TSINGHUA SCIENCE AND TECHNOLOGY
ISSN 1007-0214 03/15 pp640-647
Volume 11, Number 6, December 2006

Passive Control Reinforced Concrete Frame Mechanism with High Strength Reinforcements and Its Potential Benefits Against Earthquakes*

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Abstract: Severe earthquakes continue to cause major catastrophes. Many devices in active, hybrid, and semi-active structural control systems which are used as controllable force devices are costly to build and maintain. The passive control reinforced concrete frame (PCRCF) reinforced with high strength steel only in the columns presented here provides structural systems more resistance to lateral earthquake loadings at comparatively lower cost. The effectiveness is demonstrated by a nonlinear static analysis using fiber model for a single story single bay frame. The study shows that the use of high performance steel in columns prevents formation of plastic hinges at the critical column base sections and failures are always initiated by reinforcement yielding at the beam ends. Furthermore, after experiencing severe lateral drift, the passive control design has small residual displacements compared to ordinary reinforced concrete frames. PCRCF rehabilitation and strengthening can be achieved more easily as compared with ordinary reinforced concrete frame.

Key words: earthquake; passive control; high strength reinforcement; failure mechanism; residual displacement

Introduction

In most reinforced concrete (RC) structures, a large stiffness is needed in order to limit structural deformation for service load conditions. In seismic resistant structures, however, the energy dissipation demands are imposed and inelastic deformations are permitted in special detailed regions of structures when the severe earthquake attacks. In particular, moment resistant frames designed according to the strong column/weak beam concept are expected to undergo inelastic deformations by forming plastic hinges in the beams. The columns are supposed to remain elastic to maintain vertical load carrying capacity and prevent possible collapse. Although the required flexural strength difference between beams and columns at joint locations enforces this ideal frame deformation mechanism, the deformations at the base of the first story columns must be excessive to initiate the frame to sway[1]. Therefore, the formation of plastic hinges at the base of the first story columns is inevitable as shown in Fig. 1. Although in some instances, the formation of plastic hinges at the column bases may not be so critical regarding the safety of the structure, this formation requires extensive rehabilitation efforts. Moreover, the

Received: 2005-08-12; revised: 2005-09-28

* Supported by the Specialized Research Fund for the Doctoral Program of Higher Education (No. 20040003095) and the Cultivation Fund of the Key Grant Scientific and Technical Innovation Project, Ministry of Education of China (No. 704003)

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frame does not possess the recentering ability after undergoing severe lateral drift during strong shaking, and the chances of complete demolition of the structure are always there in case of excessive yielding at the column base sections. Furthermore, the possibility of exceeding the moment capacity at the top of the first story columns still exists, and the sway failure mechanism can be formed as shown in Fig. 2.

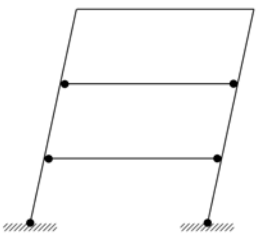


Fig. 1 Strong column/weak beam configuration

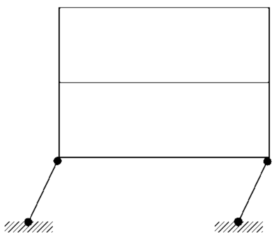


Fig. 2 Soft first story failure

This paper is to describe the alleviation and prevention of the formation of plastic hinges in frame columns by introducing high strength steel reinforcement in RC frame columns, which is called here after as passive control RC frame (PCRCF).

1 Mechanism of PCRCF

A conventional designed moment resistant frame usually cannot successfully develop its ability against unexpected earthquake loadings due to limited flexural strength and the formation of plastic hinges at the base of the first story columns. Excessive yielding at the column base sections may lead to eventual collapse, and the soft first story failure mechanism is difficult to avoid. Moreover, even after the survival of structure against extreme lateral drift, the large residual deformations may suggest the need for complete demolition. By introducing high strength reinforcement in columns, PCRCF can safeguard its column base section from excessive yielding and can resultantly adjust structural characteristics by using the reserve flexural strength at the column base sections. Furthermore, the yielding will only occur at beams ends. Due to elasticity of high strength reinforcement in columns, recentering capacity can be improved with the reduced residual lateral displacement under extreme lateral loading. Therefore, repairs can be made easier.

