APPLICATION OF YIELD DEVICES OF LOW INVASIVITY FOR SEISMIC UPGRADING OF PRECAST PORTAL RC FRAMES

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ABSTRACT:

This article presents a new upgrading technique to enhance the seismic response of portal RC frames made of precast RC members (PPRC). Low cost yield devices of C shape (C-devices) are incorporated in a non-invasive fashion around the beam-to-column joint regions in order to introduce additional sources or hysteretic energy dissipation which lead to significant reductions of lateral and residual displacements with reduced increments of base shear. A criterion for optimum calibration of device strength is defined, and an example of the application of the proposed upgraded technique is presented in terms of the study of an existing PPRC frame representative of an industrial building in its original and upgraded states. Nonlinear inelastic analyses (push over and seismic time-history) are conducted to assess seismic response. The beam-column connections of the PPRC frame under study are modelled as pinned connections, whereas the C-devices are modelled by equivalent inelastic springs. RC frame elements are modeled using fibre elements. Results indicate that the suggested upgrading technique offers an effective economic alternative to reduce damage and/or to avoid the seismic collapse of PPRC framed buildings.

KEYWORDS: seismic response of precast portal RC frames, C-device, device calibration, equivalent nonlinear spring

1. INTRODUCTION

A number of portal RC structures made of precast RC members (PPRC frames) have shown poor performance under strong earthquake ground motion. In particular, PPRC framed buildings with simple connections where beams sit on top of columns have experienced total collapse due to loss of seat. 80% of industrial buildings in Italy and South Europe reported by Palermo *et al.* (2007) are built with PPRC frames and are characterised by one storey hinged frames. Single storey PC industrial buildings are also considered the major portion (66%) of the overall population of industrial buildings constructed in Denizli, Turkey (Senel and Palanci, 2013). These buildings are typically not designed assuming the monolithic frame arrangement that is typical of cast in-situ concrete structures. Beam-to-column connections are designed for transferring shear only (*i.e.* designed as hinges) and not providing supplemental dissipation energy (Palermo *et al.*, 2007; Belleri *et al.*, 2015). The dissipative zones of the structural system are located at the bottom of columns, near of the foundation, where a plastic hinge is expected to develop when an a strong earthquake occurs.

Local retrofitting techniques that make use of hysteretic energy dissipation devices (HEDDs) to improve the seismic performance of the above frames have been proposed. Some have been applied into existing structures. A study of Martinelli and Mulas (2010) applied a retrofitting technique which involved the use of frictional HEDDs of low invasivity (Martinez-Rueda, 2002) to improve the seismic performance of PPRC frames. The technique was first developed for existing RC structures (Martinez-Rueda, 1992; Martinez-Rueda, 1996; Martinez-Rueda, 1997) and later recommended for PC structures (Martinez-Rueda, 1998a). More recently, Soydan *et al.* (2017) proposed a low invasivity technique for PC framed structures based on the use of a new application of lead extrusion damper

(LED). The device was originally developed by Robinson and Greenbank (1975) for the retrofitting of RC bridges (*e.g.* Skinner *et al.*, 1993). The effectiveness of this damper has also been experimentally investigated using a half scale PC framed structure with and without devices (Soydan *et al.*, 2017). The results for shake table tests indicated that the relative displacement between the PC members at the PC connection decreased by 50-60% after the insertion of LEDs. The new application of LEDs proved to be effective in enhancing the seismic behaviour of the PPRC frame with devices but the fabrication of such a device appears to be rather sophisticated and hence more expensive. This would limit the applicability of this type of device in PC framed structures when compared with available HEDDs with simpler designs. A feasibility study of a new retrofitting technique using yielding HEDDs of low structural invasivity is presented in this research. The aim of the technique is to provide additional hysteretic damping to PC framed structures. The new technique presented here makes use of the steel yield devices of C shape (C-devices) previously used by Martinez-Rueda (2002, 2004) for the development of dissipative bracing of low invasivity.

2. C-DEVICES FOR PPRC STRUCTURES

2.1. Definition of the structural model

A typical PPRC frame selected to study the effectiveness of the seismic performance of the C-device is given as an example and shown in Figure 1. This frame has been previously studied by Martinelli and Mulas (2010) and by Al-Mamoori (2019). However, in this study C-devices are placed around the beam-column connections as shown in Figure 2. Further details of the 2D frames and the FE mesh of nonlinear fibre elements used to model the beams and columns is given elsewhere (Al-Mamoori, 2019). To model the C-devices shown in Figures 1 and 2, the approach proposed in Al-Mamoori and Martinez-Rueda (2019) was adopted here to model the C-devices as an equivalent nonlinear spring.

