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Retrofit and Repair of Reinforced Concrete Columns with Post-tensioned External Clamps

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ABSTRACT

Reinforced concrete (RC) columns designed and constructed before the 1970s may be earthquake vulnerable due to insufficient shear strength and inadequate drift capacity. Previous earthquakes have evidenced the vulnerability of such columns and the need for their retrofit. Popular retrofit techniques including steel and concrete jacketing, and fiber-reinforced polymer (FRP) wrapping have been shown to increase the shear strength and deformability of RC columns. Nonetheless, those techniques have also reportedly been described as labor-intensive and requiring complicated installation procedures. Ease of installation is also critical when it comes to emergency repairs after strong ground motion. An alternative that is reliable and simple to implement is needed especially for developing countries and mass retrofit of large inventories of structures. The use of post-tensioned external clamps fastened around the column is proposed as a new retrofit and repair technique. The proposed technique is simple to design and implement and does not require specialized workmanship. Its effectiveness is evaluated using experimental data from large-scale RC columns subjected to displacement reversals at increasing drift ratios and constant axial loads. The results of the tests have proven the efficacy of the proposed technique in improving the shear strength and the drift capacity of RC columns.

Keywords: drift capacity, post-tensioned external clamps, RC columns, repair, retrofit

INTRODUCTION

Past earthquakes have highlighted the vulnerability of older reinforced concrete (RC) columns that lack transverse reinforcement [1]–[4]. Transverse reinforcement serves two purposes: it resists shear and provides confinement for the concrete core and longitudinal reinforcement, both of which are crucial to drift capacity. Retrofit methods for RC columns have aimed to replicate these effects by adding confinement through fiber-reinforced polymer (FRP) wraps or steel jackets. While successful in increasing shear strength and drift capacity, these methods have some disadvantages [5]–[7]. The difficulty in estimating the contribution to shear strength from the FRP sheets leads to undesirable costly design [8]. Also, RC columns with FRP reinforcement can fail suddenly after debonding of the FRP sheets [9]. Steel jackets require welding and grouting, and both procedures are involved and cause disruption.

An alternative retrofit approach for RC columns with rectangular cross sections is proposed here. It is based on supplemental confinement by using external post-tensioning clamps. The post-tensioned clamps apply lateral pressure perpendicular to the longitudinal axis of the column. In conventional retrofit systems, confining reinforcement is passive until the expansion of the concrete core that inevitably relates to perceived damage. In the proposed retrofit approach, post-tensioned clamps provide active confinement by confining the concrete with lateral pressure before any damage occurs. Active confinement is expected to help reduce damage, provide resistance to shear, and improve drift capacity. While much research has been done on conventional passive confinement, relatively little has been done on active confinement.

Yamakawa, who worked with small-scale specimens with cross-sectional dimensions not exceeding 250 mm [10], and Saatcioglu, who worked with prestressing strand wound around RC columns [11], have investigated the potential of external post-tensioned reinforcement. Andrawes investigated the use of shape-memory alloys to apply active confinement to RC columns [12], [13]. Skillen tested two RC columns to study the effect of lateral pressure by means of external hoops [14]. Nevertheless, a number of questions remain. In the case of the strands tested by Saatcioglu, the main concern is the ease of installation. The shape-memory alloys investigated by Andrawes are expensive. In relation to the work of Yamakawa, there are questions about how to extrapolate his results with small-scale columns to full-scale columns with sizes more representative of what is common in the field. The specimens studied by Skillen were larger but he tested only two columns, which is not enough in a problem with as much uncertainty as shear has.

After an earthquake, the speed at which repairs are made is extremely important, especially for critical facilities such as hospitals, emergency services centers, and shelters that accommodate large numbers of people. The rapid restoration of the functionality of bridges is also essential after an earthquake, highlighting the significance of fast-to-implement retrofit and repair methods.

