Available online at www.eccomasproceedia.org
Eccomas Proceedia COMPDYN (2017) 3426-3438

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COMPDYN 2017
6th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Rhodes Island, Greece, 15–17 June 2017

SEISMIC BEHAVIOR OF PRESTRESSED PRECAST SHEAR WALL BUILDINGS USING MULTIPLE ROCKING JOINTS

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Keywords: Self-centering, Multiple Rocking Joints, Structural Dynamics, Prestressed Precast Concrete Structures, OpenSEES.

Abstract. *The use of prestressed precast concrete shear walls (PPCW) as the primary lateral force resisting system in seismic regions has been studied in previous researches. The PPCW structures could concentrate the deformation at the joints and use the unbonded prestressed tendons to provide restoring force to achieve self-centering capacity, thus the residual drift is rather small. In this paper, analytical models of 16-story PPCW frame-shear wall structures designed using displacement-based approaches were established using OpenSEES to study the influence of location of joints and number of joints under both cyclic loading and earthquake loading. Case study buildings were analyzed under three levels of seismic hazards. The comparison was made between the behavior of the PPCW structures and the conventional cast-in-place concrete shear wall (CIPW) structures including the inter-story drifts, shear force distribution, base moment distribution and concrete crushing stress at contact edges of rocking sections. The results indicated that higher mode effects were mitigated on shear and moment actions and residual drifts of buildings were negligible.*

Peer-review under responsibility of the organizing committee of COMPDYN 2017.
doi: 10.7712/120117.5655.16797

1 INTRODUCTION

Resilient structural design is a recent development in earthquake-resistant structural systems. Self-centering structure is a branch of earthquake resilient structures. In comparison with traditional structures, self-centering structures characterize less energy dissipation, negligible structural damage and smaller residual deformation under severe earthquakes [1-2]. This system is featured by a softening force displacement response with minimal damage as a result of geometric nonlinearity. Self-centering capability, especially when combined with replaceable energy-dissipating elements, provide structures higher performance levels than conventional seismic lateral force resisting systems.

Prestressed precast concrete shear wall (PPCW) structures belong to self-centering structures. Figure 1 shows the typical details of a PPCW schematically. The PPCW structure is normally constructed by precast wall panels across horizontal joints at the floor levels using unbonded post-tensioned prestressing (PT) tendons. Debonded mild steel reinforcement crossing the wall joints, which is referred to as energy dissipation (ED) steels in this paper, is provided to enhance the hysteretic energy dissipation of the structure. To prevent significant gap opening at the upper joints, mild steel should be designed crossing these joints to make the joints strong enough. The adoption of PT tendons allows the precast wall panels to open at the base and also minimizes the residual drift by providing a restoring force, resulting in a flag shaped hysteresis loop. A series of experimental and analytical studies conducted on the unbonded PPCWs have demonstrated the excellent seismic performance of PPCW structures [3-7].

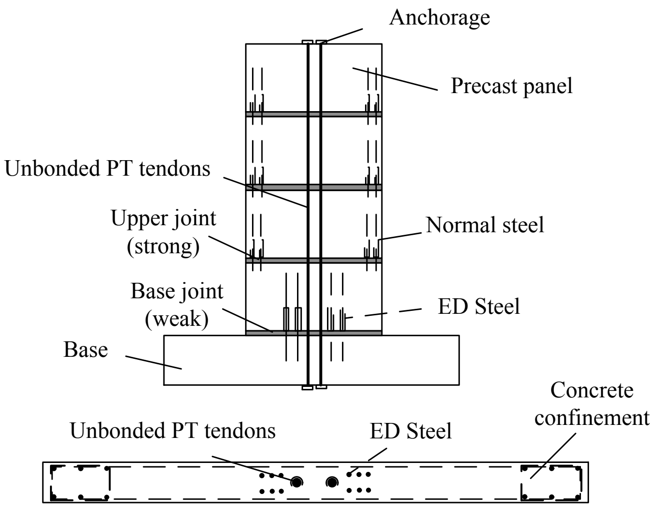


Figure 1: Schematic representation of a PPCW

Wiebe et al. [8] investigated the seismic performance of multiple rocking sections over the height of one single shear wall models. The results showed that the bending moment envelope was proved to be significantly reduced by providing multiple rocking sections, while maintaining the inter-story drifts within permissible limits. Khanmohammadi et al. [9] employed multiple rocking systems for PPCW systems and all rocking sections were equally designed by strength. The results indicated a great reduction of seismic shears and overturning moments along the vertical height with negligible residual drifts.

In order to investigate the behavior of multiple rocking systems in high-rise buildings, two alternatives of rocking joints design (only base, base & mid height) were considered. For comparison with the conventional design, for each building, corresponding cast-in-place shear wall structures designed by the current design code [10-12] were also established. Criteria including peak drifts, inter-story drift, distribution of shear force and bending moment ratio, and concrete stress strains at base rocking sections were evaluated and compared with the traditional design building to discover the effect of multiple rocking sections over height on the structural response.