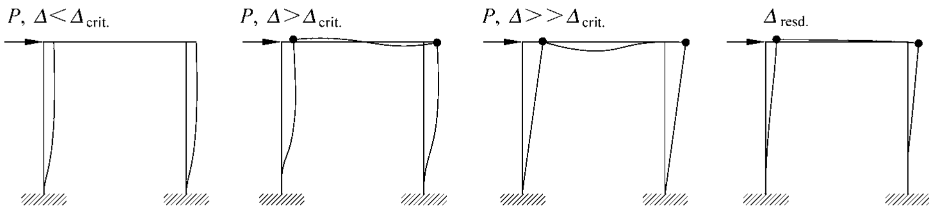
Seismic behavior of the structures has been the subject of extensive study over the last several decades. To date, the basic philosophy behind seismic resistant structures is that a structure should not collapse during severe earthquake, although it may undergo structural as well as non-structural damage. However, reduced residual displacement and minimum rehabilitation after seismic events are aims of research in recent years.

So far, a large number of passive control systems have been developed and installed. Furthermore, structural systems designed with self-centering capabilities after experiencing large nonlinear deformations commonly use un-bonded post tensioned steel tendons in various types of construction, such as in pre-cast concrete by Priestley et al.[2], El-Sheikh et al.[3], Kurama et al.[4]; in steel structures by Ricles et al.[5]; in partially prestressed concrete for bridge piers by Zatar and Mutsuyoshi[6]; and in unbonded post tensioned bridge piers by Kwan and Billington[7].

Concrete ductility with fiber reinforced polymer (FRP) tendons has been studied by Naaman and Jeong[8], Alsayed and Alhozaimy[9], while as hybrid FRP reinforcement with inherent ductility by Harris et al.[10] However, with the development in the engineered cementitious composites (ECC), a frame system with intrinsic collapse prevention capabilities has also been proposed by Fischer and Li[11] by utilizing ECC and FRP reinforcement in columns.

The ideal frame deformation sequence is shown in Fig. 3. Besides reduced residual displacements, absence of potential collapse mechanism by avoiding yielding at the column base sections is observed. However, being a new innovative material, ECC has been hence scarcely introduced to construction industry. It is still needed to explore cheaper materials and to investigate the conventional materials to get this ideal proposed mechanism.

In the present study, this ideal frame mechanism by using ordinary conventional concrete with high strength steel reinforcement is investigated.



P, lateral load; Δ, lateral deflection; Δ_{crit}, critcal lateral deflection; Δ_{resd}, residual lateral deflection

Fig. 3 Deformation sequence of passive control RC frame

2 Analysis Models and Method

To demonstrate the PCRCF mechanism and to investigate the behavior difference between the ordinary frame and the PCRCF, two single-story single-bay frames, ordinary frame (ODF), and PCRCF, were analyzed. The behaviors and failure mechanism of both the frames were estimated with nonlinear static analysis. Figure 4 presents the selected geometry and the loading pattern for both the frames.

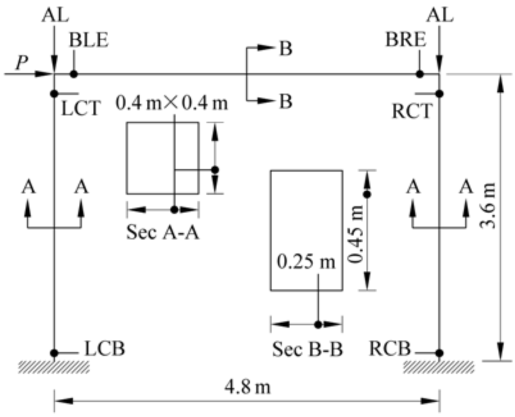


Fig. 4 Selected frame geometry

To describe the behaviors of both frames, the critical sections such as left column base (LCB), left column top (LCT), beam left end (BLE), beam right end (BRE), right column top (RCT), and the right column bottom (RCB) are marked. The lateral point load (P) applied at the top left end of frame and the dead axial load (AL) equal to 10% of the gross capacity of columns was applied on columns. Beam was loaded with a uniformly distributed load (UDL) of 18 kN/m. Material self weight of the frame was also considered in the analysis. The frame was designed according to the ACI code specifications and the details of the selected strengths and steel area ratios are given in Tables 1 and 2.