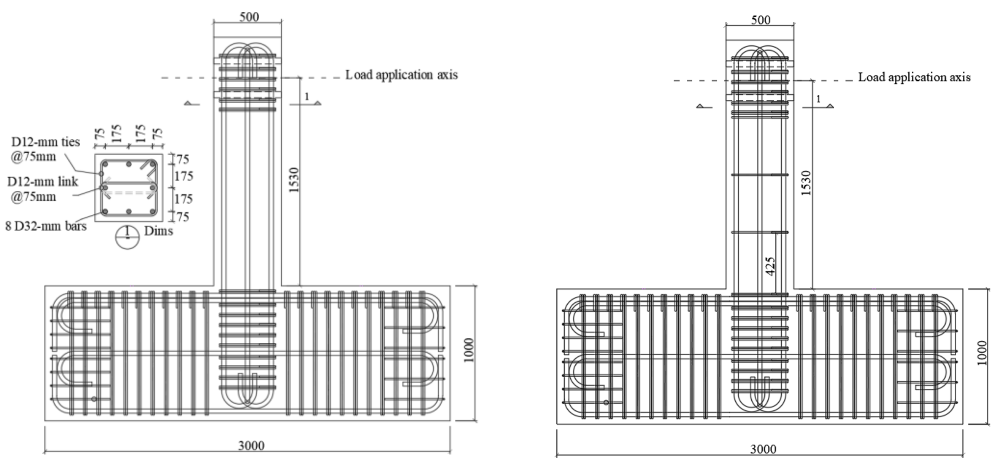
This paper presents test results from RC columns with post-tensioned external clamps from ongoing research at the University of Canterbury (UC), New Zealand and at the National Taiwan University (NTU). Test results are divided in two parts: Part I describes the use of post-tensioned clamps as a retrofit measure, and Part II describes the use of the clamps as a repair measure.

EXPERIMENTAL DATA

Specimens description

The dimensions and details of the specimens tested at UC are shown in Figure 1. Test results from four RC column specimens, labeled C3, C9, C10, and C11, are presented here. The columns had a shear span to effective depth ratio (a/d) of 3.6. The longitudinal reinforcement consisted of eight Grade 500E AS/NZS 4671 deformed steel bars with a diameter of 32 mm. The longitudinal reinforcement ratio was about 2.6%. The mean cylinder compressive strengths measured at test day were between 23 MPa and 30 MPa.

Except from column C11, which had two widely spaced ties, no internal ties were placed along the clear height of the column. This was done for two reasons: 1) to represent the worst-case scenario of the absence or wide spacing of stirrups in older RC columns; 2) to simplify the calculation of the shear that is resisted by the external transverse reinforcement (clamps). The clamps consisted of four corner brackets, made with pairs of 16-mm thick steel angles, and 16-mm high-strength threaded rods Class 8.8 connecting the corner brackets (Figure 2).



a) b)

Figure 1. Details of the specimens tested at UC: a) Columns C3, C9, and C10, b) Column C11

Figure 2. Post-tensioned clamps

The dimensions and details of the specimens tested at NTU are shown in Figure 3. The ratio of the shear span to effective depth (a/d) was 2.2. Results from two large-scale RC columns, labeled SC1 and SC2, are presented here. The longitudinal reinforcement consisted of twelve deformed steel bars with a diameter of 32 mm and measured yield stress of 1250 MPa. The longitudinal reinforcement ratio was about 1.7%. The mean cylinder compressive strengths measured at test day was 21 MPa for SC1 and 23 MPa for SC2. The post-tensioned clamps used were made of pairs of 25-mm thick steel angles and 18-mm high-strength threaded rods. Table 1 summarizes the measured material properties for Columns C3, C9, SC1, and SC2.

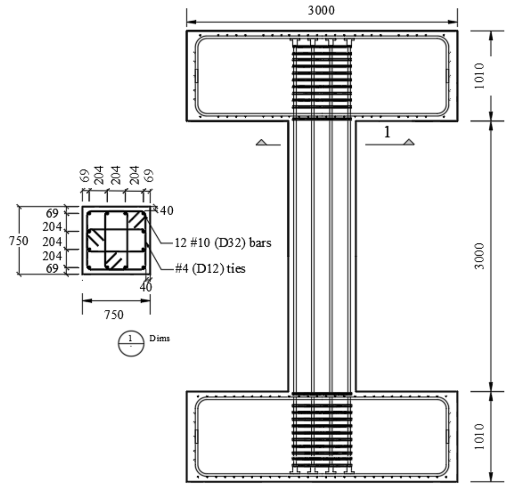


Figure 3. Details of the specimen tested at NTU

Table 1. Measured material properties and key test variables.