Finally, the nonlinear staged construction analysis procedure provided by CSI (2015) was used as a part of a sequence of direct integration time-history load cases. The staged construction was only used for time history analysis of the adopted structure in its upgraded state, so that the C-devices are activated primarily by the seismic actions.

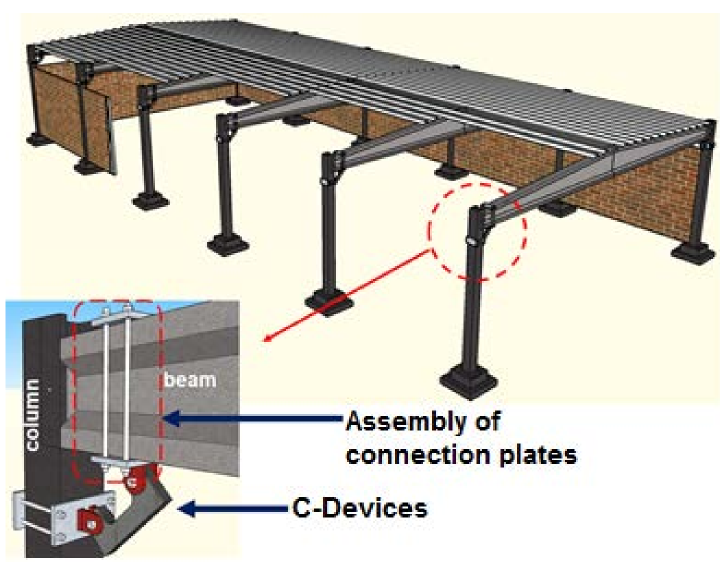


Figure 1. Precast structure retrofitted with yield C-devices incorporated in a low invasivity fashion.

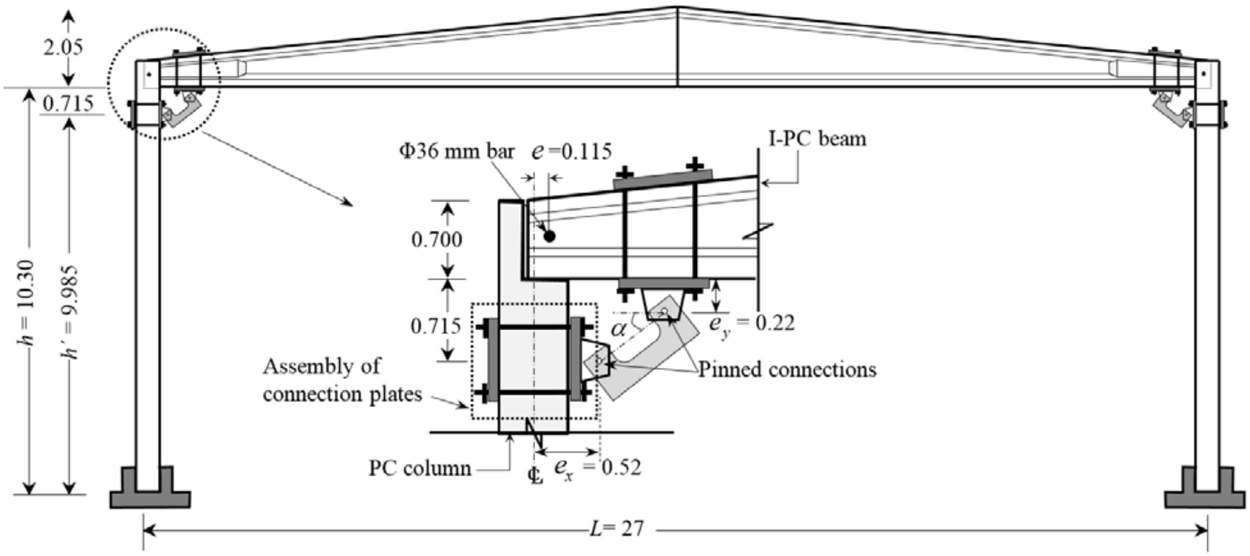


Figure 2. Geometric dimensions of a typical PC industrial frame in its upgraded state (dimensions in m) (frame adapted from Martinelli and Mulas, 2010 in Al-Mamoori, 2019)