Both the frames were analyzed on MSC.Marc using beam element 52 with the hypoelastic material option. A fiber model programmed with user subroutine UBEAM was used to simulate the section behavior in the analysis. Three different section discretization schemes with 100, 64, and 36 concrete fibers while keeping the 4 steel fibers in each case were investigated to confirm the convergence requirement with a relative force tolerance of 0.1. The cover and the core concrete fiber areas were different; however, the 25-mm clear cover was selected for all the sections. The scheme with 36 concrete and 4 steel fibers at each corner, as shown in Fig. 5, meets the convergence requirement.

Table 1 Material strength properties used in the analysis

Frame analyzed	Steel yield strength (MPa)		Concrete compressive strength (MPa)	
	Columns	Beam	Columns	Beam
ODF	400	400	40	30
PCRCF	1860	400	40	30

Table 2 Selected steel area ratios in frames

Frame analyzed	Steel ratio in columns $A_s/(b \times h)$	Tensile steel ratio in beam $A_s/(b \times d)$
ODF	0.02	0.02
PCRCF	0.02	0.02

A_s: steel area; b: width of section; h: depth of section; d: effective depth of section

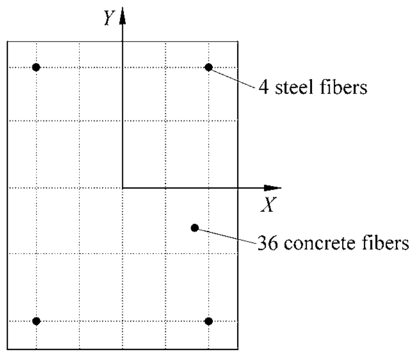


Fig. 5 Section discretization

Finite element length was kept equal to 300 mm for the frame elements in both frames. Furthermore, at critical sections where there were more chances of distress, it was reduced to 100 mm. The finite element model used for analysis is also shown in Fig. 6. The selected Beam element 52 in MSC.Marc has three integration points along the length selected so the results are further at the three sections.

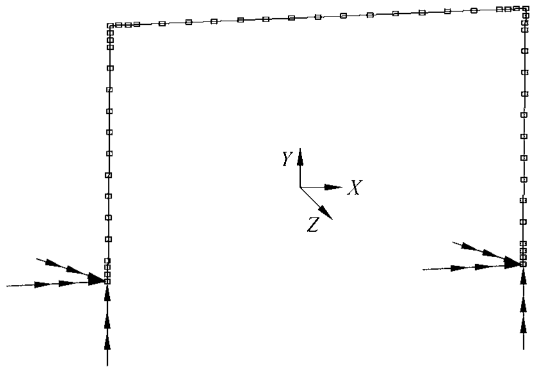


Fig. 6 Finite element model used for analysis (from MSC.Marc)

Uniaxial stress-strain relations are given for concrete and steel fibers as shown in Figs. 7 and 8. The concrete stress-strain relation with peak strength strain ε_0=0.002 and ultimate strength strain ε_u=0.004 was selected for concrete fibers. The elastoplastic model was used to describe the stress-strain relation of steel. Because of the smaller contribution to ultimate strength, the concrete tensile strength was considered zero during analysis. The ordinary and high performance steel yield strains (ε_y) were selected as 0.002 and 0.009. The lateral load P for both the frames was selected as the static load increased gradually until the failure state was attained.