Specimen	Use	f'c [MPa]	ρ [%]	f_y [MPa]	r_{tr} [%]	f_{ty} [MPa]	s_{pt} [mm]	r_{pt} [%]	f_{pty} [MPa]	f_{pti} [% f_{pty}]	σ_L [MPa]	A.L.R.
C3	Retrofit	30	2.6	555	-	-	300	0.21	820	10%	0.2	0.15
C9	Retrofit	23	2.6	520	-	-	300	0.21	820	70%	1.1	0.15
SC1	Retrofit	21	1.7	470	-	-	200	0.27	1250	10%	0.3	0.3
SC2	Retrofit	23	1.7	470	-	-	200	0.27	1250	55%	1.8	0.3
C10	Repair	23	2.6	518	-	-	200	0.32	820	70%	1.7	0.15
C11	Repair	23	2.6	518	0.11	550	425	0.15	820	70%	0.8	0.15

f'_c: Concrete cylinder compressive strength; ρ: longitudinal reinforcement ratio; f_y: longitudinal reinforcement yield stress; r_{tr}: transverse reinforcement ratio of internal ties; f_{ty}: internal ties yield stress; s_{pt}: spacing of the post-tensioned transverse reinforcement; r_{pt}: post-tensioned transverse reinforcement ratio; f_{pty}: Post-tensioning transverse reinforcement yield stress; f_{pti}: Initial post-tensioning stress in the clamps; σ_L: lateral confining stress caused by the clamps in the column; A.L.R: Axial Load Ratio.

The design of the post-tensioned transverse reinforcement was based on Equation 1. Equation 1 was first proposed by Richart [15]. The equation calculates the nominal resistance to shear v_n, represented by the sum of the shear contributions from the concrete v_c and the transverse reinforcement v_s. While Equation 1 was initially formulated for conventional ties, Skillen showed that it can be used for post-tensioned transverse reinforcement with satisfactory results [14]. Equation 1 is rewritten as Equation 2.

$$v_n = v_c + v_s \tag{1}$$

$$v_n = v_c + r_{pt} \cdot f_{pty} \tag{2}$$

where r_{pt} is the post-tensioned transverse reinforcement area ratio; and f_{pty} is the yield stress of post-tensioned transverse reinforcement. The post-tensioned transverse reinforcement area ratio r_{pt} is calculated using Equation 3. The equivalent lateral confining stress caused by the clamps in the column σ_L is expressed as transverse reinforcement ratio times the initial prestress in the clamps (Equation 4).

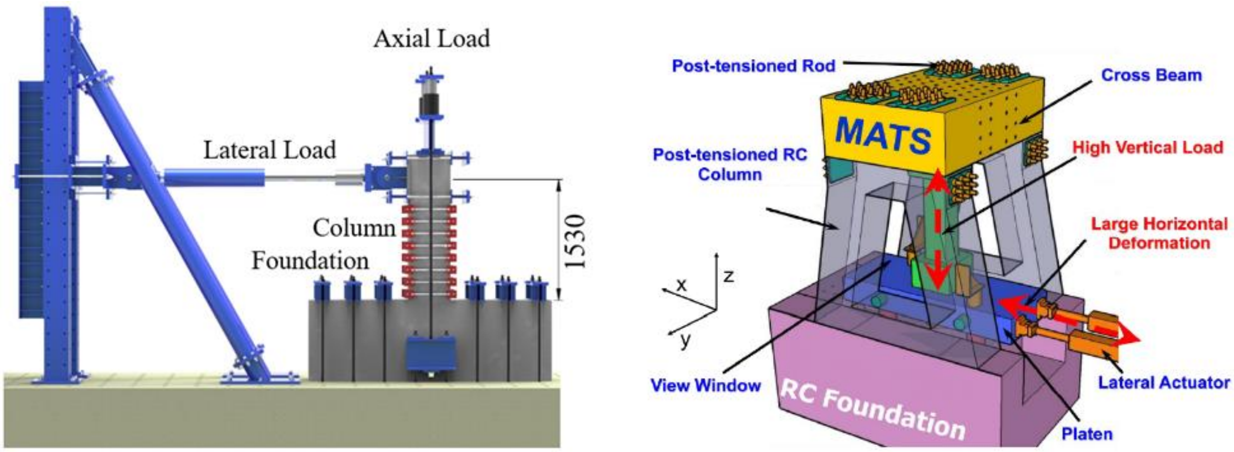
$$r_{pt} = \frac{A_{pt}}{b \cdot s_{pt}} \tag{3}$$

$$\sigma_L = r_{pt} \cdot f_{pti} \tag{4}$$

where A_{pt} is the total area of post-tensioned transverse reinforcement within spacing s_{pt}; b is the width of the column; s_{pt} is the spacing of the post-tensioned transverse reinforcement; σ_L is the lateral confining stress caused by the clamps; f_{pti} is the initial prestress in the clamps.