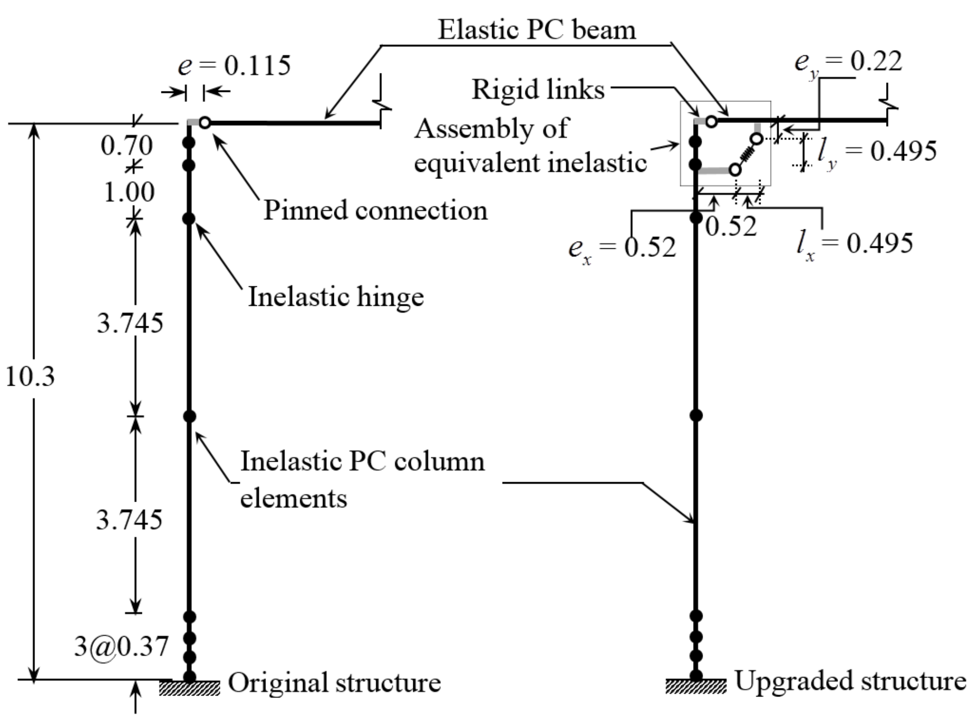


Figure 3. FE meshes for original and upgraded frames (dimensions in m)

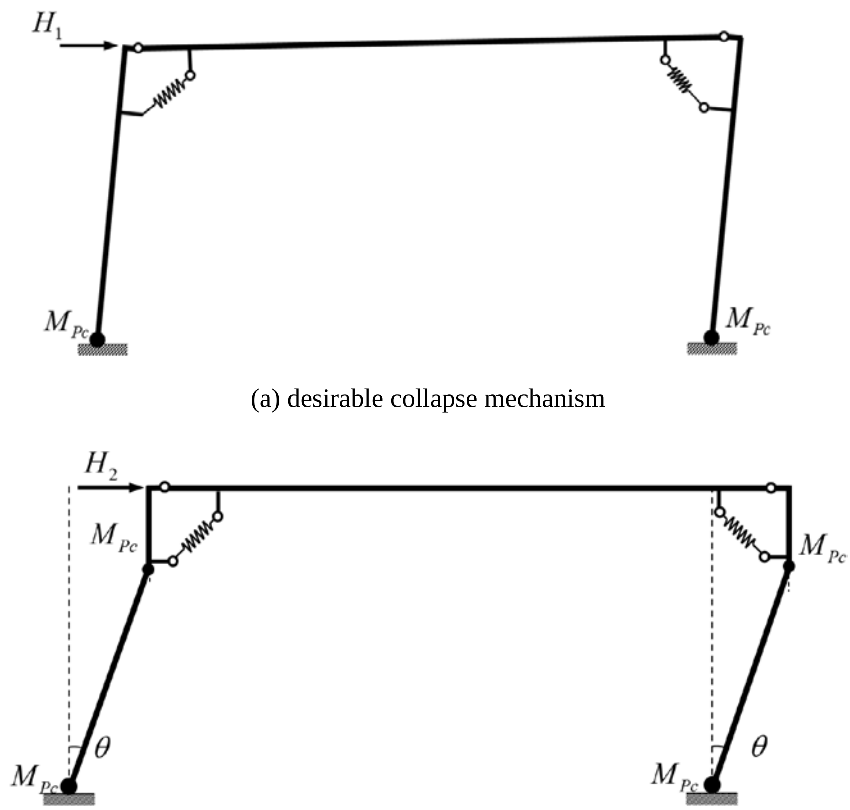
2.2. Identification of the Limit of Efficient Device Strength (LEDS)

To avoid the undesirable failure mode shown in Figure 4(b), the collapse load H_1 of the upgraded frame shown in Figure 4a should be smaller than that of the undesirable failure mode (H_2). Using plastic analysis techniques it can be shown that the maximum device strength F_d is given as

$$F_d \leq \left[\frac{2}{h'} - \frac{1}{h}\right] \frac{hM_{Pc}}{(\cos\alpha)\left[l_y + e_y\right] + (\sin\alpha)\left(e_x + \frac{2ee_x + 2el_x - 2e^2}{(L-2e)}\right)} \qquad (1)$$

Using the notations of Figures 2 and 3: h and h´ are column length and a distance between lower ends of the device to column base; L is the bay length. l_x and l_y are the horizontal and vertical lengths of the device, α is the angle of inclination of the device with respect to the horizontal. M_{pc} is the flexural plastic strength of the column (with no axial load). e_y and e_x are the vertical and horizontal eccentricities of the assembly of the devices, respectively.

