.5 even after these systems were designed with R ranging from 5 to 9 following the proposed design approach.

Absolute acceleration and residual drift

Rahman and Sritharan[28] also made recommendations for the allowable limits for the maximum absolute acceleration and residual drift of buildings for different levels of seismic excitations to ensure acceptable performance and self-centering capability. The allowable limits recommended for the maximum absolute acceleration and residual drift, respectively, are 1.2g and 0.5% for DBE events and 1.8g and 0.75% for MCE events. The absolute acceleration ratio a_{max} is the ratio of the calculated maximum values of absolute acceleration to the allowable performance limits and varied between 0.58 and 0.79 when buildings were subjected to different ground motions. Similarly, the residual drift ratio d_r is the ratio of the calculated maximum value of residual drift to the allowable performance limits and varied between 0.02 and 0.08. The average of a_{max} and the average of d_r are shown in **Fig. 12** as a function of the damping ratio of the buildings for all ground motions, as previously presented for d_{ave} in Fig. 11. For each set of records (either NF or FF events), the figures show five data points for each building, with a specific damping ratio that represents the average response ratios (a_{max} and d_r) of six-, nine-, and twelve-story buildings during DBE and MCE events. The results for 12-story buildings subjected to MCE events were excluded from this data set because they were not available for SRWs.

Figure 12 shows that the maximum absolute acceleration of all buildings satisfied the performance limits. This figure also indicates that the average a_{max} was relatively independent of the damping ratio of the system (the red dashed line is the average trend of a_{max} during NF and FF records), and the buildings



6-story buildings during FF-DBE

12-story buildings during NF-DBE

Figure 10. Ratio of the maximum drift to the allowable limit d_{max}. Note: DBE = design-basis earthquake; FF = far-field ground motion; NF = near-field ground motion; PreWEC = precast concrete wall with end columns; SRW = single rocking wall.