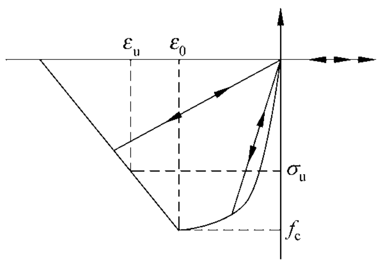


Fig. 7 Stress-strain relation for concrete fibers

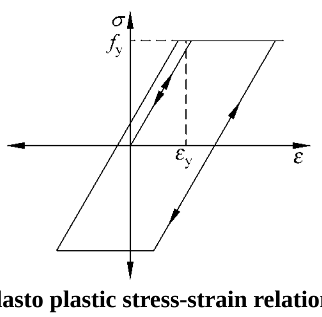


Fig. 8 Elasto plastic stress-strain relation for steel

3 Analysis of Results

3.1 Response stages

For comparative study, both the ODF and PCRCF were analyzed and the results at each of the response stages were described. The lateral load and displacement relations are shown in Fig. 9 for the ODF and PCRCF. The lateral load displacement relation of both the frames can be divided into four response stages. The end of each response stage is marked as A, B, C, and D. The four response stages are described as follows.

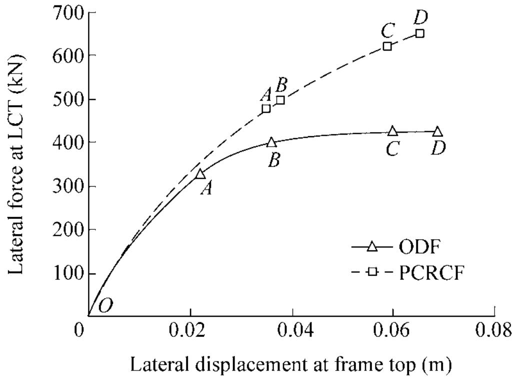


Fig. 9 Response stages comparison between ODF and PCRCF

The response Stage 1 (O-A) for both the ODF and PCRCF ranged from the start of lateral load application till the initiation of the steel yielding in beam or column critical sections in the frame. The critical sections shown in Fig. 4 always dominated in frame failure initiation. The response Stage 2 (A-B) ranged from the steel yield initiation till the concrete design compressive strain ε_0=0.002 in beam or column critical sections in the frame. The response Stage 3 (B-C) ended when the concrete reached its maximum usable strain which was selected as 0.0035. The response Stage 4 (C-D) which was also the termination of the analysis was selected when the concrete reached its ultimate compressive strain $\varepsilon_u = 0.004$.

The selection of these four response stages was based on the performance, rehabilitation, and strengthening demands imposed on the frames. The main goal was to get a better comparison of the performance between both the frames in terms of load bearing capacity, the failure initiation locations, repair, or rehabilitation demands in both frames at the end of each response stage. Moreover, in order to check the residual deformation, the unloading was performed at each response stage end.

3.2 Loading performance

The results for the lateral load versus displacement for the ODF and PCRCF at the end of each response stage are given in Table 3. Concrete and steel fiber strains at the end of each response stage at critical sections are summarized in Table 4.

Table 3 Loading performance at the end of each response stage for ODF and PCRCF

Response stage	Lateral load (kN)		Lateral displacement (mm)	
	PCRCF	ODF	PCRCF	ODF
1	481	331	35.0	22.0
2	494	400	38.0	36.0
3	624	425	59.0	60.0
4	650	426	65.0	68.0

At the end of Stage 1, marked as *A* in Fig. 9, the PCRCF has resisted more lateral load of almost 150 kN more than the ODF. Moreover, yielding occurred at the BRE section for PCRCF as compared to the more critical and vital LCB and RCB sections for ODF. This behavior difference at the end of Stage 1 shows better performance of PCRCF. It is easier and cheaper to strengthen the cracked or yielded beams as compared to more vital column base sections at restricted locations.