A full set of design equations to define the LEDS for all possible types of undesirable collapse mechanisms of the upgraded frame is given elsewhere (Al-Mamoori, 2019). It is important to highlight that the flexural strength of the columns was associated with pure bending. The value of M_{pc} for the columns of the frame under study is 809 kNm. Using the calibration equation (1), the device strength F_d (referred to as the LEDS) that ensures the formation of the desirable collapse mechanism (see Figure 4a) was 1291 kN.



(a) desirable collapse mechanism

(b) undesirable collapse mechanism

Figure 4. Desirable and undesirable collapse mechanisms of the upgraded frame

2.3. Behaviour under reversed cyclic loading

Figures 5a and b compare the cyclic response of the original and the upgraded frames with device strengths equal to $0.50F_d$. It can be seen that the cyclic loops of the upgraded frame enclose a larger area and show reduced pinching. Therefore, a higher hysteretic damping is present in the upgraded structure. Figure 6a shows a more detailed comparison between the original and upgraded frame in terms of the evolution of the hysteretic energy. As expected, the hysteretic energy dissipated by the upgraded frame was around 3.98 times the hysteretic energy dissipated by the frame with no devices. Figure 6b shows the evolution of secant stiffness in terms of the *stiffness ratio*. This is defined as the secant stiffness at a given peak displacement divided by the secant stiffness of the original structure assessed at the first peak displacement. The stiffness of the upgraded frame is about 2.3 times that of the original structure, and this difference is reduced for increasing displacements as shown in Figure 6b.

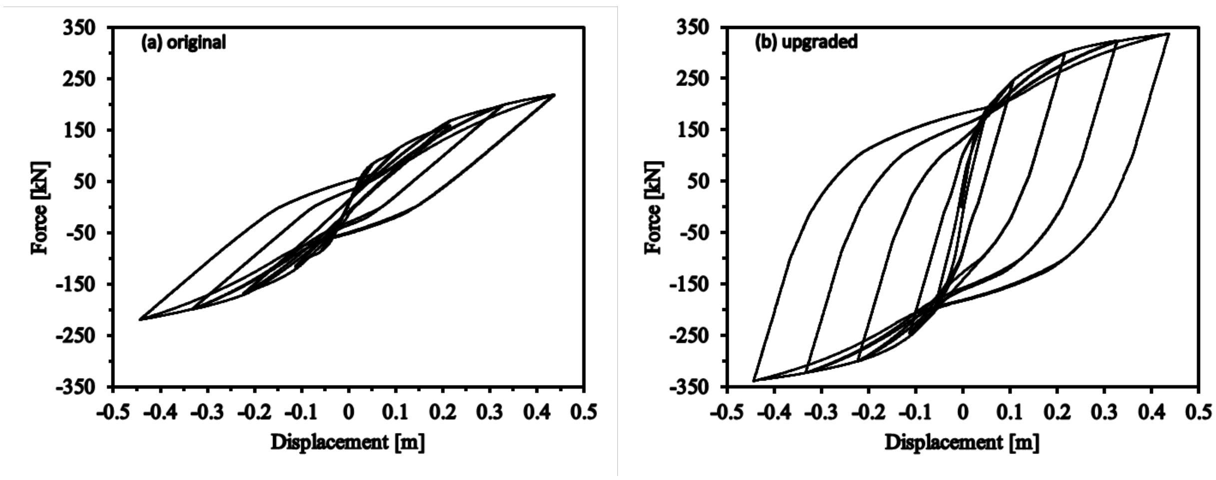


Figure 5. Hysteretic response of the original and the upgraded structure (load history given in Al-Mamoori (2019))