2 NUMERICAL MODELING VERIFICATION

2.1 Test Description

The experimental program investigating the behavior of precast concrete walls utilizing unbonded PT tendons conducted by Perez *et al*. [4] was selected to validate the simulation method. The test setup is shown in Figure 2. Specimen TW1 were tested under monotonic loading and TW2 was tested under cyclic loading. Material properties are listed as below: compressive strength of the unconfined concrete f_c=52Mpa; compressive strength of the confined concrete f_{cc}=110Mpa; yielding stress of the PT tendons f_{py}=951MPa; initial stress of PT tendons f_{pi}=0.6f_{pu} (design ultimate stress f_{pu}=1102MPa).

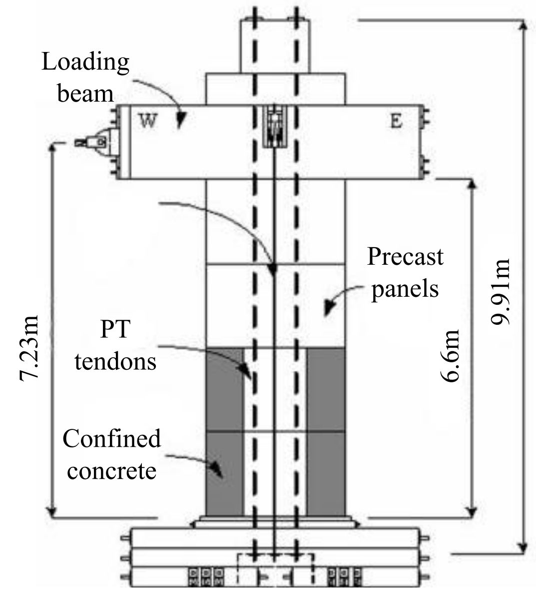


Figure 2: Test set up

2.2 Simulation Method

Figure 3 illustrates a schematic view of the methodology used for simulation of the experimental PPCW specimen in OpenSEES. It is assumed that no sliding-slip exists between the contact surfaces. The proposed simulation model included a bed of truss elements at the base of wall where the gap-opening is expected, corotational elements for the PT tendons, and fiber force-based beam-column elements for the precast wall panel. In beam-column elements, each section is discretized into unconfined concrete fibers, confined concrete fibers, and steel fibers. One force-based element per story with five integration points was used for the walls and the columns. For concrete material, Concrete02 material model is used in which the modified Kent and Park model is used in compression, an initial linear elastic branch together with a linear softening branch up to zero stress is used in tension. For reinforcing steel, Steel02 material model is used. Concrete fibers were modeled with no tensile capacity which is achieved by combined the traditional material and the Elastic-No-Tension material in series. A truss bed height of twice the length of plastic hinge is recommended (h_{spring}=2l_p). Plastic hinge length l_p=min $(2t_w^{"}, 2a^{"})$, where $t_w^{"}$ is thickness of confined concrete, and $a^{"}$ is depth of the equivalent compression stress block excluding cover thickness. The “PDelta” algorithm is used to consider the effect of large deformations.

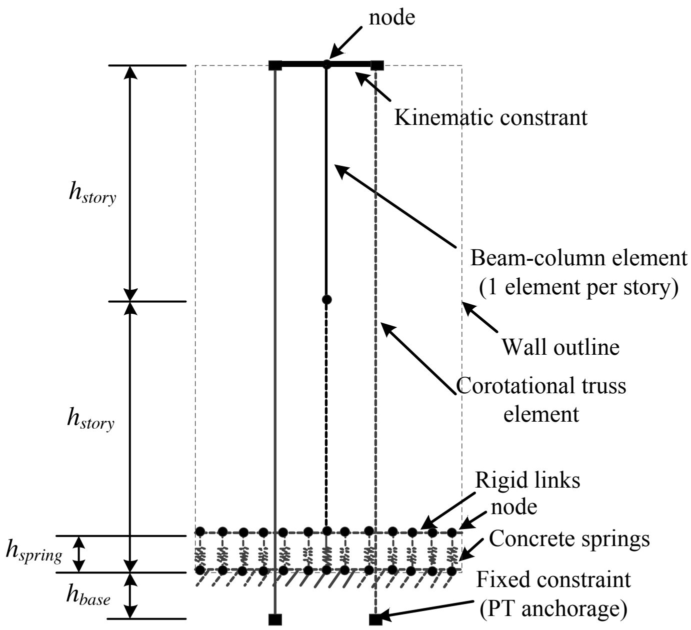


Figure 3: Modeling of PPCW

2.3 Results of Numerical Analyses

Figure 4(a) compares the experimental results of the lateral response for TW1 with the analytical results, which shows that the initial stiffness and base shear of TW1 are well captured. Figure 4(b) compares length of contact between the wall base panel and foundation. Figure 5 compares the results of the lateral response for TW2. A good agreement between analytical and experimental response was found in the results pertaining to both hysteretic curves and uplifts. The results indicated that the bed of truss elements using the uniaxial concrete stress-strain material model accurately captured the wall-to-foundation rocking interface. Analysis based on fiber section elements is accurate enough to predict both the global and local response of concrete structures although shear-flexure interaction was not considered.