Tests description

Columns at UC were tested as single-curvature cantilevers under lateral displacement reversals and approximately constant axial load of $0.15A_gf'_c$. Figure 4a shows the test setup at UC. Lateral displacements were increased in 0.5% drift increments with exception of the first two increments from 0 to 0.25% and from 0.25 to 0.5%. The columns were subjected to three lateral cycles at every target drift ratio. Axial loads were applied through vertical post-tensioning rods. Columns at NTU were tested in double curvature under lateral cyclic loading and large axial load of $0.3Agf'_c$. These columns were tested on the Multi Axial Testing System (MATS) at NCREE in Taipei, Taiwan (Figure 4b). The cyclic loading protocol was the same protocol used at UC.



a) b)

Figure 4. Test set-up: a) at UC, b) at NTU

The procedure for installation of clamps on the columns at UC and NTU consisted in placing pairs of steel angles on the four corners of the concrete column. The pairs of angles were then connected to each other using steel threaded rods. Columns C3 and SC1 were tested with clamps with low initial prestress ($\sigma_L \leq 0.3$ MPa) whereas C9 and SC2 had clamps with high initial prestress ($\sigma_L \geq 1.1$ MPa). Clamps on columns with low initial prestress were snug-tightened with a spanner. In contrast, clamps on columns with high initial prestress, were prestressed using a hydraulic bolt tensioner. Gradual increments in force, following a criss-cross tightening sequence, guaranteed even forces in the rods and prevented rotation of the clamps.

Columns C10 and C11, which were used to study the feasibility of the clamps as a repair measure, were loaded in two stages, A and B. Stage A consisted in applying three cycles at 0.25% drift and 0.5% drift, and the first cycle at 1% drift. The intent of Stage A was to cause initial damage represented in flexural and shear cracks. C10 and C11 had no post-tensioned clamps during stage A. After initial damage, the columns were repaired by furnishing them with clamps. Then, stage B of the loading protocol was applied until end of test.

TEST RESULTS

PART I: Specimens with clamps tested as a retrofit measure

Figure 5 shows the hysteretic response of columns C3 ($\sigma_L = 0.2$ MPa) and C9 ($\sigma_L = 1.1$ MPa). The maximum shear force that C3 resisted was 480 kN, and the associated unit shear stress v_{max} was 2.3 MPa. The force associated with the sectional moment capacity V_p, obtained for measured material properties, was 510 KN, and the associated unit plastic shear stress v_p was 2.4 MPa. Therefore, it can be concluded that C3 did not reach flexural yielding of the longitudinal reinforcement. In contrast, C9 resisted a maximum applied lateral force of 540 kN ($v_{max} = 2.5$ MPa) versus a calculated force associated with flexural capacity of 510 kN. C9 not only reached flexural yielding of the longitudinal reinforcement (at a drift ratio close to 1%), but its response post-yielding was more ductile and stable than C3. In this study, column drift capacity was defined as the drift that the column reaches before its lateral-load resistance drops to 80% of the maximum. The drift capacities of C3 and C9 were 3.0% and 4%, respectively.

Figure 6 shows C3 (on the left) and C9 (on the right) at a drift ratio of 3.5%. Fewer and narrower cracks were observed in C9 than in C3. The lateral prestress caused by the clamps precluded the formation of large, inclined shear cracks observed in C3. In addition, the lateral prestress increased the drift capacity from 3% (C3) to 4% (C9), which is a relative increase of 33%. Table 2 summarizes key test results.