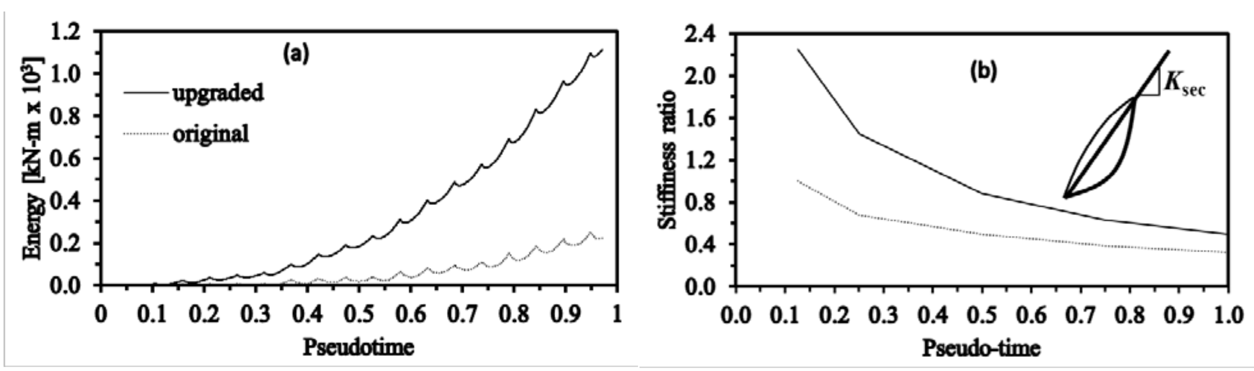


Figure 6. (a) Hysteretic energy dissipated by original and upgraded frames; (b) Stiffness ratio under reversed cyclic loading

2.4. Response under earthquake ground motions

An ensemble of 7 accelerograms rather than the minimum of three as recommended in EC8 (Eurocode 8, 1998) was used for the analysis of the structure. The natural accelerograms adopted in this study were selected from the PEER-NGA (2009) database. Originally 660 horizontal records of EGM were selected (Al-Mamoori, 2019). These records were defined based on the ground type C given a peak ground acceleration greater than 0.2g. In the present study, the selection of the 7 natural accelerograms and their scaling were conducted using the procedure recommended in Martinez-Rueda (2017). Table 1 shows a summary of the EGMs selected for time-history analysis. Note that the last columns of Table 1 give the time required to achieve 1% (t_{01}) and 98% (t_{98}) of the Arias intensity (Arias, 1970) for each record. These times were used to estimate the strong motion phase used for analysis. Additionally, to be able to assess residual displacements 10 sec of zero ground acceleration were included after the strong motion phase of each accelerogram. Finally, a peak ground acceleration (PGA) of 0.6g was used to describe the intensity of seismic action considered here as of high seismicity (HS) level. Figure 7 shows an example of the spectra for the selected EGMs scaled to HS level.

Table 2 summarises the results from the time-history analysis for the average and entire range of scaled EGMs. In general, the average results indicate that the incorporation of C-devices tuned at a strength of $0.50F_d$ at the beam-column connections can result in significant reductions of seismic response in terms of the top displacements Δ_{max}, rotational ductility demand θ_{max} at the plastic hinges formed at the base of the columns and residual displacements Δ_{res}. An opposite trend is observed with respect to the base shear V_{max}.

Table 1. Summary of 7 EGMs (horizontal and vertical components) selected for time-history analysis

EGM Code	Earthquake	Year	Component	Magnitude (M_w)	PGA (horizontal) [m/sec2]	PGA (vertical) [m/sec2]	t_{01} [sec]	t_{98} [sec]
1551	Chi-Chi, Taiwan	1999	TCU138W	7.62	2.022	1.104	24.60	66.77
1197	Chi-Chi, Taiwan	1999	CHY028N	7.62	7.459	3.368	29.58	50.81
4457	Montenegro, Yugo.	1979	ULA090	7.10	2.237	2.213	2.31	28.38
787	Loma Prieta	1989	SLC360	6.93	2.719	0.890	6.01	24.49
1052	Northridge-01	1994	PKC360	6.69	4.246	1.660	2.34	16.24
4863	Chuetsu-oki	2007	65036EW	6.80	3.677	1.549	20.84	46.01
1513	Chi-Chi, Taiwan	1999	TCU079N	7.62	4.163	3.845	24.30	58.62