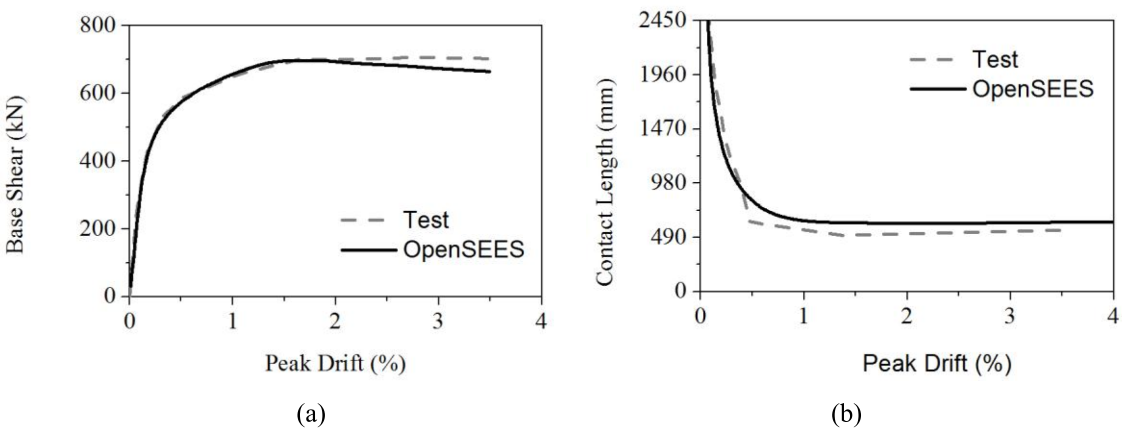
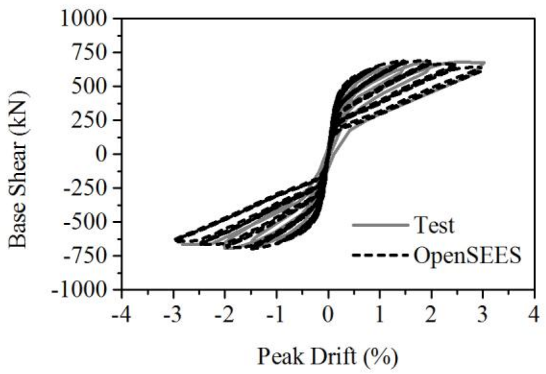
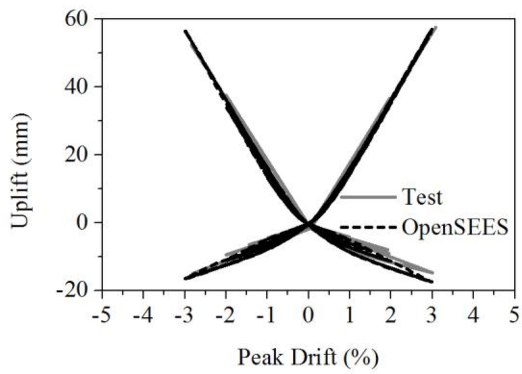


Figure 4: Comparison of the analytic model with experimental results for TW1: (a) Wall lateral response; (b) Contact Length



(a)



(b)

Figure 5: Comparison of the analytic model with experimental results for TW2: (a) Wall lateral response; (b) Uplift of wall ends

3 CASE STUDY STRUCTURE

A group of 16-story buildings of rectangular plan used for office buildings were evaluated. The buildings of the first group were designed by the conventional provisions, and those of the second group using PPCW structural system were designed following the direct displacement-based design (DDBD) [13]. The buildings were almost identical in geometry. In addition to rocking system at the base, double rocking joints (one at the base and the other at mid-height) were also considered. The 16-story PPCW buildings allowing gap-opening only at base is designated as PPCW-16-1, while allowing gap-opening at base and mid-height is designated as PPCW-16-2 (Figure 6). Similar notation is adopted for CIPW buildings.