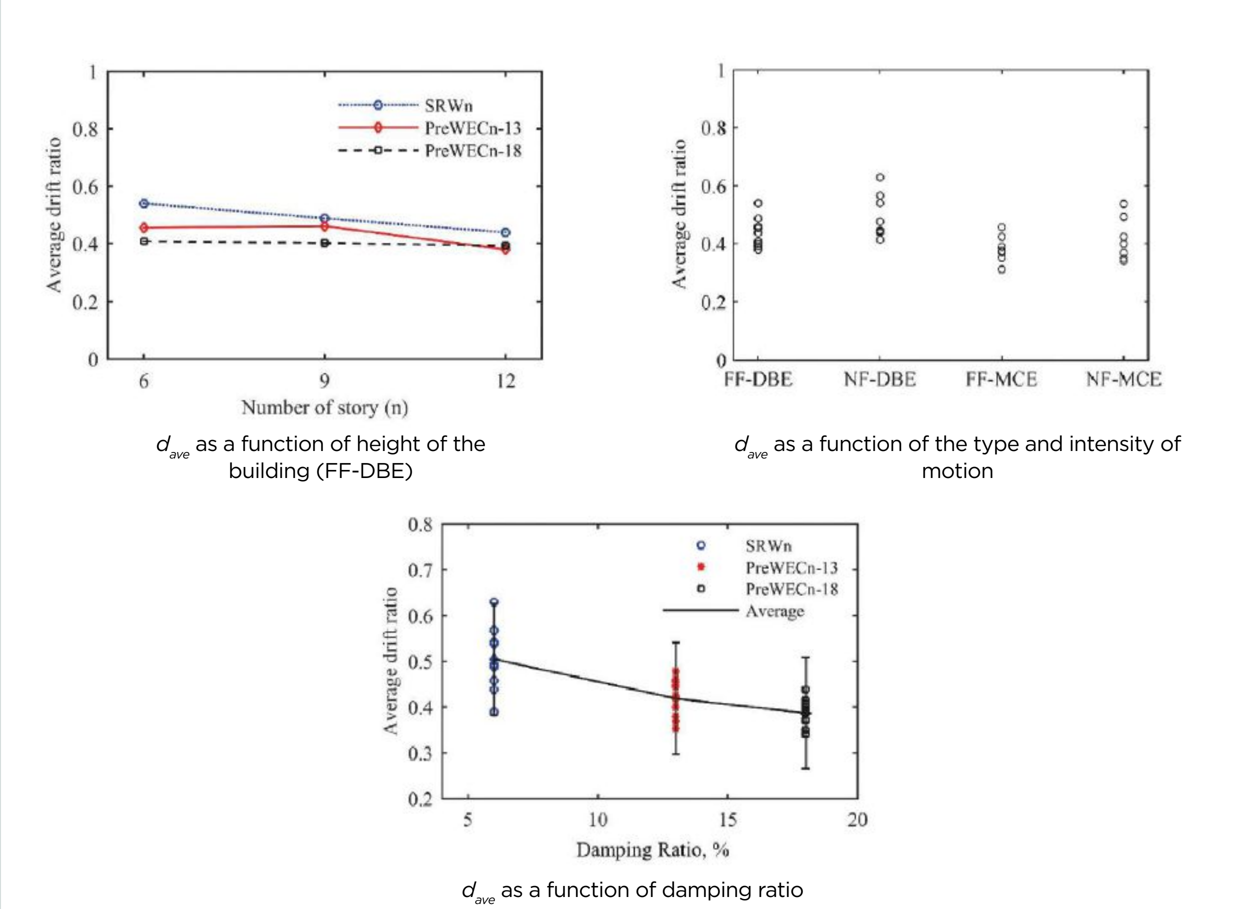


Figure 11. Average of the maximum drift ratio d_{ave} as a function of height of the building (FF-DBE), type and intensity of the motion, and damping ratio. Note: Error bars in the bottom portion of the figure represent ±2 standard errors of the mean, the 95% confidence interval. DBE = design-basis earthquake; FF = far-field ground motion; MCE = maximum considered earthquake; n = number of stories in the case study buildings; NF = near-field ground motion; PreWEC = precast concrete wall with end columns; SRW = single rocking wall.

consistently produced larger accelerations for the NF than for the FF events. Figure 12 also shows that the residual drift ratios of the buildings after being subjected to DBE and MCE input motions are consistently low, confirming that both SRWs and PreWECs are excellent lateral-load-resisting systems that produce self-centering seismic-resilient buildings.

Cost-effective design of precast concrete rocking wall systems

It has been shown that a precast concrete rocking wall system can be designed to perform satisfactorily in high seismic regions with equivalent damping as low as 6% or as high as 18% when a damping-dependent R is used. Therefore, the ultimate choice of a preferred system may be decided based on cost-effectiveness. A cost index has been developed to highlight the difference in costs of the buildings used in this parametric study. The cost index, which reflects the price of concrete and post-tensioning for the wall panels and end columns as well as the O connectors, is presented by the following equation:

$$P_{cost} = N_{wall} \times [(\alpha + \beta) \times (PT_{L,W} + PT_{L,col}) + \gamma \times N_{conn.w} + \delta \times V_C + P_{embeds} + P_{crane}]$$

where

P_{cost} = cost index

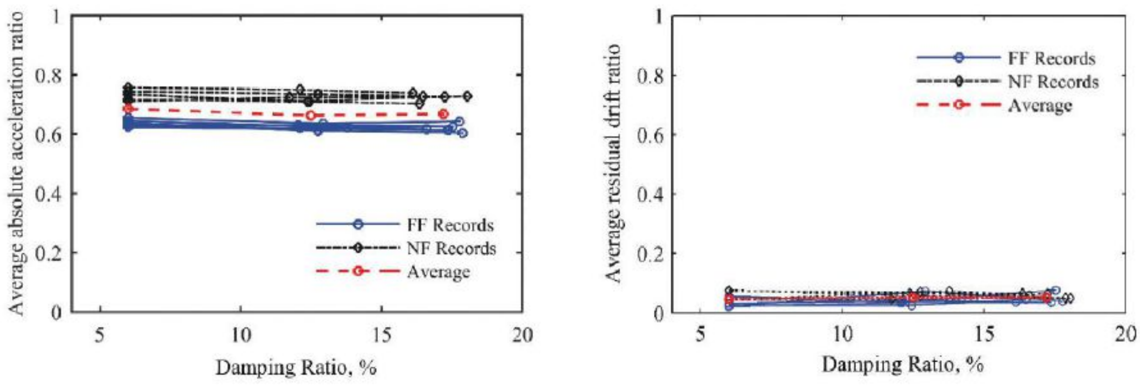
N_{wall} = number of wall systems per building