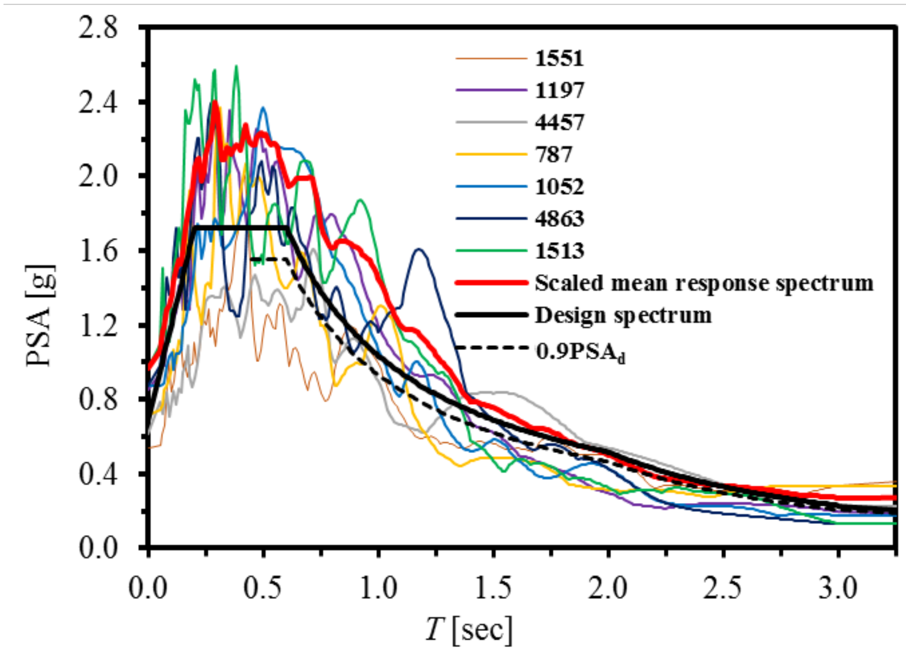


Figure 7. Mean and response spectra of scaled EGMs compared to the adopted design spectrum

Table 2. Summary of time-history analysis results

EGMs	Δ_{max} [m]		θ_{max} [rad]		Drift reduction (Δ_{red}) [%]	Δ_{res} [m]		$[V_{max}]$ [kN]	
	Orig.	Upg.	Orig.	Upg.		Orig.	Upg.	Orig.	Upg.
1551	0.84	0.47	0.043	0.025	44.03	0.107	0.058	282	367
1197	0.33	0.31	0.012	0.015	4.98	0.013	0.074	218	328
4457	0.52	0.40	0.022	0.021	22.22	0.054	0.012	229	363
787	0.57	0.27	0.024	0.013	52.43	0.042	0.019	240	343
1052	0.35	0.24	0.013	0.011	30.67	0.036	0.005	197	309
4863	0.37	0.25	0.014	0.011	33.31	0.040	0.016	215	331
1513	0.38	0.29	0.014	0.013	25.37	0.034	0.012	222	333
Mean	0.48	0.32	0.0203	0.0157	30.43	0.047	0.028	229	339

The seismic response of the portal frames with and without C-devices subjected to the most and least critical scaled EGM (see Table 2 in terms of Δ_{max}), are illustrated in Figure 8. It is observed that the incorporation of the C-devices tuned at a strength of $0.5F_d$ is effective as it leads to reduction in the displacements (Δ_{red}) of about 44% under the most critical EGM while a displacement reduction of 5% was found for the structure under the least critical EGM scaled at the HS level. Figures 9 compares the hysteretic behaviour of the original and upgraded frames under the scaled critical EGM. It can be clearly seen that due to the application of the devices, the stiffness degradation has been reduced. It is important to highlight that the effectiveness of the device calibration procedure under the most and least critical EGM were also studied by Al-Mamoori (2019). These studies showed that the device strengths tuned at LEDS have not induced an undesired brittle behaviour in the upgraded structures.

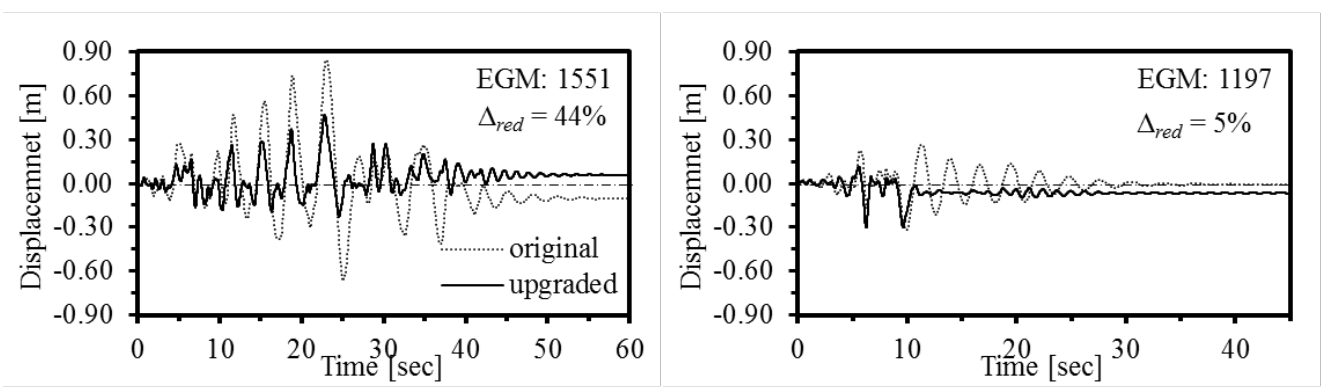


Figure 8. Comparison of time-history response between original and upgraded frames under the most and the least critical scaled EGMs.