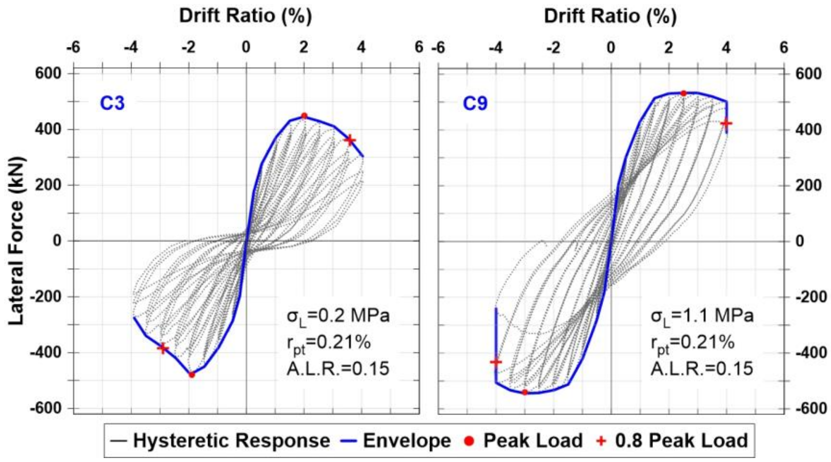


Figure 5. Hysteretic response of Columns C3 and C9

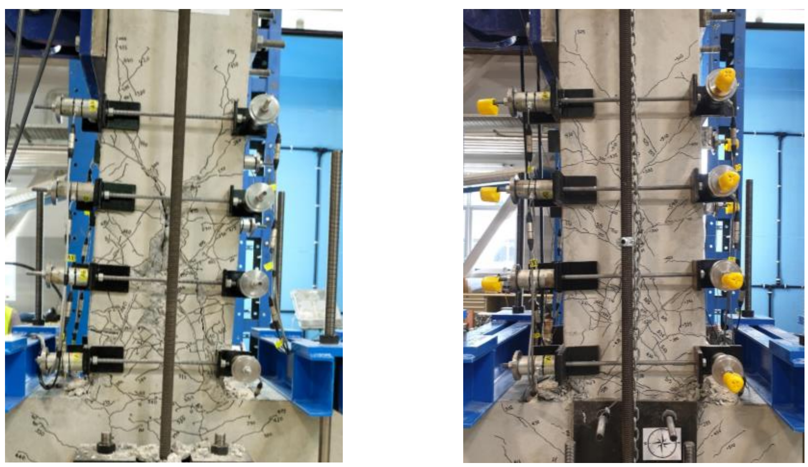


Figure 6. Columns C3 (left) and C9 (right) at a drift ratio of 3.5%

Table 2. Key test results from Columns C3, C9, SC1, and SC2.

Specimen	s_{pt} [mm]	r_{pt} [%]	σ_L [MPa]	$\sigma_L/\sqrt{f'_c}$	V_{max} [kN]	v_{max} [MPa]	D.C. [%]
C3	300	0.21	0.2	0.04	480	2.3	3.0
C9	300	0.21	1.1	0.23	540	2.5	4.0
SC1	200	0.27	0.3	0.06	1495	2.9	2.5
SC2	200	0.27	1.8	0.38	1625	3.2	4.0

s_{pt}: spacing of the post-tensioned transverse reinforcement; r_{pt}: post-tensioned transverse reinforcement ratio; σ_L: lateral confining stress caused by the clamps in the column; V_{max}: maximum measured shear force; v_{max}: maximum shear stress demand corresponding to V_{max}; D.C.: Drift Capacity.

Figure 7 shows the hysteretic response of columns SC1 ($\sigma_L = 0.3$ MPa) and SC2 ($\sigma_L = 1.8$ MPa). The maximum shear force that SC1 resisted was 1495 kN, and the associated unit shear stress v_{max} was 2.9 MPa. The force associated with the sectional moment capacity V_p, obtained for measured material properties, was 1290 KN, and the associated unit plastic shear stress v_p was 2.5 MPa. Column SC2 resisted a maximum applied lateral force of 1625 kN ($v_{max} = 3.2$ MPa). SC1 reached flexural yielding at a drift ratio close to 1%. After flexural yielding, a gradual disintegration of the concrete core diminished the lateral-carrying capacity of SC1. Large criss-crossing inclined cracks and vertical splitting cracks were observed in the column (Figure 8). Column SC2 had a ductile response after flexural yielding, maintaining its full lateral-load-carrying capacity at drift ratios up to 3.0%. Subsequent displacements at drift ratios larger than 3% caused severe concrete crushing and buckling of the longitudinal reinforcement (Figure 8). The drift capacity increased from 2.5% (SC1) to 4% (SC2), which is a relative increase of 60%. Tests results are shown in Table 2.