α = cost of material for post-tensioning, \$2.60 per meter (\$0.80 per foot) for 15.2 mm (0.6 in.) diameter tendons

β = cost of labor for post-tensioning, \$0.70 per meter (\$0.20 per foot), respectively, for 15.2 mm (0.6 in.) diameter tendons

$PT_{L,W}$ = length of post-tensioning tendons in the wall panel

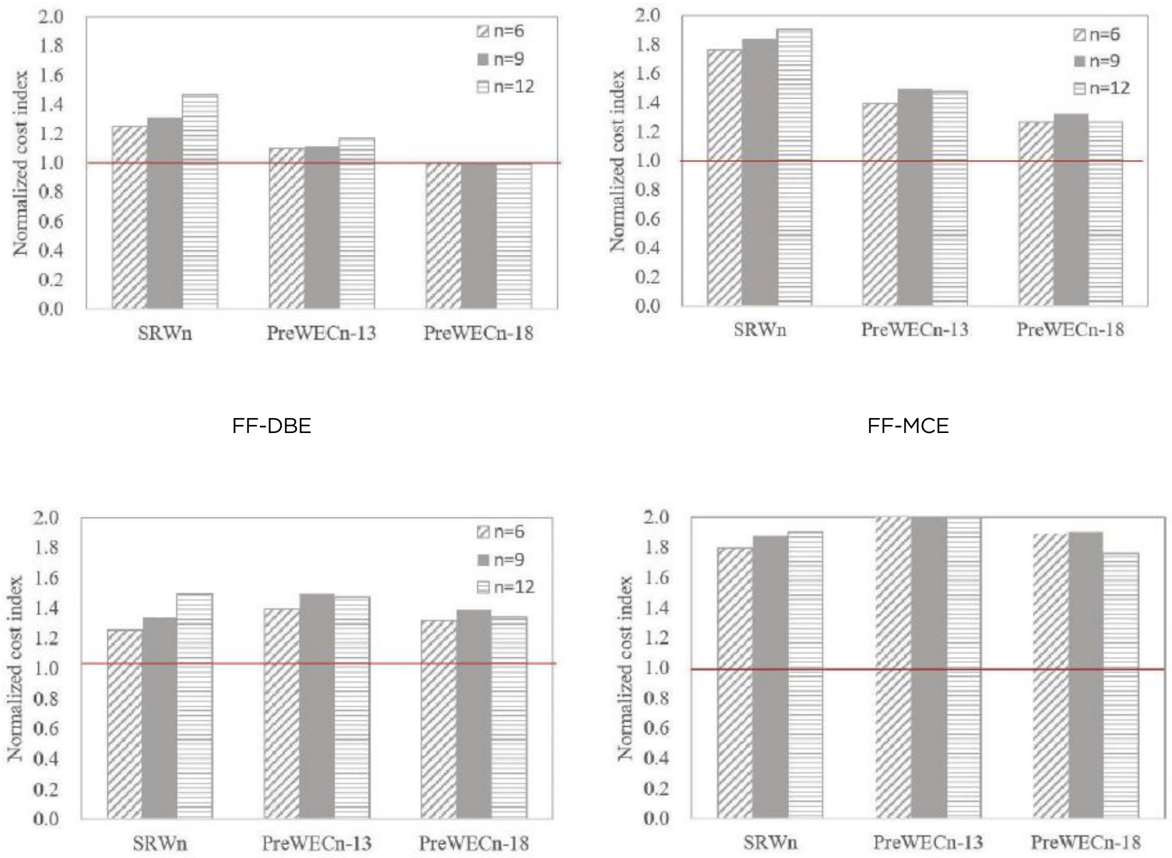
$PT_{L,col}$ = length of post-tensioning tendons in the end columns



Maximum absolute acceleration ratio

Residual drift ratio

Figure 12. Average response ratios of the calculated maximum values to the allowable limits. Note: FF = far-field ground motion; NF = near-field ground motion.