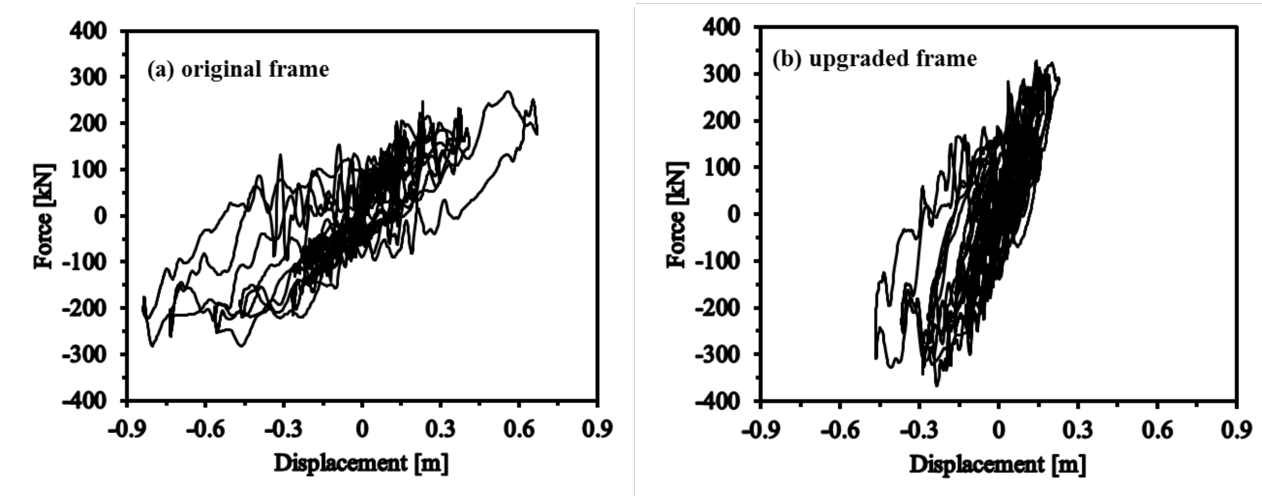


Figure 9. Comparison of hysteretic responses of the original and upgraded frames under the scaled 1551 EGM.

2.5.Seismic displacement demand

Figure 10a shows the value of Δ_{max} for the frame in its original and upgraded states under the scaled EGMs while Figure 10b compares the results in the original and upgraded frames in terms of average value of the displacement. It can be seen that the points corresponding to the upgraded frame have an upper limit of about 0.45 m while the same limit for the original frame is about 0.85 m. It can be also noted that all points lie below the 45 degree line (100%). This fact suggests that the incorporation of devices results in consistent reductions of damage in terms of displacement demands.

With respect to inter-storey drift, Figures 11a and b show the full set and average drift values, respectively. According to Figure 11b, the average inter-storey drift exceeds 4.65% and 3.1% for the frame in its original and upgraded states, respectively. The influence of the devices is relatively significant leading to a drift reduction of 30%.