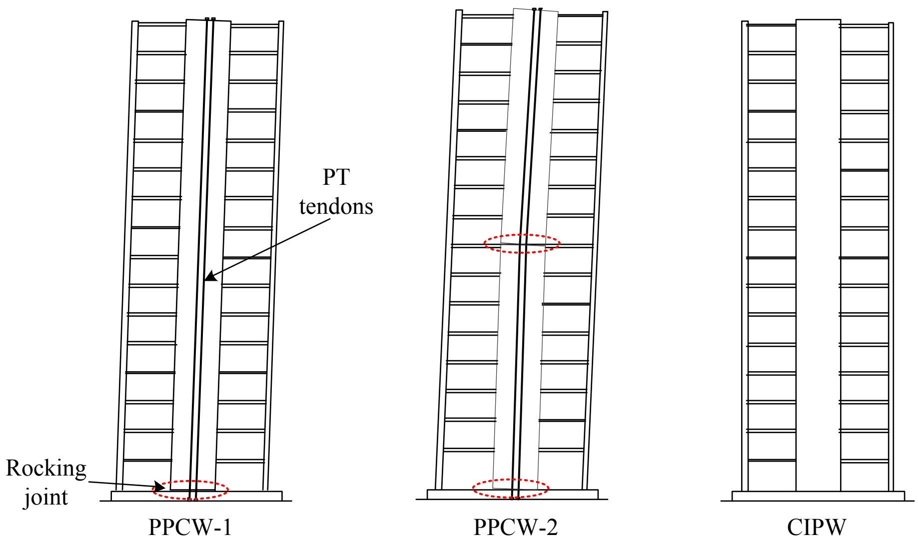


Figure 6: Schematic representation of considered opening joints

The lateral force resisting system of the buildings consists of a frame-wall system in the NS direction and a moment-resisting frame system in the EW direction. The plan of the considered buildings with a dimension of 36m×18m and a story height of 4.0m is illustrated in Figure 7. The dead load (*DL*) and live load (*LL*) is 5kPa and 2kPa, respectively [10]. Gravity load combination is assumed to be 1.00*DL*+0.50*LL*. The seismic site design is Group 2 and Site Class IV according to the related design criteria [11].

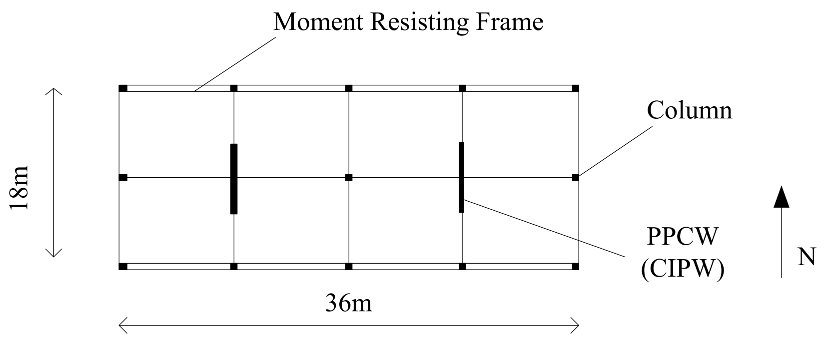


Figure 7: Building plan for the case study structure

3.1 Design of PPCW buildings

The PPCW structural structures were designed following the DDBD approach. Each wall consists of one precast concrete wall panel each story, post-tensioned to the foundation with unbonded high strength PT tendons. The beam-column connection is designed as hybrid precast ductile connections following the recommendations given by El-Sheikh et al. [14]. Column-foundation connection is similar with the wall-foundation connection. The connection between the floors and the precast wall panels is assumed to be a vertical sleeved-type connection to allow the lateral forces to be transmitted from the floor to the wall while preventing any vertical actions to be transmitted from the wall panel to floor [15]. A listing of the important features of PPCW buildings is provided in Table 1 and Table 2.

Story Number	Column (mm×mm)	ρ_{ptc} (%)	Beam(NS) (mm×mm)	ρ_{ptbNS} (%)	Beam(EW) (mm×mm)	ρ_{ptbEW} (%)
1-4	800×800	1.09	600×300	0.58	900×300	0.58
5-8	750×750	1.24				
9-12	700×700	1.43	600×300	0.66	900×300	0.58
13-16	650×650	1.66				

Note: ρ_{ptc} = area ratio of PT steel in columns; ρ_{ptb} = area ratio of PT steel in beams; Beam(NS) means beams in NS direction; Beam(EW) means beams in EW direction.

Table 1: Member sizes of PPCW buildings

l_w (mm)	t_w (mm)	ρ_{ptw} (%)	ρ_s (%)	l_{hoop} (mm)	$l_{debonded}$ (mm)
8500	500	0.053	0.52	2000	1000

Note: l_w =wall length; t_w =wall thickness; ρ_{ptw} = area ratio of PT steel in wall; ρ_s = area ratio of ED steel; l_{hoop} = confinement region length; $l_{debonded}$ = unbonded length of ED steel.

Table 2: Design details of precast walls used in PPCW buildings

3.2 Design of CIPW buildings

In addition to PPCW buildings, the third model was considered with the cast-in-place shear wall (CIPW) buildings. The CIPW buildings were almost identical to PPCW buildings in geometry. The CIPW buildings were designed by the seismic provisions in China. Detailed size information of the CIPW building is listed in Table 3 and Table 4.

Story Number	Column (mm×mm)	ρ_c (%)	Beam(NS) (mm×mm)	ρ_{bNS} (%)	Beam(EW) (mm×mm)	ρ_{bEW} (%)
1-4	800×800	1.42	800×300	0.7	900×400	0.72
5-8	750×750	1.35				
9-12	700×700	1.24	700×300	0.71	800×300	0.74
13-16	650×650	1.26				

Note: ρ_c = area ratio of longitudinal steel in columns; ρ_b = area ratio of longitudinal steel in beams.

