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Evaluation of Vulnerable Reinforced Concrete Frames Retrofitted with Pre-Loaded Thin Panels

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ABSTRACT

Interaction between a panel and its boundary frame has to be sufficient when vulnerable reinforced concrete (RC) frames are retrofitted with thin RC panels. This affects the transfer mechanism of the lateral forces in the overall system. In this experimental study; a bare RC frame retrofitted with a pre-loaded thin RC panel is tested under lateral reversed cycling loads. Overall response of this frame is investigated. To assess the effect of the pre-loaded panel on the behaviour of the RC frame, the results are compared with the one retrofitted with a non-loaded panel. The experimental study shows that for the chosen pre-load level the lateral load carrying capacity of the frame has increased slightly and no significant effect is observed about failure mode of the system.

Keywords. Strengthening, Reinforced Concrete Frame, Pre-loaded Thin Panel

INTRODUCTION

Introducing RC shear walls to RC buildings is one of the most commonly used retrofitting techniques. The advantage of this technique is that, these walls are efficient in controlling the overall lateral strength and stiffness and thereby reducing damages in existing frame members. However, at RC structures with low concrete compressive strength, the poor interaction of these walls with the surrounding frames can cause brittle shear failure in the columns that can lead to massive failures and undesired soft-story mechanisms.

Many research on structural walls and results of detailed applications have been reported, (Sugano and Fujimura, 1980; Yuzugullu, 1980; Higashi et al., 1980; Altin et al., 1992; Pincheira and Jirsa, 1995; Frosch et al., 1996; Lombard et al. 2000; Inukai and Kaminosono 2000). The common results show that the response of walls with the structure depends mainly on the application details. In particular, proper anchorage of rebars to beams and closely spaced mesh increases the deformation abilities. If there is poor detailing and lack of load transfer between existing and new members, this may lead to brittle failure of infill walls or reduction in ductility of the system.

In this study, nearly ½ scale, one bay and one story specimens were tested by using the loads simulating earthquake effects. The experimental work is composed of testing one bare frame that is strengthened with a non-loaded thin panel and another one strengthened with a pre-loaded thin panel. The idea of imposing pre-loads on panels is to enhance the contact between the beam and the infill wall, causing the shotcrete panel and the surrounding frame to work together for a long time ensuring that the lateral loads continue to be transferred through the system.

EXPERIMENTAL WORKS

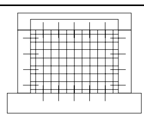
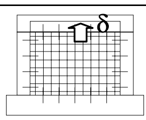
Cast-in-situ panel is used to form an infill in the vulnerable bare RC frame that was chosen to represent the weak column/strong beam type structures. The frame had non-seismic details such as large spacing of hoops and no hoop in beam-column connection region. All test specimens are single story and single bay frames as shown in Table 1.

Properties and construction details of the test specimens

One story, one bay nearly half-scale RC frames with a portion of slab on top and a foundation at the bottom has been constructed in the laboratory, Figure 1. The cross sectional dimensions of columns and beam of the frames are 20 cm by 25 cm and 20 cm by 32.5 cm, respectively. The height and the width of the frames are 152.5 cm and 220 cm, respectively. The height of the thin panel is 120 cm and the width is 170 cm. Longitudinal reinforcement of the frames consist of 16 mm re-bars having an average yield stress of 270 MPa and the wire mesh used in the shotcrete panel has a diameter of 6.0 mm and has yield stress of 320 MPa. Longitudinal reinforcement ratio of the column is 1.6%. The horizontal and vertical reinforcement ratios for the shotcrete panels are 0.21%, per unit length. The dimensions and the reinforcement details are given in Figure 2. Longitudinal reinforcement of the columns continues to the bottom of the foundations, so there is no lap-splice problem at the level of foundation.

The concrete compressive strengths of frames and the panels are 10 MPa and 25 MPa, respectively. The pre-load level is L/300 of the span. L is the distance between the inside face of the columns.