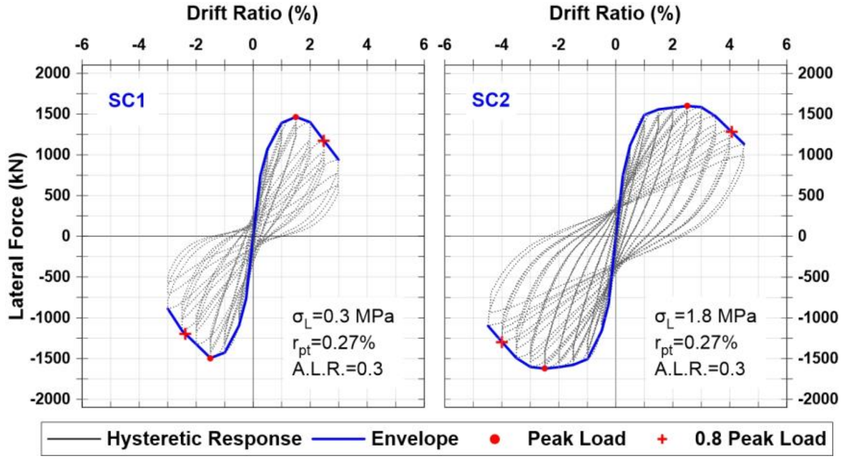


Figure 7. Hysteretic response of Columns SC1 and SC2

Figure 8. Columns SC1 (left) and SC2 (right) at a drift ratio of 2.5%

PART II: Specimens with clamps tested as a repair measure

Figure 9 shows the hysteretic responses of columns C10 and C11, before repair (Stage A), and after repair (Stage B). Columns C10 and C11, were loaded in two stages, A and B. During Stage A, damage in the column was caused in the form of flexural cracks and a critical shear crack (Figure 10). A critical shear crack is defined as a crack that occurs before flexural cracking in the surrounding area. The flexural cracks observed in both C10 and C11 were similar, with thicknesses of up to 0.45mm. The maximum thickness of the critical shear crack in C10 and C11 was measured as 4.0 mm and 1.0 mm, respectively. When the critical shear crack was observed, the lateral loading was stopped, and the axial load was removed. Clamps were then applied and prestressed in less than 4 hours. The columns were tested the next day after reapplying the axial load. Table 3 summarizes key test results.

Table 3. Key test results from Columns C10 and C11.

Specimen	Stage	r_{tr} [%]	s_{pt} [mm]	r_{pt} [%]	σ_L [MPa]	V_{max} [kN]	D.C. [%]
C10	A (No Clamps)	0	-	0	-	+300*	0.65**
	B (With Clamps)	0	200	0.32	1.7	+530/-550	5.3
C11	A (No Clamps)	0.11	-	0	-	+390/-380*	1.0**
	B (With Clamps)	0.11	425	0.32	0.8	+530/-510	3.7

r_{tr}: reinforcement ratio of conventional ties; s_{pt}: spacing of post-tensioned clamps; r_{pt}: reinforcement ratio of post-tensioned clamps; σ_L: lateral confining stress on the column caused by the clamps; V_{max}: maximum measured shear force; D.C.: Drift Capacity.