Table 3. Member sizes of CIPW buildings

l_w (mm)	t_w (mm)	ρ' (%)	ρ'' (%)	ρ_t (%)	l_{hoop} (mm)
8500	500	1.25	0.35	0.13	2000

Note: ρ'= ratio of longitudinal boundary reinforcement area to gross section area of wall boundary; ρ''= ratio of volume of transverse confining steel to gross concrete area of web; ρ_t = volumetric ratio of transverse reinforcement; l_{hoop} = confinement region length.

Table 4. Design details of shear walls used in CIPW buildings

4 GROUND MOTIONS

ID No.	Record Seq.	Year	M_w	Event	Recording Station	R_{rup} (km)	Record Component
1	169	1979	6.5	Imperial Valley	Delta	22.03	IMPVALL_H-DLT262 IMPVALL_H-DLT352
2	741	1989	6.9	Loma Prieta	BRAN	10.72	LOMAP_BRN000 LOMAP_BRN090
3	1106	1995	6.9	Kobe, Japan	KJMA	0.96	KOBE_KJM000 KOBE_KJM090
4	1120	1995	6.9	Kobe, Japan	TAKATORI	1.47	KOBE_TAK000 KOBE_TAK090
5	1158	1999	7.5	Kocaeli, Turkey	Duzce	15.37	KOCAELI_DZC180 KOCAELI_DZC270
6	1244	1999	7.6	Chi-Chi, Taiwan	CHY101	9.94	CHICHI_CHIY101-E CHICHI_CHIY101-N
7	1605	1999	7.1	Duzce, Turkey	Duzce	6.58	DUZCE_DZC180 DUZCE_DZC270

Table 5: Details of the selected input accelerations for the nonlinear time history analysis

The seismic fortification intensity of the case study buildings site is assumed to be 8, which means the peak ground motion (PGA) under frequent, basic, and rare level of seismic hazard is 0.07g, 0.2g, and 0.4g respectively [10]. Structures are designed to be elastic under frequent level of seismic hazard. Thus except for basic and rare level earthquake (PGA=0.2g and 0.4g), severe level earthquake was considered, which has a PGA of 0.6g. A total of seven earthquake records (Table 5) were selected from the PEER Ground Motion Database. The magnitude of the selected earthquake M_w is greater than 6.5.

The mean acceleration response spectrum with 5% damping of the scaled ground motion records under rare level of seismic hazard is shown in Figure 8. The bold line is design acceleration response spectrum provided in Chinese seismic code GB50011-2010.

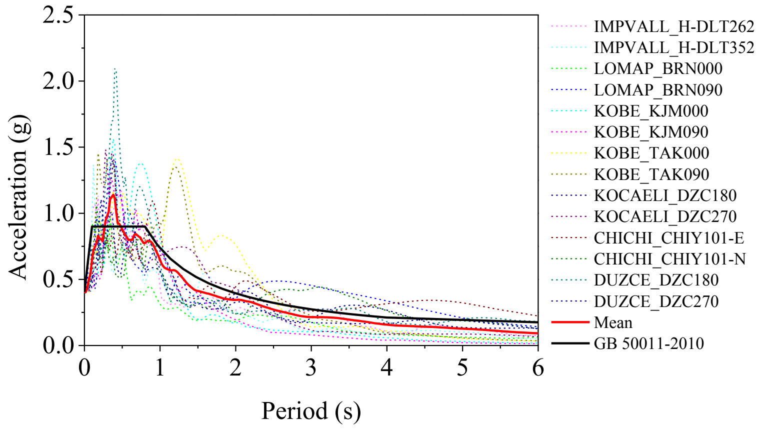


Figure 8: Code-specified design acceleration spectrum and median response spectrum S_a of selected input ground motion records

5 RESULTS OF ANALYSES

Three-dimensional analytical models for the two designs were created, and nonlinear static and dynamic time history analyses were carried out under bidirectional excitations. The ratio of PGA(NS) to PGA(EW) should be fixed as 1:0.85 according to the Chinese seismic code.

5.1 Nonlinear static analyses

A total of 50 contact springs were used to model the gap-opening interface of precast walls in the 16-story PPCW buildings. Nonlinear pushover analyses of the case study buildings were conducted adopting inverted triangular distribution of lateral forces. The response of case study buildings under monotonic loading is shown in Figure 9(a). Figure 9(b) compares hysteretic energy dissipation for PPCW and CIPW structures. Hysteresis curve of the CIPW building is fuller, which indicates the possibility of large residual deformations after an earthquake and better energy dissipation. As expected, the hysteresis curves of the PPCW buildings show significant ‘flag-shape’ characteristic. Energy dissipation of PPCW buildings is smaller and results in minor residual inter-story drifts.