Table 1. Test specimens

Specimen	Type	Pre-Load Level	Concrete Compressive Strength (MPa)	
			Frame	Panel
R		0	10	25
SP1		L/300	10	25

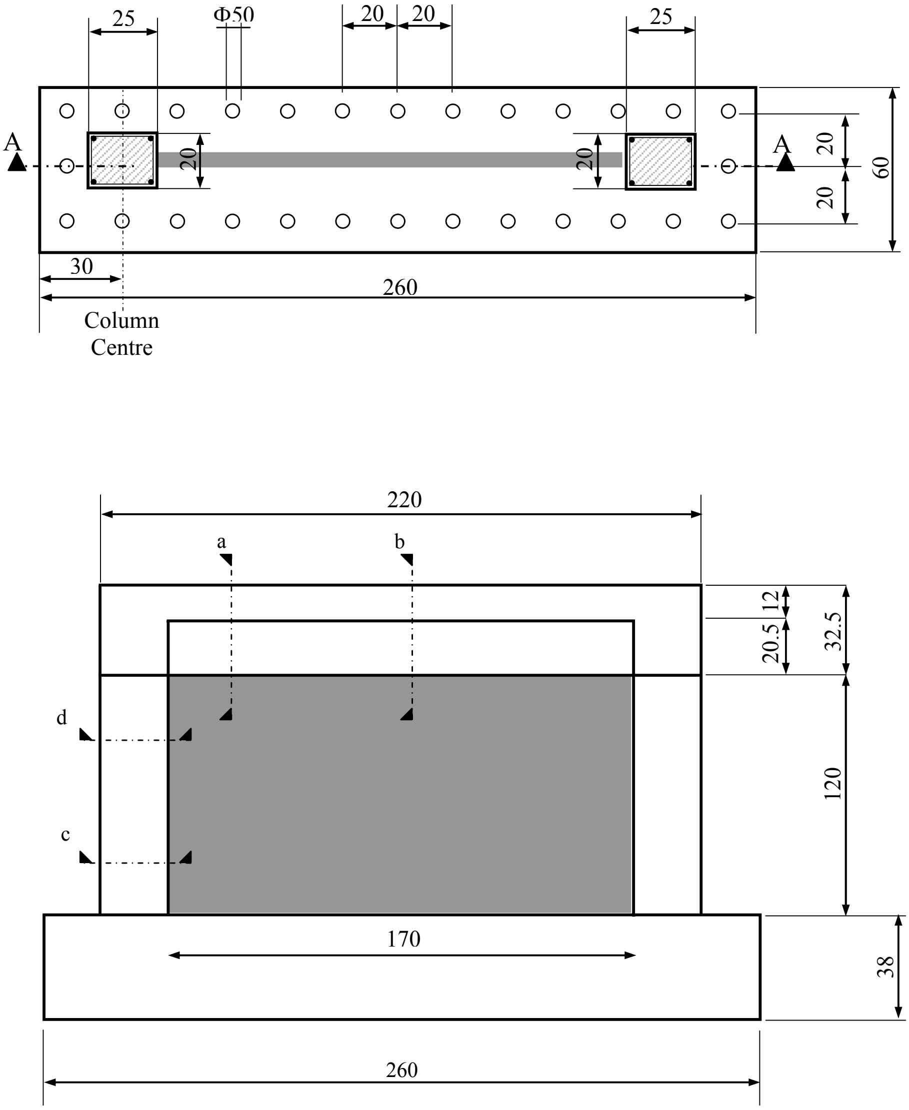


Figure 1. Geometry of the specimens

R is the bare frame retrofitted with a non-loaded panel, which was tested as a reference specimen. *SP1* is the specimen which is retrofitted with a pre-loaded panel. The wire mesh is placed at the centre axis of the frame. Full contact of the panel is established by lapping the infill reinforcement to the anchorages placed in the frame members. The anchorages used are Φ10 mm steel bars which are placed in the beam and the columns 30 cm apart from each other by epoxy resin. The total length of an anchorage is 35 cm; 20 cm of it is in the panel while 15 cm is in the RC member.