** Maximum measured force in stage A*

*** Drift ratio associated with the max. measured force in stage A*

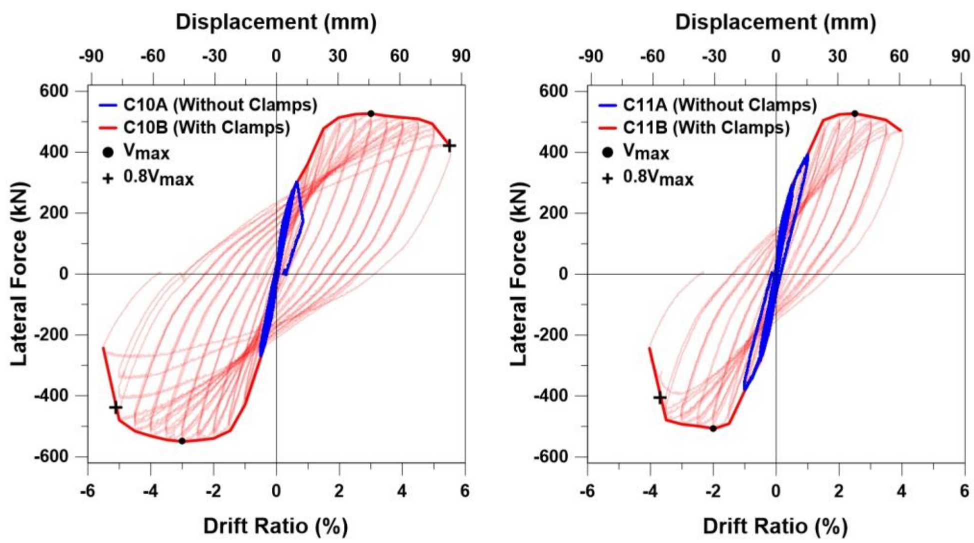
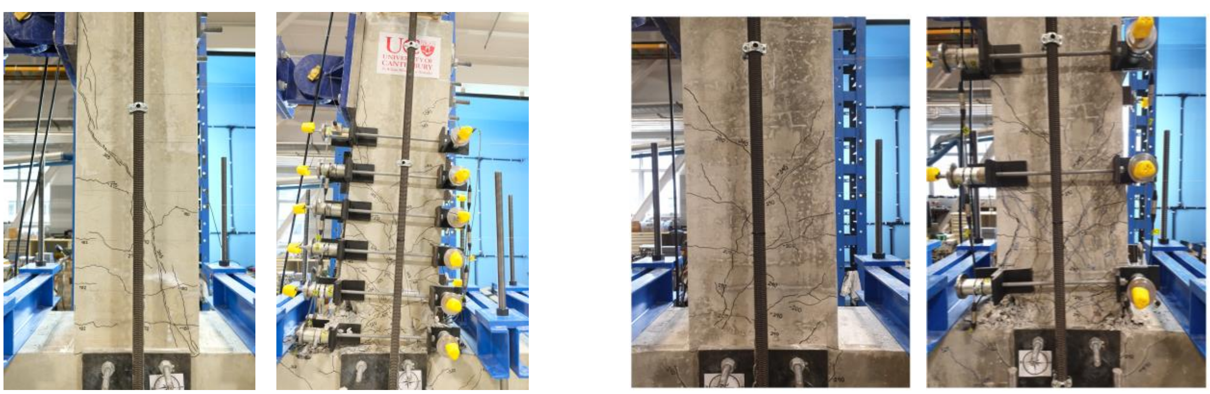


Figure 9. Hysteretic response of Columns C10 and C11

Column C10 developed its first flexural cracks at a lateral load of around 120 kN, and the critical shear crack occurred at a load of 300 kN and a drift ratio of 0.65%. As there were no conventional ties in C10, the measured stress was considered an acceptable estimate of the contribution to shear from the concrete v_c. The critical shear crack developed along the full height of the column (Figure 10a), but external clamps spaced at 200 mm helped reduce its width from 4.0 mm to 0.5 mm during repair. After repair, C10 reached flexural yielding at a drift ratio of approximately 1.3%. The existing critical shear crack did not worsen, but new flexural and flexure-shear cracks were observed (Figure 10b). C10 had a drift capacity of 5.3%.

Column C11 had three conventional ties spaced at 425 mm (r_{tr}=0.11%) with the first tie located 425 mm from the top face of the foundation. Inclined shear cracks were observed at 340 kN and -345 kN (Figure 10c). After being repaired with external clamps (r_{pt} =0.21%), C11 reached flexural yielding at a drift ratio of approximately 1.5%. At a drift ratio around 3%, flexural-shear cracks developed and quickly extended into the compression zones at the base of the column (Figure 10d). Ultimately, C11 failed in its first cycle at a drift ratio of 4%. The drift capacity of C11 was 3.7%.



a) b) c) d)

Figure 10. Column C10: a) Stage A (no clamps), b) Stage B (with clamps).
Column C11: c) Stage A (no clamps), d) Stage B (with clamps)

CONCLUSIONS

- The drift capacity of a column with a lateral confining stress caused by the clamps of at least $0.3\sqrt{f'c}$ in MPa is larger than the drift capacity of a comparable column that has the same level of transverse reinforcement, but with snug-tight clamps
- The lateral clamp stress on columns C10 and C11 helped narrow the critical shear crack that occurred during the initial loading stage. After being repaired, C10 and C11 achieved flexural yielding of the longitudinal reinforcement and failed in a ductile manner, with a drift capacity of 5.3% and 3.7%, respectively.