5.2 Nonlinear dynamic analyses

Modal analysis was carried out first to evaluate the natural characteristics of the case study buildings. The vibration periods calculated using OpenSEES are shown in Table 6. The Rayleigh damping of the PPCW and CIPW structures were considered 3% and 5%, respectively. Base uplift due to rocking eliminates the development of concrete tensile stresses at the wall base and provides a ‘base isolation effect’ through the elongation of the system’s structural period.

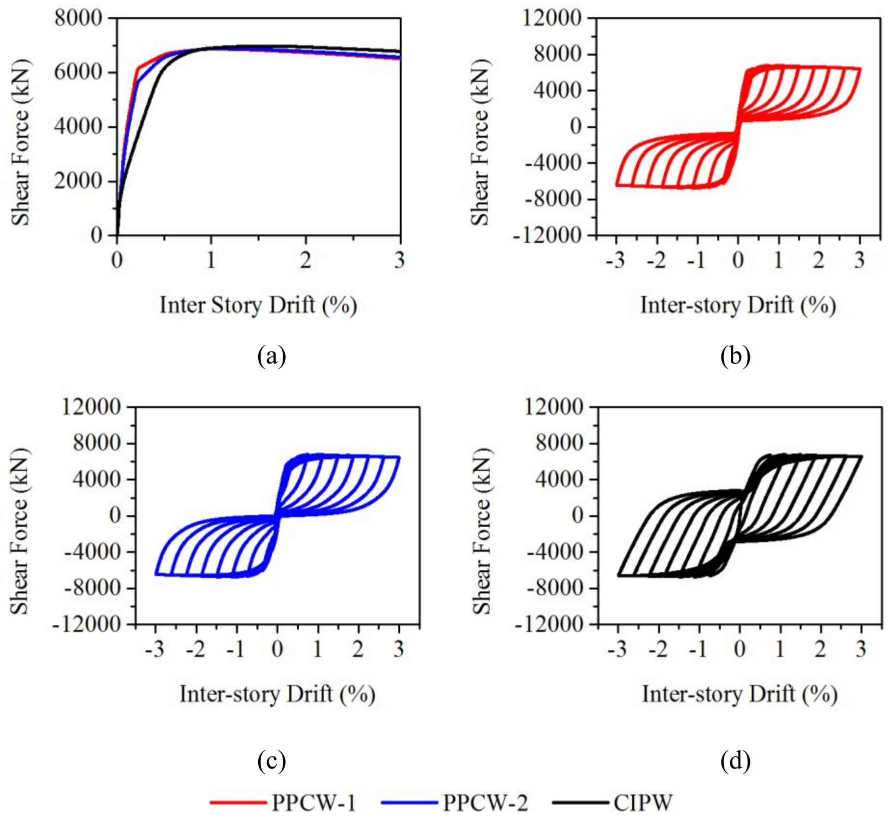


Figure 9: Static response of case study buildings: (a) monotonic loading; (b) cyclic loading of PPCW-1; (c) cyclic loading of PPCW-2; (d) cyclic loading of CIPW

	1st Mode Period (s)	2nd Mode Period (s)
PPCW-1	1.39	0.24
PPCW-2	1.38	0.24
CIPW	1.35	0.23

Table 6: Fundamental periods of PPCW and CIPW buildings in NS direction

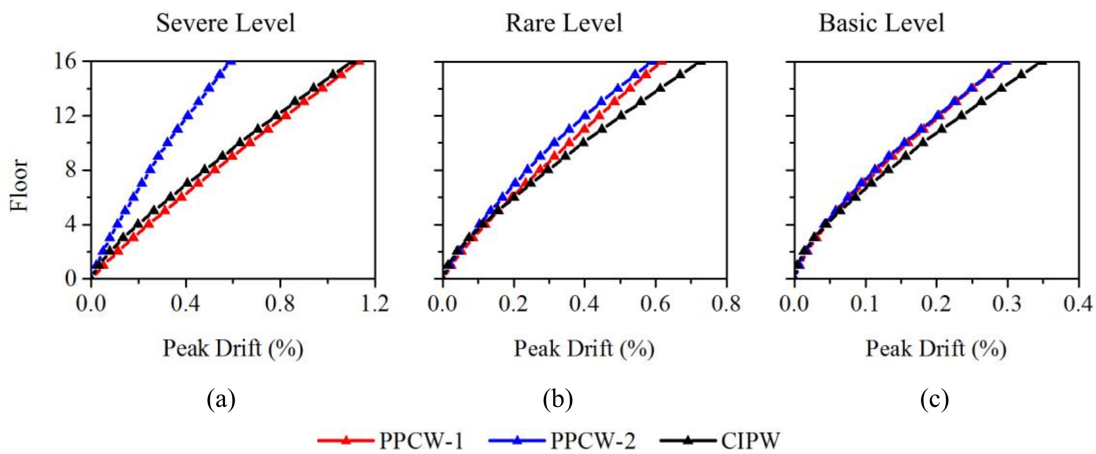


Figure 10: Mean values of inter-story drift under seismic loading

The effects of gap-opening at the different locations of the precast shear wall on the maximum peak roof drifts are investigated, as shown in Figure 10. The results indicate that because of the joint opening at mid height, the peak drift is significantly reduced by nearly 50% at severe level seismic hazard. In rare and basic level seismic hazard, the peak drift of PPCW-2 building is approximately equal to that of PPCW-2 building, but still less than CIPW buildings.