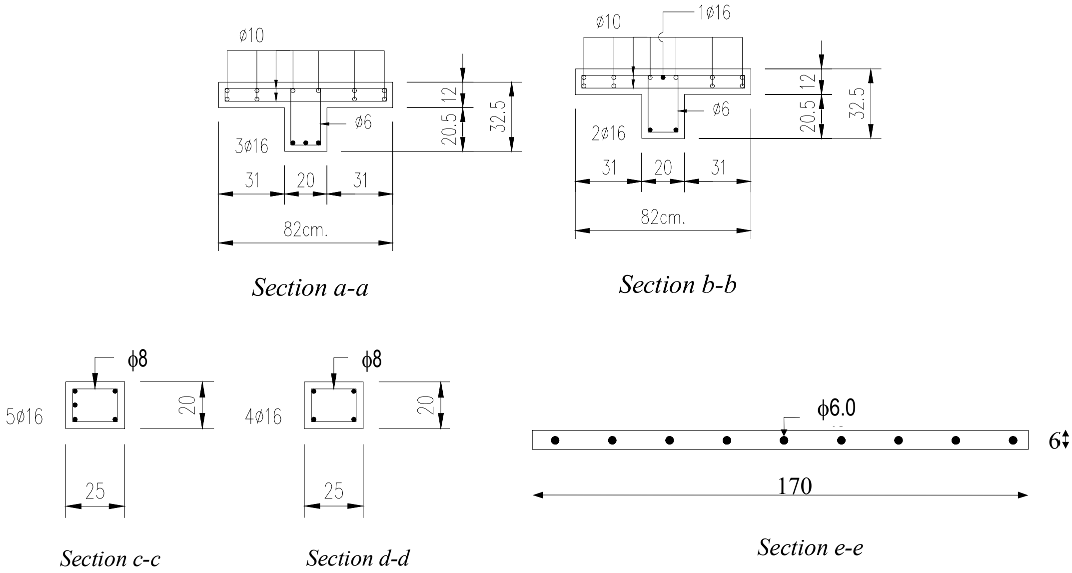


Figure 2. Reinforcement details of specimens

Construction of *R* and *SP1* are shown in Figure 3. Two layers of formwork, which are placed at a distance of 3 cm behind and in front of the wire mesh, were used and the concrete is poured from two holes right beside the beam. To construct *SP1*, the beam of a bare frame is cambered before strengthening with the panel. The beam is cambered by means of special screws that are placed symmetrically to the right and left side of the beam in the middle between the foundation and the floor. When the concrete has cured, the screws are removed and the beam is released causing a pre-load on the panel. The level of the cambering is 5.6 mm.

a) *R*, with a non-loaded panel **b)** *SP1*, with a pre-loaded panel

Figure 3. Construction of the specimens

Test Setup, Instrumentation and Data Acquisition

Axial load which was kept constant throughout the test was applied on the columns by means of a hydraulic jack and lateral cycling load imposed as displacement reversals was applied to the specimen with two MTS 250 kN-capacity hydraulic actuators that were placed at the beam centre line. The applied axial load was approximately 20% of the axial load carrying capacity of the columns.

Since the loading was aimed to simulate the effect of seismic action, reversed cycling displacement reversals with increasing intensity was applied to the specimens. Up to 0.467 mm top displacement, while observing the elastic behaviour of the specimens, the target displacement values are applied once on the specimens. Beyond 0.467 mm, each displacement cycle is repeated thrice for both pushing and pulling cycles.

EXPERIMENTAL RESULTS

Although there are several measurements taken on the specimens, the top displacements versus base shear relations and the damage patterns are selected to be presented here. The results obtained for each specimen are given below.

Test Results of R

The virgin RC frame was retrofitted with a non-loaded RC thin panel to fabricate *R*, which is the reference specimen. The axial load applied on each column was 125 kN.

Base shear versus top displacement diagram is given in Figure 4. The maximum base shears in pushing and pulling directions are 297 kN and 297 kN, respectively occurred at ±28 mm displacement cycles which corresponds to 2% story drift. The maximum story drift reached for this specimen is 2% and the corresponding base shear forces of the third cycle were 189 kN and 171 kN in pushing and pulling directions, respectively.