The end of Stage 2, marked as *B* in Fig. 9 for both the frames, indicates that PCRCF still has resisted more lateral load before reaching the concrete design compressive strain in the frame. The analysis shows that concrete design compressive strain approached at RCB sections for both the ODF and PCRCF at different lateral load and lateral displacements. The

Table 4 Condition of each of the controlling sections at the end of each response stage for ODF and PCRCF

(a) Response stage 1

Sections	Compressive strain of concrete		Tensile strain of reinforcement	
	PCRCF	ODF	PCRCF	ODF
RCB	< 0.0020	< 0.0020	$<\varepsilon_y$	0.0020
LCB	< 0.0020	< 0.0020	$<\varepsilon_y$	$<\varepsilon_y$
RCT	< 0.0020	< 0.0020	$<\varepsilon_y$	$<\varepsilon_y$
LCT	< 0.0020	< 0.0020	$<\varepsilon_y$	$<\varepsilon_y$
BLE	< 0.0020	< 0.0020	$<\varepsilon_y$	$<\varepsilon_y$
BRE	< 0.0020	< 0.0020	0.0020	$<\varepsilon_y$

(b) Response stage 2

RCB	0.0020	0.0020	$<\varepsilon_y$	0.0068
LCB	< 0.0020	< 0.0020	$<\varepsilon_y$	0.0064
RCT	< 0.0020	< 0.0020	$<\varepsilon_y$	0.0021
LCT	< 0.0020	< 0.0020	$<\varepsilon_y$	$<\varepsilon_y$
BLE	< 0.0020	< 0.0020	$<\varepsilon_y$	$<\varepsilon_y$
BRE	< 0.0020	< 0.0020	0.0033	0.0022

(c) Response stage 3

RCB	0.0032	0.0035	$<\varepsilon_y$	0.0143
LCB	0.0025	0.0031	$<\varepsilon_y$	0.0141
RCT	< 0.0020	< 0.0020	$<\varepsilon_y$	0.0058
LCT	< 0.0020	< 0.0020	$<\varepsilon_y$	0.0065
BLE	0.0024	< 0.0020	0.0099	0.0020
BRE	0.0035	< 0.0020	0.0160	0.0066

(d) Response stage 4

RCB	0.0035	0.0040	$<\varepsilon_y$	0.0171
LCB	0.0027	0.0036	$<\varepsilon_y$	0.0171
RCT	< 0.0020	0.0021	$<\varepsilon_y$	0.0072
LCT	< 0.0020	0.0022	$<\varepsilon_y$	0.0090
BLE	0.0026	< 0.0020	0.0122	0.0021
BRE	0.0040	0.0020	0.0177	0.0082

difference between the lateral load resistances at the end of this stage between both frames is almost 94 kN. PCRCF still performs better in the resistance capacity at the end of this response stage. Although at the end of this response stage, the concrete design compressive strain in both frames occurred at the same RCB sections, the response still had one major difference between both frames. As in case of ODF, the concrete reached its design compressive strain at the same RCB section where yielding of steel already had occurred at the end of the response Stage 1. But in case of PCRCF, the RCB section had not shown any sign of yielding and only concrete reached its design compressive strain which could also be controlled by providing confinement at the column base sections. The provision of confinement in case of ODF would yield little benefit as the yielding of the reinforcement at column base section would demand extensive rehabilitation efforts.

The end of Stage 3, marked as *C* in Fig. 9, further revealed the more lateral load capacity of the PCRCF. The difference between load resistances at this stage was seen almost equal to 199 kN between both frames. The end of this stage approached when concrete reached its maximum usable strain of 0.0035 in beam or column critical sections in the frame. At the end of this stage, the ODF reached the usable concrete strain 0.0035 at the column base sections (RCB and LCB) while PCRCF reached this selected strain at BRE and the RCB sections almost simultaneously. As mentioned above, the absence of steel yielding at the column base sections can offer advantage in the presence of confinement at the column base sections in case of the PCRCF. Compared with the PCRCF, the ODF at lower lateral load reached at this stage and the rehabilitation and strengthening demands at restricted column base sections are more desired because of reinforcement yielding.

At the end of failure Stage 4, marked as *D* in Fig. 9, the dominance of the PCRCF and the lateral load resistance difference between the frames were noticed almost equal to 224 kN. In case of the PCRCF, the vital column base sections LCB and RCB were still safe from yielding of reinforcement, while severe yielding along with concrete failure in case of the ODF was seen. Hence, at the failure stage, rehabilitation and strengthening would be much easier and comparatively cheaper in case of the PCRCF as compared with the ODF. Severe yielding of the column base section might suggest the complete demolition of the frame rather than strengthening and rehabilitation, which might be required in case of the PCRCF.