In Figure 11, results of the maximum inter-story drift are presented. Results of drifts in PPCW buildings did not exceed 1% under rare level ground motions, which is the design drift value in the conventional seismic code designs, the outcome results were acceptable from design point of view. The PPCW buildings have a more uniform distribution of inter-story drift ratio (IDR) compared with the corresponding CIPW buildings. Under the basic level ground motions, the inter-story drifts of PPCW-1 and PPCW-2 buildings are nearly the same. Development of rocking joints over height caused the drift to increase in the upper half and decrease in the lower half parts. With the increase of input amplitude of ground motions, inter-story drift of PPCW-2 buildings reduces significantly compared to PPCW-1 building.

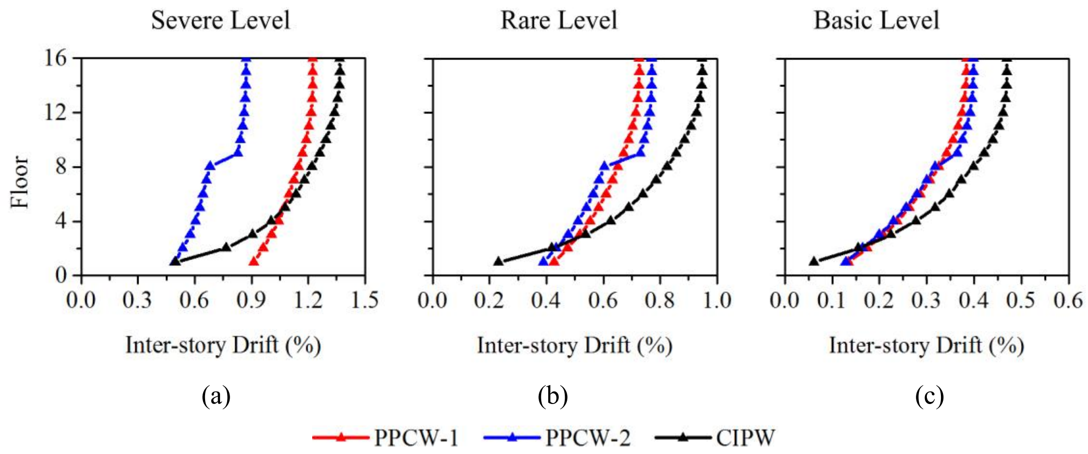


Figure 11: Mean values of inter-story drift under seismic loading

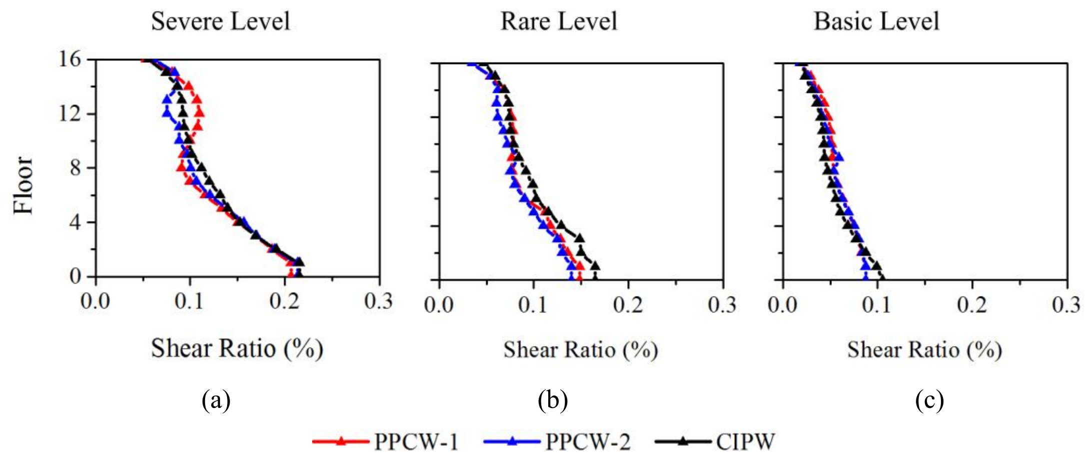


Figure 12: Mean values of shear force ratio under seismic loading

The distribution of shear forces is important in the performance-based design approaches. Distribution of shear force ratios along height under three level of seismic hazard are shown in Figure 12. To compare the results straightforward, the shear force ratios were normalized by the total weight of the building. With adding one more rocking joint at mid-height, shear forces were smaller than CIPW building, which in turn represented that the current design codes and shape of shear force distribution were more regular than those without the second increase in upper half-height. With developing rocking sections, the amounts of shear forces through upper half part of the buildings tended to be larger than PPCW-1 building.