- The proposed technique of using post-tensioned clamps can be an effective way to retrofit and repair reinforced concrete columns that lack sufficient transverse reinforcement. The clamps are easy to fabricate and install, making them suitable for rapid repair after an earthquake.

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REFERENCES

[1] R. D. Hanson and H. J. Degenkolb, *The Venezuela Earthquake, July 29, 1967*. American Iron and Steel Institute, 1969.

[2] Muguruma, Nishiyama, and Watanabe, “Lessons learned from the Kobe earthquake—A Japanese perspective,” *PCI J.*, 1995.

[3] H. S. Lew and E. V. Leyendecker, *Engineering Aspects of the 1971 San Fernando Earthquake*. U.S. National Bureau of Standards, 1971.

[4] E. Tillotson, “The agadir earthquake of February 29,” *Nature*, vol. 186, no. 4720, pp. 199–199, Apr. 1960.

[5] S. Raza, M. K. I. Khan, S. J. Menegon, H.-H. Tsang, and J. L. Wilson, “Strengthening and Repair of Reinforced Concrete Columns by Jacketing: State-of-the-Art Review,” *Sustain. Sci. Pract. Policy*, vol. 11, no. 11, p. 3208, Jun. 2019.

[6] M. Rodriguez and R. Park, “Seismic load tests on reinforced concrete columns strengthened by jacketing,” *ACI Struct. J.*, vol. 91, no. 2, pp. 150–159, Mar. 1994.

[7] R. S. Aboutaha, M. D. Engelhardt, J. O. Jirsa, and M. E. Kreger, “Rehabilitation of shear critical concrete columns by use of rectangular steel jackets,” *ACI Struct. J.*, vol. 96, no. 1, pp. 68–78, Jan. 1999.

[8] M. Del Zoppo, M. Di Ludovico, A. Balsamo, A. Prota, and G. Manfredi, “FRP for seismic strengthening of shear controlled RC columns: Experience from earthquakes and experimental analysis,” *Composites Part B*, vol. 129, pp. 47–57, Nov. 2017.

[9] R. Kotynia, H. Abdel Baky, K. W. Neale, and U. A. Ebead, “Flexural strengthening of RC beams with externally bonded CFRP systems: Test results and 3D nonlinear FE analysis,” *J. Compos. Constr.*, vol. 12, no. 2, pp. 190–201, Apr. 2008.

[10] T. Yamakawa, S. Kamogawa, and M. Kurashige, “Seismic Performance and Design of RC Columns Retrofitted by PC Bar Prestressing as External Hoops,” *J. of Structural and Construction Engg., AIJ*, vol. 537, pp. 107–113, 2000.

[11] M. Saatcioglu and C. Yalcin, “External prestressing concrete columns for improved seismic shear resistance,” *J. Struct. Eng. (N. Y.)*, vol. 129, no. 8, pp. 1057–1070, Aug. 2003.

[12] B. Andrawes and M. Shin, “Experimental Investigation of Concrete Columns Wrapped with Shape Memory Alloy Spirals,” *Improving the Seismic Performance of Existing Buildings and Other Structures*. 2009.

[13] M. Shin and B. Andrawes, “Parametric Study of RC Bridge Columns Actively Confined with Shape Memory Alloy Spirals under Lateral Cyclic Loading,” *Journal of Bridge Engineering*, vol. 19, no. 10. p. 04014040, 2014.

[14] K. C. Skillen, “The Effects of Transverse Reinforcement on the Strength and Deformability of Reinforced Concrete Elements,” PhD, University of Purdue, 2020.

[15] F. E. Richart, “An Investigation of Web Stresses in Reinforced Concrete Beams,” University of Illinois, 166, 1927.