FIELD TEST OF RC SCHOOL BUILDING RETROFITTED BY POST-TENSIONED RODS

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ABSTRACT :
The 1985 Mexico City Earthquake and the 1999 Chi-Chi Earthquake revealed the seismically insufficient performance of the RC school buildings both in Mexico and Taiwan. Seismic retrofit of these numerous vulnerable buildings is an important societal issue to be resolved. Field pushover tests provide a direct approach to assess the seismic behavior of real building structures and a precious chance to verify the seismic retrofitting methods. Renewal of school buildings provides an opportunity to make use of the buildings to be demolished as the specimens for the field tests. In August 2007, one existing 2-story school building of Guanmiao Elementary School in Taiwan had been successfully field tested. This paper reports the test results of two specimens: the school building in its as-built form and the school building retrofitted by post-tensioned rods. The as-built specimen consisted of two 4-bay/2-story RC frames partially infilled with brick walls. The seismic retrofit design required installing sixteen post-tensioned rods and stressing each rod to half of its yield strength. Retrofitting school buildings using post-tensioned tendons is a technology widely used in Mexico. It has been proved to be an economic and constructible retrofitting method. Experimental observations in field tests indicated that the seismic performance of school frames can be effectively retrofitted using post-tensioned rods. Main results of these tests are reported, analyzed and discussed in this paper.

KEYWORDS:
Field Test; Retrofit; Reinforced Concrete; School Building; Post-Tensioned Rod

1. INTRODUCTION

Numerous school buildings in Taiwan were damaged in the 1999 Chi-Chi Earthquake. Because of their standard plan layout, characterized with openings in the longitudinal direction and partition walls in the transverse direction, common failure patterns, such as failure in the longitudinal direction due to lack of walls, short-column damages due to constraint by windowsills, and strong-beam-weak-column behavior due to weak columns were found as shown in Figure 1 (Loh et al. 1999). Likewise, on September 19, 1985, an 8.1-Richter magnitude earthquake occurred in Michoacan, Mexico, which caused severe damages of school buildings. In the aftermath of the earthquake, researches on the repair and upgrading of those damaged and vulnerable buildings have been the focus of seismic retrofitting design. The application of high-strength post-tensioned (PT) steel cables was developed in Mexico, and further investigated at the University of California and at the University of Texas (Guh 1989, Teran-Gilmore et al. 1995, Pincheira et al. 1992). These studies indicated that the PT cable technique is very effective for enhancing strength and stiffness of moment resisting frames. Moreover, because of the small diameter of PT cable braces, the PT technique does not interfere with lighting and views. This is also advantageous for the architect to have more freedom in exterior architecture. Therefore, a large number of school buildings were retrofitted with PT in Mexico (Figure 2).

In order to verify the effectiveness of PT steel cables as a retrofit technique, the Center for Research on Earthquake Engineering (NCREE) and the National Autonomous University of Mexico (UNAM) cooperate for the field test of RC school building retrofitted by PT rods. The old school buildings that scheduled to be demolished in Guanmiao elementary school in Tainan, south of Taiwan have been chosen as the subject of two push over tests for the prototype and the retrofitted specimen using PT rods. Test results can verify both the retrofitting technology and the assessment methodologies.

Figure 1 School building was damaged along the longitudinal direction in the 1999 Chi-Chi Earthquake in Taiwan.

Figure 2 School building retrofitted with PT tendons along the longitudinal direction in Mexico.

2. SPECIMEN DESIGN AND TEST PLAN

2.1. Concept of Post Tensioned Cable Design

Design criterion for the post tensioned cable retrofit assumes that under maximum lateral displacements, cables will remain elastic so that after the earthquake motion the structure will not exhibit permanent deformations. To achieve this performance objective, cable tension is typically near half of its yield strength. The number of cables results from the amount of story shear to be resisted by the bracing system. Also, cable tension takes into consideration the maximum allowable displacement (deformation capacity) that the original structure may resist without significant damage. Since typically the original structure has non-ductile detailing, plus the additional axial loads imposed by the braces, allowable lateral displacement of the original RC structure may control the bracing stiffness, and therefore, the number of cables as well as the tensile force in them.