Figure 13 shows distribution of maximum bending moment under three level ground motions. The results were normalized by the product of the building height and weight which was a moment that can be regarded as overturning moment by applying a concentrated force to the roof with the value of the total seismic weight of the structure. Values of the moment demands related to CIPW models are more than those of the PPCW model, particularly at lower parts of buildings. The maximum bending moment of PPCW-2 building at mid height was reduced by 54 and 79% compared with PPCW-1 building under severe and rare ground motions, respectively.

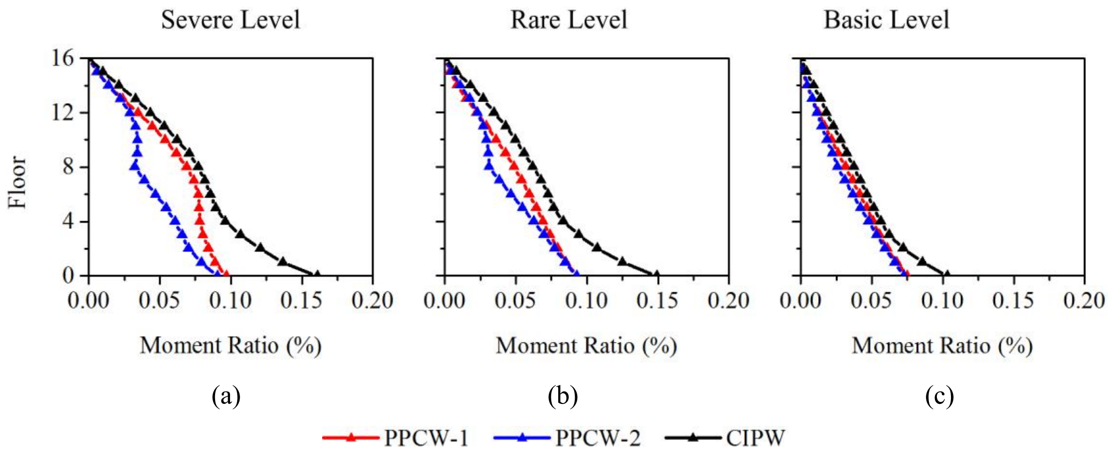


Figure 13: Mean values of moment ratio under seismic loading

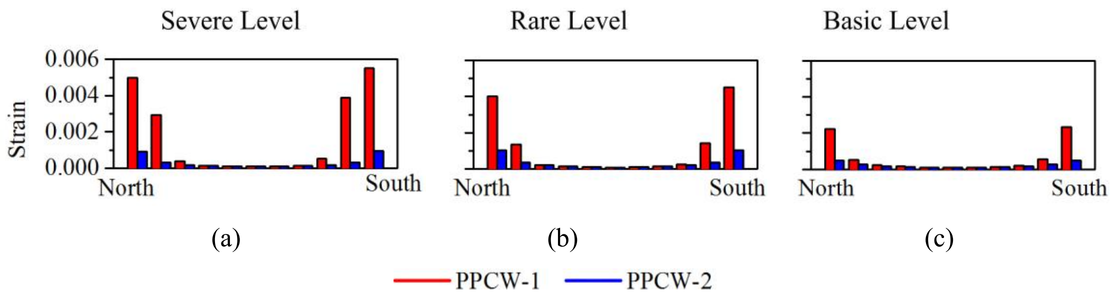


Figure 14: Concrete compressive strain on base rocking joints of PPCW buildings

The maximum suffered concrete compressive strains of the PPCW buildings are in the base-to-foundation section, which are presented in Figure 14. Unlike cast-in-place shear walls, damage to PPCWs is often featured by crushing of confined core concrete at the wall toes [3-5, 16]. As expected, the maximum concrete compressive strains are found around the wall toes, and the observed concrete compressive strains are lower than design ultimate concrete compressive strains (0.02 for confined concrete; 0.004 for unconfined concrete). The concrete strain is greatly alleviated in all cases, which means the adding joint at mid-height is a good protection of toes of concrete walls. The results showed that the concrete of the rocking sections did not reach the yielding point in any of the models under three levels of seismic hazard. No crushing of confined concrete wall toes is expected in PPCW buildings.

6 CONCLUSIONS

- The proposed fiber model based on OpenSEES of a PPCW is simple to conduct. Comparisons with the experimental results demonstrated that the simulation method can capture the nonlinear hysteretic response characteristics of PPCW reasonably well under both monotonic loading and cyclic loading. The model estimated the wall uplift and change in neutral axis accurately. The rationality of the model is verified.
- The PPCW buildings have a more uniform distribution of inters-tory drift ratio compared with the corresponding CIPW buildings and thus preventing undesirable weak story failure pattern.
- The higher mode effects on shear and moment action demands were mitigated using multiple rocking joints.
- By using multiple joint, compressive strains of the concrete at wall toes are significantly reduced and severe damage of the wall is alleviated.

7 ACKNOWLEDGMENT

This research is sponsored by Natural Science Foundation of China (NSFC) (51638012). The support is gratefully acknowledged.

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