The separation between the panel and the frame members and a shear crack took place at upper end of the right column (the column where the actuator was connected) observed at the +0.467 mm displacement cycle. The base shear was reported as 93 kN. The first diagonal crack observed on the panel during the displacement reversal of +2.8 mm. The base shear was reported as 217 kN.

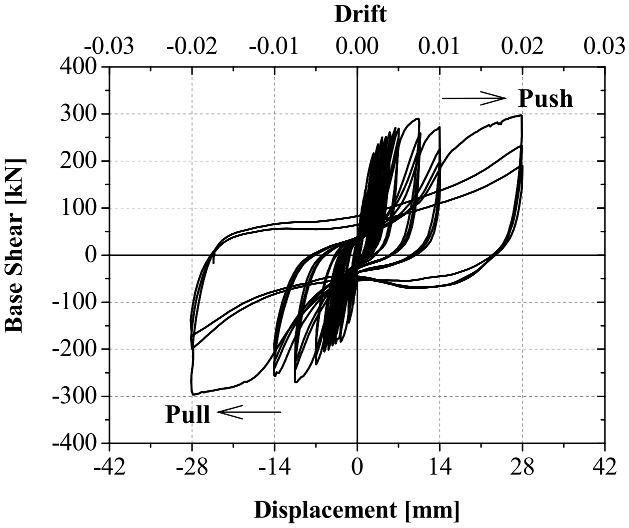


Figure 4. Base shear-top displacement relation of *R*

The cumulative damage pattern is shown in Figure 5. The maximum crack widths measured at 1% and 2% story drift are given in Table 2. The column shear failure is observed dominantly at top of the columns.

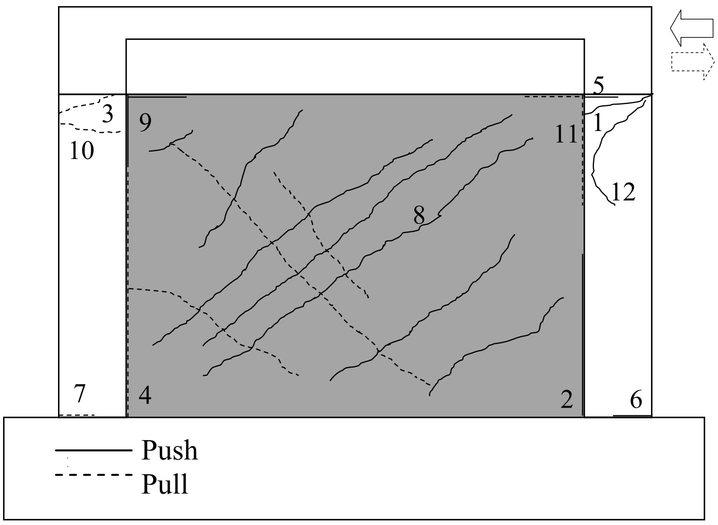


Figure 5. The cumulative crack pattern of *R*

Table 2. The maximum crack widths of *R* at 1% and 2% story drift

Crack #	story drift %1	story drift %2	Crack #	story drift %1	story drift %2
1	>3.5	>3.5	7	0.8	1.0
2	2.9	3.5	8	0.6	0.4
3	0.5	>3.5	9	>3.5	>3.5
4	2.2	2.3	10	1.7	0.2
5	2.5	>3.5	11	3.0	>3.5
6	0.6	1.0	12	-	1.8

Test Results of *SP1*

This specimen is produced by retrofitting of a bare frame with a pre-loaded panel.

Base shear versus top displacement diagram is given in Figure 6. The maximum strengths obtained in pushing and pulling directions are 332 kN and 336 kN at ±28 mm. The maximum story drift reached for this specimen is 2% and the corresponding base shear forces of the third cycle were 173 kN and 192 kN in pushing and pulling directions, respectively. The significant strength decrements were examined at the second and third cycles of the last displacement reversals.