2.2. Test plan and setup

The tested school building contains 14 classrooms distributed in 2 floors and with a cantilever corridor on one side. Two specimens, a prototype of two classrooms and a retrofitted specimen using PT rods (Figure 3a), were tested under monotonic static lateral load along the longitudinal direction. The remainder part of the building was reinforced with large steel braces to serve as reaction wall. Four hydraulic actuators were installed at roof and floor to apply the lateral loading. Loading was manually controlled to maintain a 2:1 ratio during the test to simulate the earthquake loading for the fundamental mode of vibration of typical low-rise RC buildings (Figure 3b). First, the building was cut between classrooms 5 and 6; the PT specimen was pushed to north by using classrooms 6, 7, 8 and 9 as reaction wall. Afterward, the corridor between classrooms 6 and 7 was removed to allow lateral displacement of Specimen 4 (see Figure 3), which was pushed from classroom 9 to north.

Displacement transducers were instrumented at both the free and the loaded ends to measure the relative displacement of each frame at each floor of specimens. While the specimens were tested, loading stopped at about every increment of roof drift ratio of 0.5%, so that damaging patterns could be recorded by pictures and crack pattern sketches. After the displacement gauges were removed at the drift ratio of 6%, the specimens were pushed to collapse.

2.3. Specimen

Figure 4 shows the standard structural plan and column sections. Column steel ratio is about 2.5% and the hoop spacing is 20 cm. One of the differences between Specimens 3 and 4 is in column type C4 for PT and C2 for prototype specimen at location C-3, respectively. As shown in Figure 5a, 8 sets of 22-mm high strength rods were added forming an angle of 57 degrees from the horizontal spanning column lines 2 and 3, and 3 and 4 in each frame. Two rods per set were placed at both sides of each frame. The location and amount of brick infills with 1B width of the prototype and PT specimens are shown in Figure 5. Material strengths and details of reinforcement were found by sampling after the tests. Average concrete compressive strength and steel yield strength were about 23MPa and 330MPa, respectively. In addition, yield and ultimate strengths of rods were 726MPa and 847MPa, respectively (see Table 1).