FF-DBE

FF-MCE

NF-DBE

NF-MCE

Figure 13. Normalized cost index for different case study buildings. Note: DBE = design-basis earthquake; FF = far-field ground motion; MCE = maximum considered earthquake; n = number of stories in the case study buildings; NF = near-field ground motion; PreWEC = precast concrete wall with end columns; SRW = single rocking wall.

γ = cost of material for O connectors; that is, \$30 per connector

$N_{conn.w}$ = number of connectors per wall

δ = cost of concrete for wall panels and end columns; that is, \$1308/m^3 (\$1000/yd^3), including the price of concrete, reinforcement, formwork, labor, and shipping from the precaster

V_C = concrete volume in cubic meters (cubic yards) for wall panels and end columns

P_{embeds} = installation cost for member bracing and embeds

P_{crane} = crane charges, using a Manitowoc 16000 crane with a unit price of \$4600 per day (For example, six PreWECs and SRWs in a six-story building are erected in six and two days, respectively.)

Figure 13 presents normalized cost indices for different design solutions established for the FF and NF events. The value 1 in these figures corresponds to the most economical design option for each building, whereas the most expensive design produced the maximum value, 2. For this comparative cost study, SRW12 was designed with seven walls for the MCE records. Despite the higher assembly costs for the PreWEC systems, it is clear that these systems with higher damping will be more cost-effective than SRWs for FF earthquake design. Due to the increased design base shear and despite using the same R factor, the cost of the systems will be higher if the MCE is used, which is consistent with the observation made by Rahman and Sritharan.[29] Similar observations hold for NF earthquakes except that the costs for SRWs and PreWECs are similar. PreWECs with lower damping do not appear to be cost-effective. The design solutions developed for NF earthquakes cost more than the solutions for FF earthquakes, which was expected due to the fact that a greater number of wall systems were required when designing structures for NF ground motions.

Conclusion

The first part of the study presented in this paper introduced an improved seismic design procedure for precast concrete rocking wall systems. The unique feature of the methodology is that it recognizes that the R factor used in FBD should be damping dependent, which in turn will make the FBD solutions comparable to those derived from DDBD and produce more cost-effective precast concrete building solutions. To validate proposed design modifications, a parametric study was conducted in the second part of the study using low- to midrise buildings. An experimentally validated OpenSees analytical approach was used to evaluate the seismic performance of six-, nine-, and twelve-story buildings with an identical floor plan when subjected to a set of NF and FF ground motions representing the DBE and MCE events. The buildings were designed to resist lateral forces in their transverse direction using rocking wall systems. The systems were designed with different R factors corresponding to their total damping ratios, varying from 6% (for an SRW) to 18% (for a PreWEC). The following conclusions were drawn based on the dynamic analysis results, the criteria established for performance-based seismic evaluation of rocking wall systems, and an index developed for cost-comparison purposes:

- The seismic performance of the structures with precast concrete rocking wall systems designed with the proposed R factors satisfied the performance limits for maximum lateral drift, residual drifts, and maximum floor acceleration for design-level and higher-intensity near-field and far-field earthquake motions. This confirms that using a damping-dependent R factor for the seismic design of precast concrete rocking wall systems is appropriate.

- PreWECs with larger hysteretic damping generally experienced lower maximum drifts with respect to the permissible limits, implying that although the R factor used in the design of these systems was as high as 9.3, the suggested R factors are indeed conservative. SRWs with the least amount of energy dissipation capacity responded satisfactorily, as designed with the R factor of 3.7.

- Drift ratios (that is, the maximum drift divided by the permissible value) generally decreased with increasing hysteretic damping ratios of the rocking wall buildings of different heights for all levels of applied ground motions. The additional hysteretic damping of PreWECs led to a faster decay of the building seismic response compared with SRWs.

- The absolute acceleration ratios were mostly unchanged for all buildings, but they were slightly higher for NF than for FF ground motions.