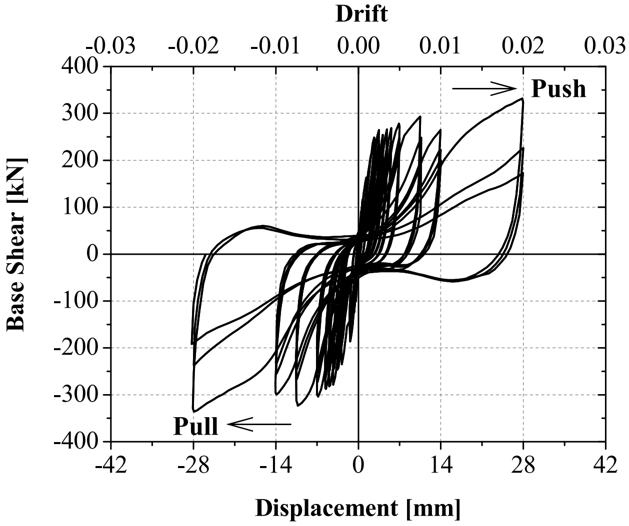


Figure 6. Base shear-top displacement relation of *SP1*

The separation between the panel and the frame members and a shear crack took place at upper end of the right column (the column where the actuator was connected) observed at the +0.467 mm displacement cycle. The base shear was reported as 72 kN. The first diagonal crack observed on the panel during the displacement reversal of -4.9 mm. The base shear was reported as 282 kN.

The cumulative damage pattern is shown in Figure 7. The maximum crack widths measured at 1% and 2% story drift are given in Table 3. The column shear failure is observed dominantly at top of the columns.

Table 3. The maximum crack widths of *SP1* at 1% and 2% story drift

Crack #	story drift %1	story drift %2	Crack #	story drift %1	story drift %2
1	0.4	0.4	10	>3.5	>3.5
2	>3.5	>3.5	11	>3.5	>3.5
3	1.2	1.9	12	0.4	0.4
4	>3.5	>3.5	13	>3.5	>3.5
5	0.4	0.4	14	0.3	0.3
6	1.8	2.4	15	2.0	2.6
7	0.4	0.4	16	0.4	0.6
8	0.3	0.3	17	1.0	>3.5
9	0.3	0.3	18	0.5	0.6

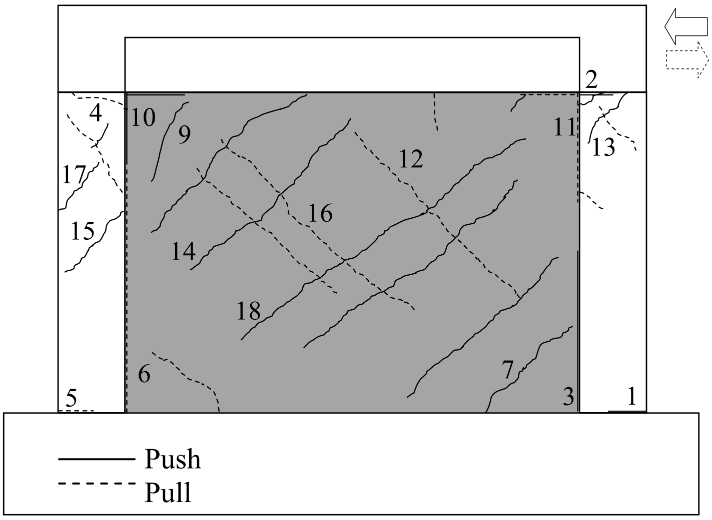


Figure 7. The cumulative crack pattern of *SP1*

EVALUATION of EXPERIMENTAL RESULTS

The comparisons of the test results of the specimen are presented below. Failure modes, load carrying capacities, initial stiffnesses, and energy dissipation capacities are discussed in detail.

Failure modes

In the reference frame and the retrofitted frame, the observed failure mode is shear failure at column ends.

The separation between the shotcrete panel and the columns was gradually increased as the target displacement increased. Its width reached to approximately 2.0 mm at 0.5% story drift. At 0.75% story drift, the shear cracks at top ends of the columns were increased dramatically. As a result of this damage pattern, a noticeable strength decrement has been observed. However, when the lateral drift was beyond 1.0%, the authors estimate that a new load interaction mechanism was developed based on arose large contact surface between the shotcrete panel and the beam. Therefore, the specimen regained its strength until 2% story drift.