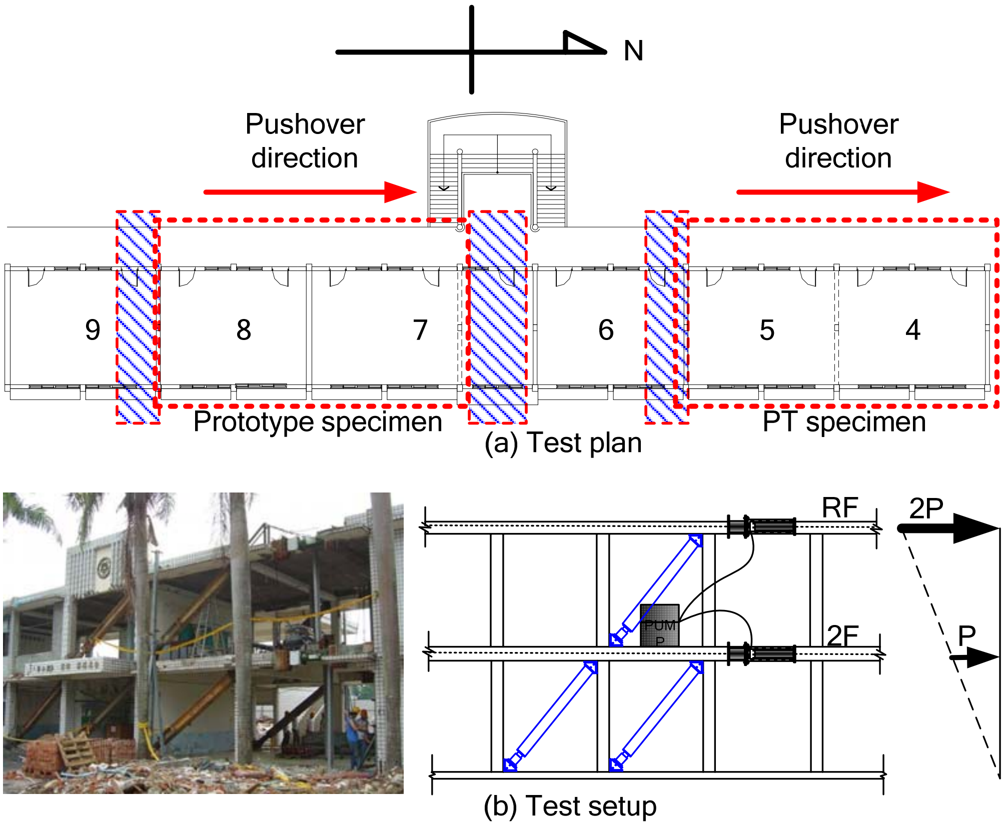


Figure 3 Test plan and test setup.

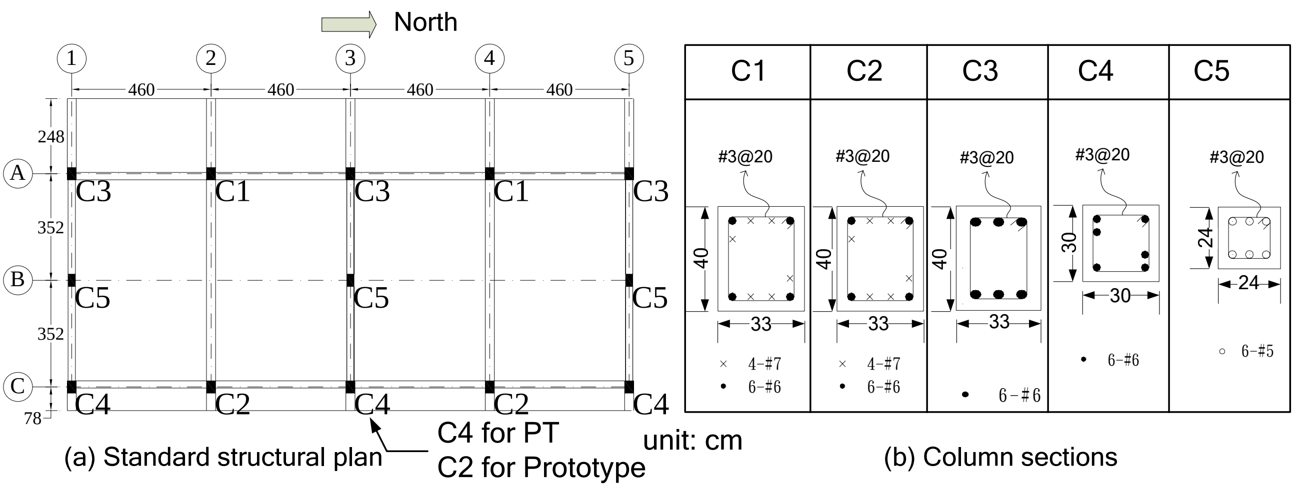
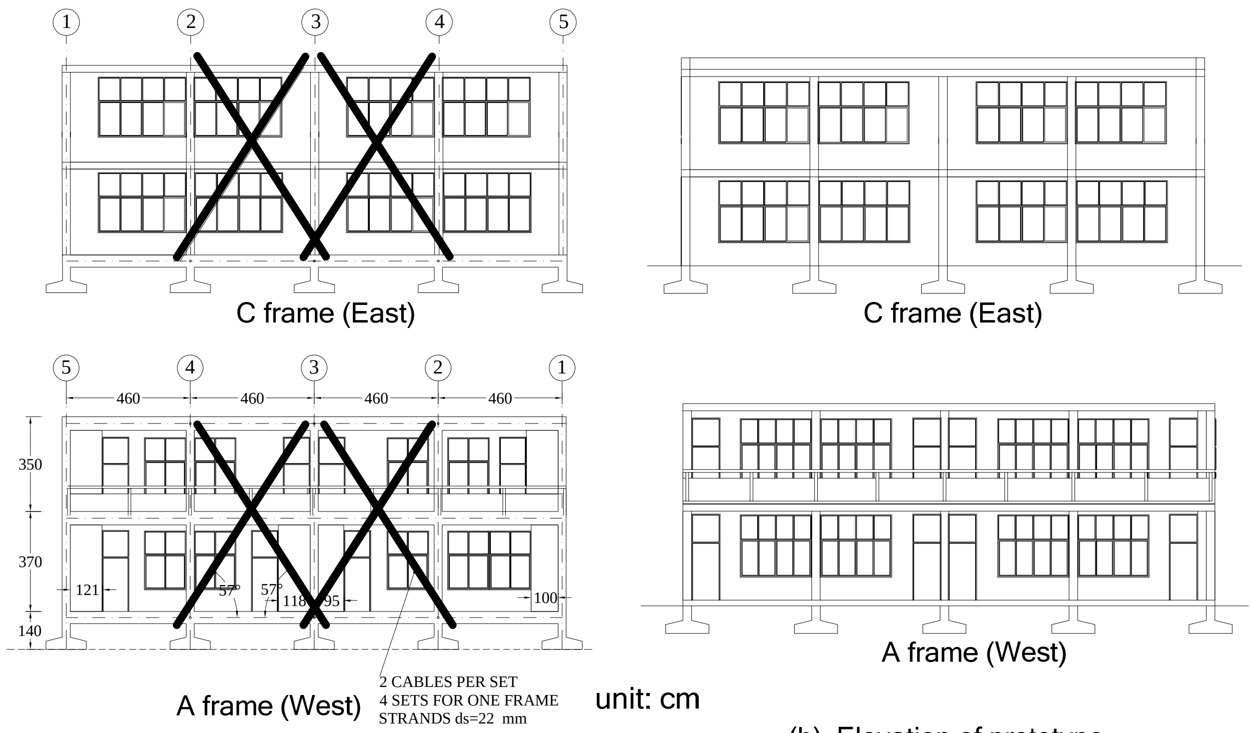