- All buildings were self-centered with negligible residual drift ratios of less than 0.1 for DBE and MCE ground motions.

- Comparing cost estimates of different buildings in this parametric study, PreWECs with high hysteretic damping are generally the most economical rocking wall systems for the design of low- to midrise buildings. SRWs are generally a less expensive option for NF earthquakes compared with PreWECs with lower damping ratios (that is, 13%).

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Notation

a_{max} = absolute acceleration ratio, the ratio of the maximum absolute acceleration to the acceptable performance limits

A_p = area of post-tensioning tendons in the wall panel

$A_{p,col}$ = area of post-tensioning tendons in the end columns

d_{ave} = average of the maximum lateral drift ratios

d_{max} = maximum lateral drift ratio, the ratio of the maximum lateral drift to the acceptable performance limits

d_r = residual drift ratio, the ratio of the residual drift to the acceptable performance limits

$D\%$ = design drift ratio

f_{pi} = initial post-tensioning stress in the wall panel

$f_{pi,col}$ = initial post-tensioning stress in the end columns

f_{pu} = ultimate strength of the post-tensioning tendons

$F_{c,ave}$ = average of $F_{c,y}$ and $F_{c,D\%}$

$F_{c,D\%}$ = force in the connector when the precast concrete wall with end columns is subjected to design drift ratio $D\%$

$F_{c,y}$ = yield strength of the connector

H_{eff} = effective height of the equivalent single-degree-of-freedom system

H_s = seismic height

l_{con} = distance to the center of the connector leg attached to the uplifting end of the wall panel measured from the neutral axis

m_{eff} = effective mass of the equivalent single-degree-of-freedom system

$m_{eff/Wall}$ = effective mass of the equivalent single-degree-of-freedom system per wall

m_i = mass of each floor in the n-story building

M_{design} = required base moment resistance of the wall system at the design drift

M_n = nominal moment capacity of the wall system at the design drift

n = number of stories in the case study buildings

$N_{conn.}$ = total number of connectors

$N_{conn.w}$ = number of connectors per wall

N_{wall} = number of wall systems per building

P_{cost} = cost index

P_{crane} = crane charges

P_{embeds} = installation cost for member bracing and embeds

PGA_{max} = peak ground acceleration

$PT_{L,col}$ = length of post-tensioning tendons in end columns

$PT_{L,W}$ = length of post-tensioning tendons in the wall panel

R = seismic response modification coefficient

V_C = concrete volume for wall panels and end columns

V_d = design-level shear resistance

$V_{D\%}$ = shear resistance of the wall system at the design drift ratio

V_e = base shear force corresponding to the elastic response of the system

α = cost of material for post-tensioning

β = cost of labor for post-tensioning

γ = cost of material for O-connectors

δ = cost of concrete for wall panels and end columns

$\Delta_{c,y}$ = yield displacement of connectors

Δ_e = elastic response displacement

Δ_i = displacement of each floor in the n-story building

$\xi_{conn.D\%}$ = equivalent damping for the hysteretic action of connectors

ξ_{eq} = total equivalent damping ratio of the system

f = strength reduction factor

About the authors

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Abstract

Following the design practice for cast-in-place concrete walls in which energy dissipation is a byproduct of the walls' seismic response, force-based seismic design of precast concrete rocking walls uses a response modification coefficient, or R factor, of 5 and a minimum equivalent viscous damping ratio of about 8%. However, single rocking walls (SRWs) with total damping of 6% and precast concrete walls with end columns (PreWECs) designed with as much as 16% damping showed satisfactory responses when subjected to shake-table testing. These findings suggest that the current design approach used for precast concrete rocking walls is unnecessarily restrictive and does not account for the superior behavior of the wall systems in design. To overcome this challenge, a damping-dependent R is proposed for the seismic design of precast concrete rocking walls and its effectiveness is demonstrated using a parametric study. A cost index is also developed to determine the relative benefits of SRWs and PreWECs.

Keywords

Parametric study, precast concrete walls with end column, PreWEC, response modification coefficient, R factor, rocking wall, seismic design, unbonded post-tensioning.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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