3.3 Failure mechanism

From the observed fiber strains which are summarized in Table 4, the extent of yielding at the four response stages at the critical sections can easily be studied. Moreover, the strains also provide guidance in the exact determination of the failure mechanism in each of the frame studied. From the available data in Table 4, Fig. 10 shows the location of plastic hinges in the frames at the 4 response stages studied.

Figure 10 and values of the fiber strains given in Table 4 show that at the response Stage 2, the failure mechanism developed in the ODF; however, PCRCF yielded only at BRE. Further even up to Stage 4, potential failure mechanism did not appear in the PCRCF. After the first significant yield at RCB, the ODF has shown displacement ductility of smaller magnitude as compared to the PCRCF. The lateral displacement values given in Table 3 show that the ODF at the end of response Stage 1 has 22.0 mm lateral displacement and at response Stage 2 ended was 36.0 mm, where failure mechanism developed in the ODF. However, the PCRCF laterally displaced to 35.0 mm at the end of response Stage 1 and until the end of the

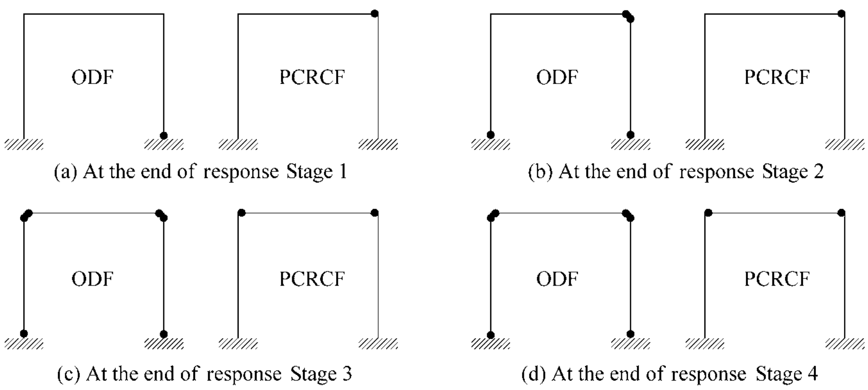


Fig. 10 Location of the plastic hinges in ODF and PCRCF

response stages, it laterally displaced to 60.0 mm with beam yielding at BLE and BRE sections. Moreover, the ODF almost yielded at all the critical sections significantly at the end of the response Stage 3. However, in comparison, the PCRCF only reached its tensile yield strain at the BLE and the BRE sections, against more lateral loads as compared to the ODF.

3.4 Unloading performance

In order to monitor the residual deformation in both the frames after unloading, separate loading and unloading cycles were carried out for both frames. The frames were laterally loaded gradually at their respective lateral loads observed at each response stage end and then gradual unloading was carried out.

The unloading was schemed inside MSC.Marc by allowing the gradual removal of the lateral load at the same rate used for application. The loading-unloading curves at each response stage are shown in Fig. 11. For both the frames, the residual displacement at the end of loading and unloading cycles is also given in Table 5.

The residual displacements given in Table 5 and Fig. 11 show that at the first cycle of loading and unloading, the residual deformation is not very large in both frames, because the ODF was unloaded when yielding just started and PCRCF was still in its elastic range. However, in the oncoming loading and unloading cycles, the difference between residual deformations gradually increased and at the response Stages 3 and 4, the PCRCF showed considerably smaller residual deformations as compared to the ODF.