Figure 4 Standard structural plan and column sections



(a) Elevation of PT specimen

(b) Elevation of prototype specimen

Figure 5 Elevation of specimens

Table 1 Material property

Specimen	Concrete	Steel				Rod(D22)	
	f_c' (MPa)	f_y (MPa)				f_y (MPa)	f_u (MPa)
		#3	#4	#5	#7		
Prototype	21.1	359.0	289.0	402.7	329.7	-	-
PT	23.5	324.7	327.7	359.3	339.3	726.1	847.1

2.4. Anchorage Design

According to the PT retrofitting scheme (Figure 5a), a single pair of PT rods was anchored at the ends of columns 2 and 4, whereas in column 3, two pairs of PT rods were anchored at both ends of the column for each frame. Thus two types of anchorage blocks were designed to support biaxial moment and shear forces due to PT systems. Besides, shear-friction between the new and the existing concrete interfaces was considered as well. This section illustrates the design concept considering shear friction.

First, consider the single pair of PT rods (columns 2 & 4) and assume that the rods are tensioned to yield (Figure 6a). The demand on shear-friction may be evaluated by

$$V_{sf} = 2A_t f_y \cos\theta = 2\times 380\times 726\times \cos 57°\times 10^{-3} = 300\ kN \tag{2.1}$$

where A_t is the sectional area of rod, (=380 mm^2), and θ is the horizontal angle of rod, (=57°). Thus, the sectional area of shear-fraction reinforcement may be calculated by ACI 318-05 code as

$$A_{vf} = \frac{V_{sf}}{\phi f_y \mu} = \frac{300\times 10^3}{0.85\times 4200\times 0.6\times 9.81} = 14.3\ cm^2 \tag{2.2}$$