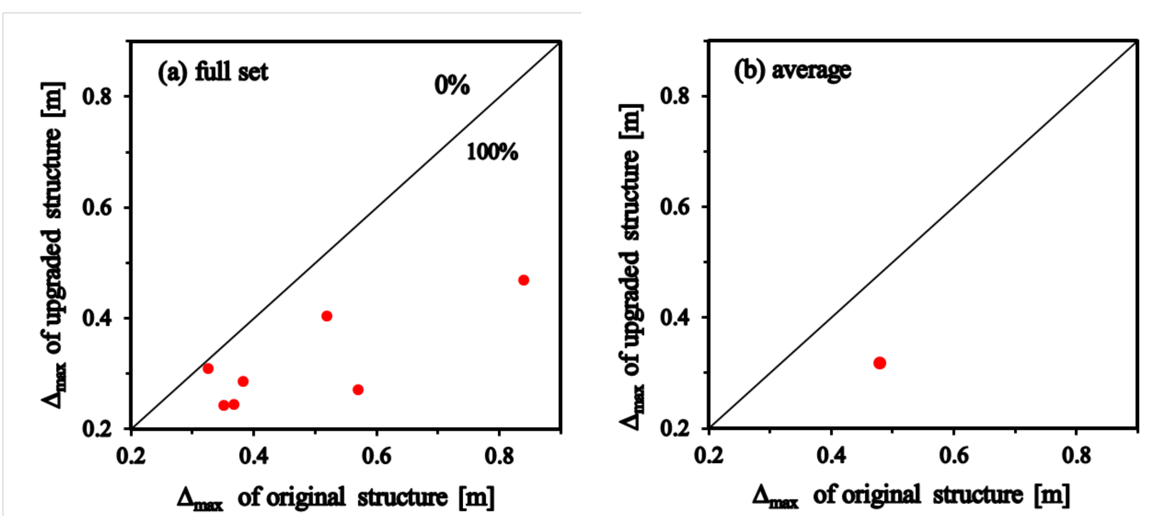


Figure 10. Comparison of displacement demands between original and upgraded structures for the entire range of records.

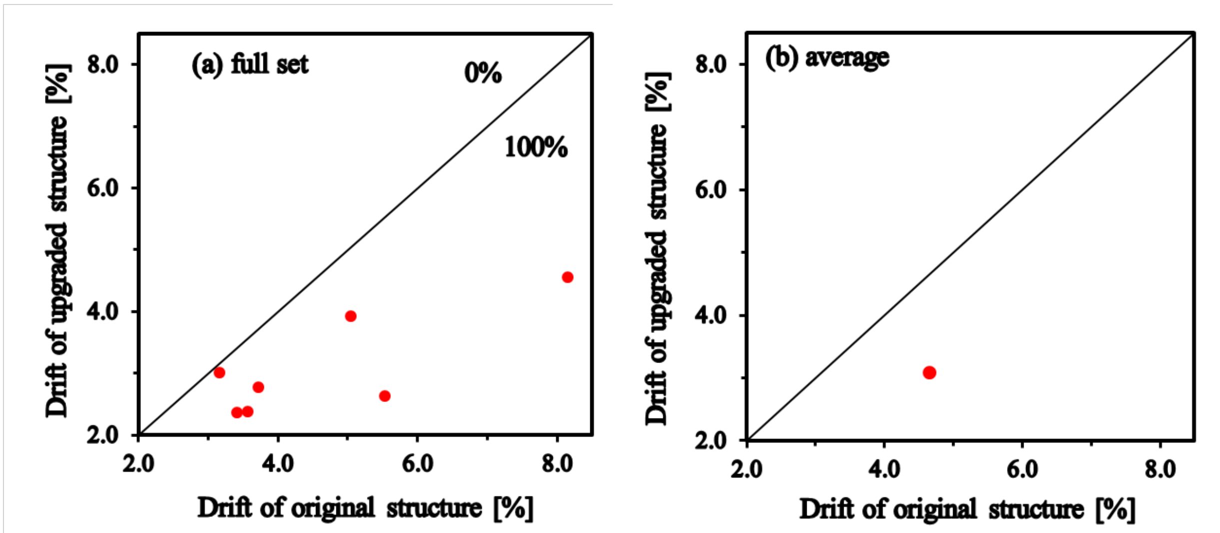


Figure 11. Comparison of inter-storey drifts for the entire range of scaled EGMs

3. CONCLUSIONS

A new seismic retrofitting technique for precast RC portal frames using yield C-devices locally incorporated at the PC beam-column joint regions has been proposed. This technique aimed to introduce hysteretic damping as the structure responds to seismic excitation through the insertion of C-devices made of mild steel at the PC connections.

From the results of time-history analyses it can be concluded that the performance of the portal frames with devices is highly satisfactory. An equation to estimate the maximum device strength required to avoid an undesirable failure mode of the upgraded frames was also proposed.

The influence of the device strength tuned at $0.5F_d$ is relatively significant leading to an average drift reduction of 30% for upgraded structures subjected to 7 EGMs scaled to a HS level. This technique also results in a substantial reduction of residual displacements and rotational demands at the critical region of the columns. It can be concluded that the application of the devices produces a moderate increment in both the structural stiffness, and the base shear.

Although beyond the scope of this paper, the concept of the frictional joints at the PC beam- column joints aimed to improve the quality of the analytical tools has also been considered by the authors elsewhere to investigate the application of the C-Device for seismic upgrading of the precast concrete structures. With proper consideration of the interaction between the PC beam and columns, it is possible to more closely assess the actual ductility demand of a given PC portal frame in its original and upgraded frames and to assess more accurately its deformation capacity and the effectiveness of the adopted device strengths.

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