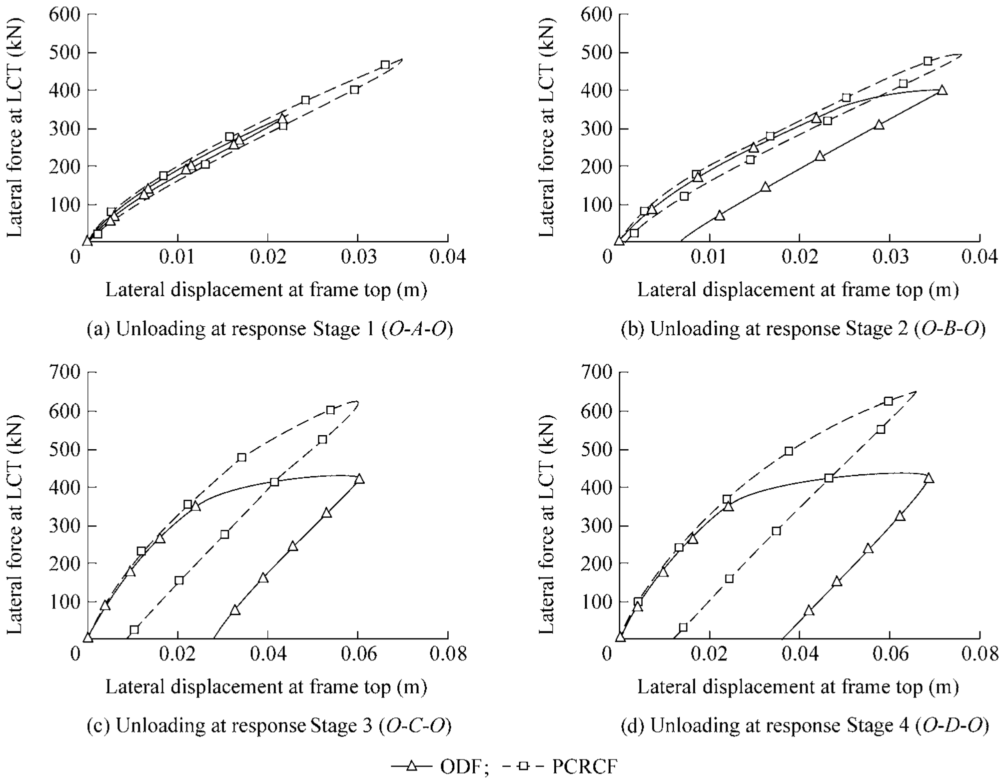


Fig. 11 Unloading performance by ODF and PCRCF

Table 5 Residual displacements at unloading in the ODF and the PCRCF

Loading-unloading cycle	Lateral load before unloading (kN)		Residual deformation (mm)	
	PCRCF	ODF	PCRCF	ODF
(*O-A-O*)	481	331	0.5	0.3
(*O-B-O*)	494	400	1.0	7.0
(*O-C-O*)	624	425	9.0	28.0
(*O-D-O*)	650	426	12.0	36.0

4 Conclusions

The passive control RC frame with high strength reinforcement and its expected benefits against earthquakes has been compared with ordinary RC frames. Two single-bay single-story frames were selected and compared. The following conclusions can be drawn.

(1) The PCRCF prevents the soft story failure and provides more lateral load resistance capacity with less reparable cost by simple replacement of ordinary conventional steel in the frame columns with high tensile strength steel.

(2) The PCRCF shows signs of distress mainly at the beam end sections which are potentially safe from the stability point of view of the entire frame as compared with the ODF where column base sections are badly yielded.

(3) Compared to the ODF, the PCRCF rehabilitation and strengthening is easier because the repairs focus on beam end sections instead of the more restricted column base sections.

(4) PCRCF reduces the residual displacement in the frames after the large lateral displacement.

(5) The PCRCF mechanism reduces the chances of complete demolition by avoiding excessive yielding at column base sections.

The performance of PCRCF can be further improved by providing concrete confinement at the beam ends and column base sections, since confinements at the beam and column ends, as well as high strength steel reinforced columns, increase the ultimate deformation capacity at the plastic hinges, and raise the deformation capacity of the whole frame.

Since the demonstration of the PCRCF mechanism has been performed by using the single-story single-bay frame, the PCRCF response needs to be demonstrated for multi-story frames with dynamic loadings in future studies. It may be helpful to mix some proportion of the high performance steel with ordinary one to achieve the response benefits. Hence, the optimum use of high performance steel in multi-story frame columns also needs to be investigated.

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