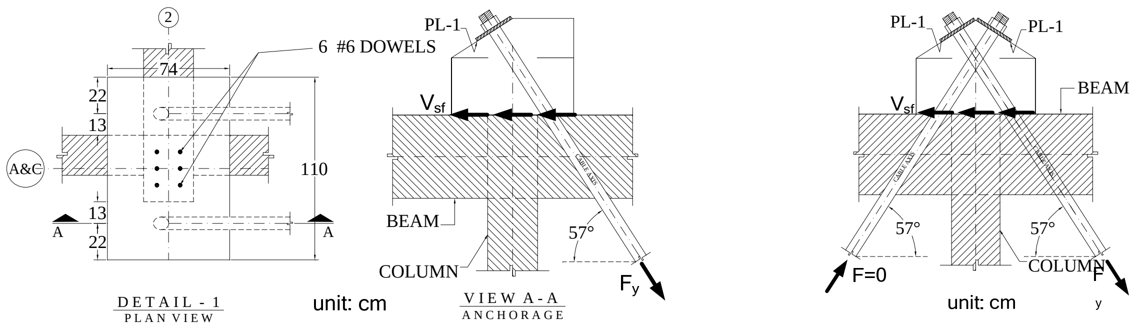
If 19-mm diameter bars (D19) are chosen, the number of dowels may be estimated by

$$n = \frac{A_{vf}}{A_{st}} = \frac{14.3}{2.87} = 5 \tag{2.3}$$

It was decided to use six D19 dowels with 180° standard hooks, which should be anchored into the column capital.

Secondly, as shown in Figure 6b, the anchorage for the two pairs of PT rods was designed to resist tension forces from the two diagonals. Initially, each rod was post-tensioned to $0.5\,f_y$. Because of the monotonic nature of the pushover test, the force in the tension side will reach the yield strength, while the force in the compression side will decrease near zero. Therefore, the demand on shear-friction for the double pair of rods was the same as that for the single pair. Six D19 dowels with 180° standard hooks were used for anchorage in column 3.

Following the same reasoning, as shown in Figure 7, the anchorage of foundation block was designed by using six D25 anchor bars doweled into the footing and was combined with foundation beam in the longitudinal direction. Figure 8 shows the photo of anchorage block.



(a) single pair of PT sets (b) double pairs of PT sets

Figure 6 Roof anchorage.

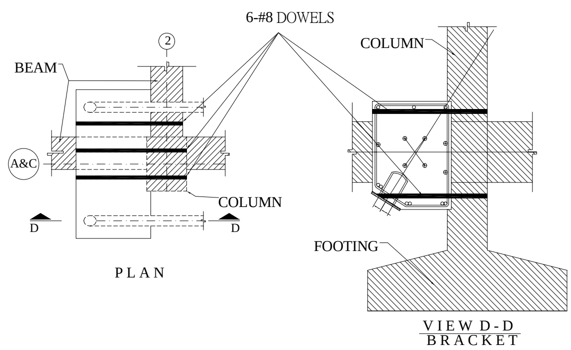


Figure 7 Detailing of foundation anchorage

Figure 8 Anchorage block of foundation

3. TEST RESULTS

3.1. Pushover Response

The test results of the PT and prototype specimens are illustrated with solid and dashed lines, respectively, in Figure 9. The maximum base shear for the PT specimen was 2985kN when the roof drift ratio was 0.98%. The base shear was reduced to 80% of the maximum strength of 2388kN at the roof ratio of 3.48%. Likewise, the maximum lateral force for the prototype specimen was 1444kN at the roof ratio of 0.86% and the reduced strength of 80% maximum lateral force was 1155kN occurred at 3.03%.

The maximum lateral force of PT was 2.1 times that of the prototype specimen. It means that the PT retrofitting technique can efficiently increase the lateral strength for typical school buildings. In addition, the initial stiffness, defined at 75% of the maximum strength was 72.5 kN/ mm for prototype and 97.4 kN/ mm for PT specimen, respectively. As may be observed from the results presented in Table 2, lateral stiffness was evidently improved with PT.

However, the ductility defined as the ratio of the displacement at 80% of the maximum strength to that measured at yield was 10.7 and 8.1, respectively, for the prototype and PT specimens (Figure 9 and Table 2). This means that the increase in axial loads in the existing columns due to the post-tensioned forces will decrease the ductility of structure.

3.2. Damage Patterns

Figure 10 shows pictures of the specimens at nearly collapse. Usually, the first story of school building possesses the severe damages. It is because that the school building carries the uniform floor plan and that the first story exerts the largest shear force. As may be observed, the strong-beam-weak-column behavior is apparent in both specimens. Beams and slabs remained almost undamaged when the columns and wing walls had failed to nearly collapse. As shown in Figure 10, the captive columns exhibited diagonal shear cracks because of the constraint of windowsills induced larger shear force. However, the flexural failures were observed in longer columns without constraint of windowsills.