During the cambering process, a 1.5 mm-width crack has occurred in the middle of the slab (along the width) all the way to the beam. The authors estimate that this crack might have a negative effect on the behaviour of the system, so that the lateral load carrying capacity of the frame did not increase as much as it was expected, and also the failure mode did not change much.

Lateral load carrying capacity

The lateral load carrying capacity has increased by 12% for the specimen with a pre-loaded thin panel compared with the reference frame's. The backbone curves of the hysteretic responses of the two specimens are presented in Figure 8. The points of backbone curves are the maximum load value of the first cycle at each target displacement level.

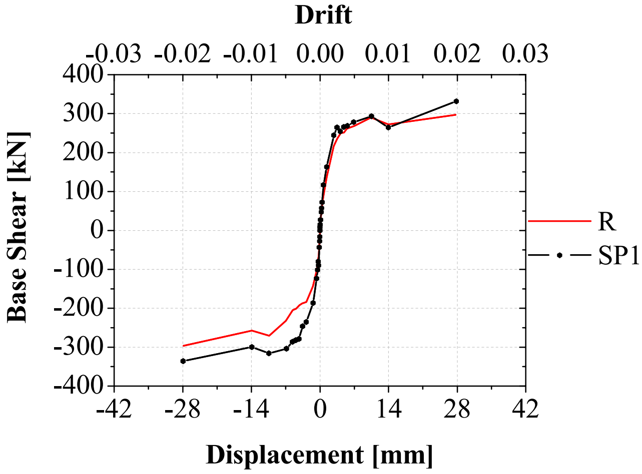


Figure 8. The comparison of backbone curves of the specimens

Initial stiffnesses

The initial stiffness of the frame with the pre-loaded panel right before the first cracks occurred in the system is the same as the frame with the non-loaded panel.

Cumulative energy dissipation

As shown in Figure 9, the cumulative energy dissipation of the specimen with the pre-loaded panel is 1.3 and 1.1 times greater than the non-loaded one at 1% and 2% story drift levels, respectively.

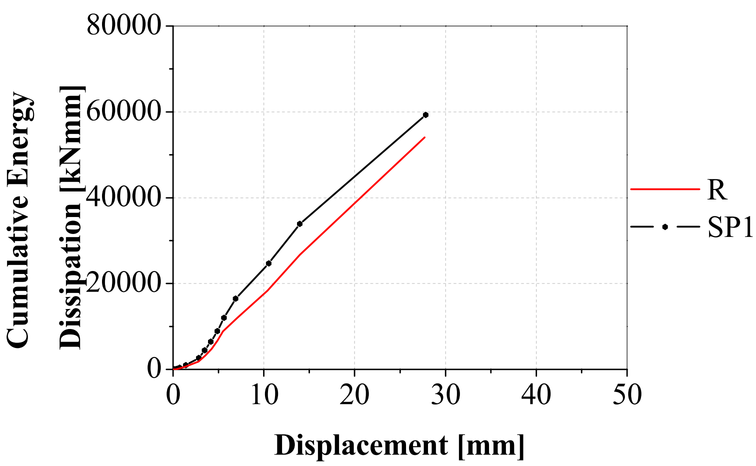


Figure 9. Cumulative energy capacities

CONCLUSIONS

In this study, beam of a RC frame is cambered before the construction of a thin RC panel. After the cure of the concrete of the panel, the beam is released causing pre-loads on the infill walls to enhance contact surface for raising their seismic performance.

Study shows that the lateral load carrying capacity increased by 12% for the specimen with a pre-loaded thin panel compared with the one with a non-loaded panel. The initial stiffnesses of the frames are the same. The cumulative energy dissipation of the specimen with the pre-loaded panel is 1.3 and 1.1 times greater than the non-loaded one at 1% and 2% story drift levels, respectively. Damage mode has not been affected much.

Further studies can be conducted at different levels of the pre-loads to examine overall behaviour of the system and to see if any pre-load level can be effective on the lateral load carrying capacity and/or the failure mode of the system.

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