As it was mentioned before, the PT rods enhanced not only the lateral strength due to the contribution of the horizontal component, but also increased the axial loading on the column due to the vertical component. Because of the larger axial load of the columns, the less ductility was observed. As shown in Figure 11, for the PT specimen, column A2 exhibited earlier shear cracks than those in the prototype. In addition, if Figures 11 and 12 are compared, side column C2, with low axial loading, showed fewer cracks than the inner column A2 because the latter supported higher axial loading at the same displacement. It means that the decrease in ductility of vertical members due to the increase in axial loading cannot be overlooked in design.

3.3. Discussion

Test results indicated that the increase in lateral force between the prototype and PT specimens was 1541kN which could be attributed to the horizontal component of PT force, to the axial loading effect from vertical component of PT force, and to the shear strength of brick wing walls.

Assuming an elastic perfectly-plastic material for the tendon, the ultimate strength can be estimated as the yield strength. As shown in the plan of the retrofitted specimen in Figure 5a, four sets of PT were tensioned during the monotonic push over test. Because there were two rods per set, the total horizontal component of the yield strength of 8 rods can be calculated as

$$P_{PT} = nA_t f_y \cos\theta = 8 \times 380 \times 726 \times \cos 57° \times 10^{-3} = 1202\, kN \tag{3.1}$$

where n is the number of rods in tension.

Moreover, the axial load upon column members due to the vertical component of PT force would increase both the flexural moment and the shear strength of the column. According to the retrofitting design used herein (Figure 5a), the increment in axial load of each column with PT was

$$\Delta N_{PT} = 2A_t f_y \sin\theta = 2 \times 380 \times 726 \times \sin 57° \times 10^{-3} = 463\, kN \tag{3.2}$$

Thus, the enhancement on lateral strength of columns due to the increment on axial load was estimated using

ACI318-05 code, and was equal to 109kN.

On the other hand, brick wing walls also contributed to the base shear of the buildings. Using the preliminary seismic evaluation method proposed by NCREE in 2003, the shear strength of brick wing walls can be evaluated as

$$V_{BW3} = \tau_{BW3} \sum A_{BW3} = 0.147 \times 4340 \times 240 \times 10^{-3} = 153\,kN \tag{3.3}$$

where τ_{BW3} is shear stress per section area of brick wing wall (=0.147 MPa), A_{BW3} is section area of brick wing wall (mm^2).

Summation of the contribution of the above-mentioned terms (cables, columns and wing walls) was 1465kN, which is close to the measured strength being equal to 1541kN. The difference could be attributed to variation of material properties. Comparing the three contributions, it can be concluded that the largest contribution to strength came from the tension force of the PT rods.

Table 2 Comparison of stiffness and ductility

Specimen	P_{max}	P_{80}	Δ_{max}	Δ_{80}	75% P_{max}	Δ_{75}	K	Δ_y	μ_{80}
	kN	*kN*	*mm*	*mm*	*kN*	*mm*	*kN/ mm*	*Mm*	
Prototype	1444	1155	61.1	213.9	1083	14.9	72.5	19.9	10.7
PT	2985	2388	70.0	248.5	2239	23.0	97.4	30.7	8.1

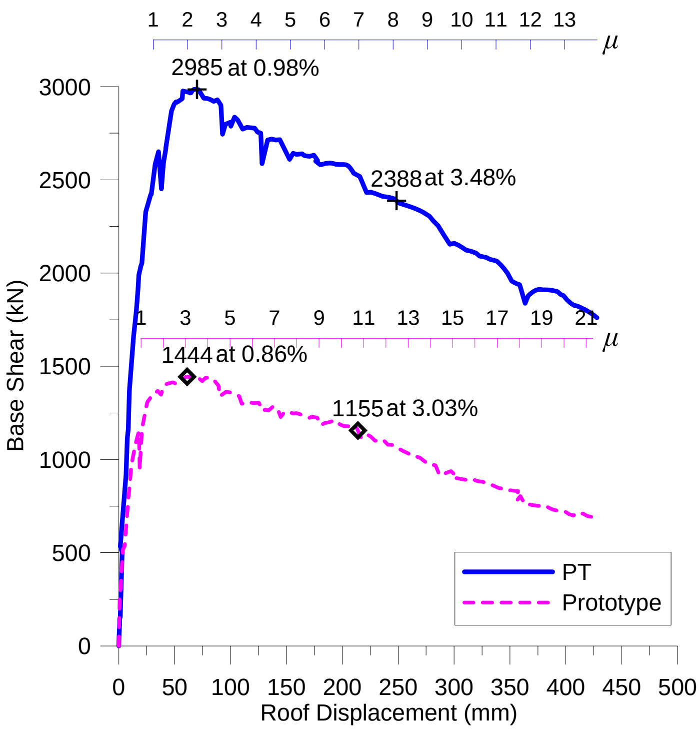
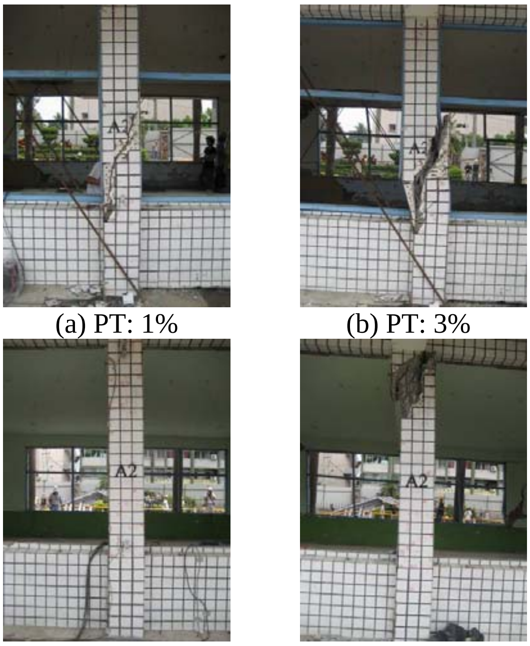


Figure 9 Comparison of pushover curves

(a) PT specimen

(b) Prototype specimen

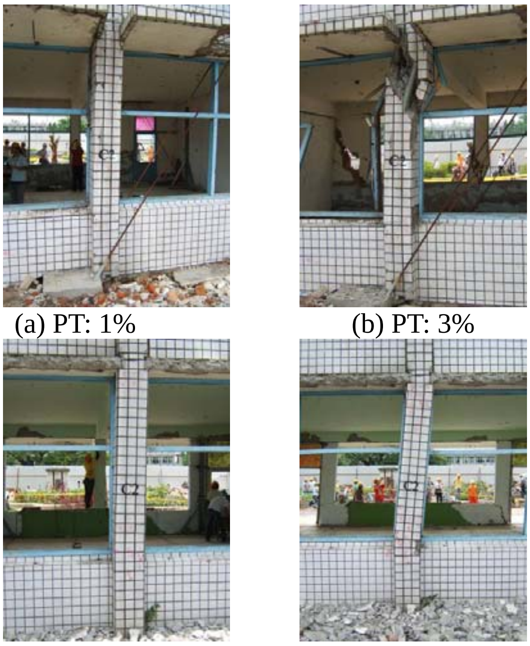
Figure 10 Pictures of the specimens at nearly collapse



(a) PT: 1% (b) PT: 3%

(c) Prototype: 1% (d) Prototype: 3%

Figure 11 Comparison of damage patterns for A2 columns



(a) PT: 1% (b) PT: 3%

(c) Prototype: 1% (d) Prototype: 3%

Figure 12 Comparison of damage patterns for C2 columns

4. CONCLUSION

Two field push over tests, prototype and PT specimens, of school building were presented in this paper. These specimens were pushed by static lateral loads up to collapse. Their structural behavior and failure patterns were recorded. Test results indicated that the PT rods could efficiently enhance both the lateral strength and stiffness of moment resisting space frames However, the ductility of building was decreased due to the increase of column axial load provided by PT.

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