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Seismic retrofit of reinforced concrete bridge columns using titanium-alloy bars

Reforço sísmico de pilares de betão armado com barras de titânio

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Abstract

This paper presents results of full-scale laboratory tests of a novel solution for retrofitting seismically vulnerable square reinforced concrete columns using externally mounted titanium alloy bars. The use of titanium alloy bars expands the options available to designers for improving the seismic performance of older reinforced concrete structures that do not meet modern design requirements. The experimental results from three column tests are presented in this paper: a control specimen (conventionally reinforced with detailing representative of existing vintage columns typical of US practice prior to 1970), and two specimens retrofitted with externally mounted titanium alloy bars acting as flexural ligaments and confining shell reinforced with a continuous titanium alloy spiral. Test results demonstrated greatly improved ductility and energy dissipation for the retrofitted columns and stable cyclic response. These features can produce more resilient bridges that are able to meet modern-day performance requirements to achieve larger deformation demands without loss of gravity load capacity. The well-defined material properties and excellent environmental durability of the titanium alloy bars make them a viable long-term solution and the retrofit approach allows for visual inspection to observe damage and performance of the component.

Keywords: Lap-splice / Columns / Reinforced concrete / Seismic retrofit / / Titanium-alloy bars

Resumo

Este artigo apresenta resultados de ensaios experimentais executados à escala real de uma solução de reforço sísmico para pilares quadrados de betão armado por encamisamento e utilizando barras de titânio. No artigo apresentam-se resultados experimentais de três pilares: (1) um pilar de referência correspondente ao dimensionamento e pormenorização tipicamente efetuados antes dos anos 70 nos Estados Unidos, sem consideração de ações sísmicas, e dispondo de empalmes realizados ao nível imediatamente acima da sapata; (2) dois pilares reforçados por encamisamento e uso de barras de titânio quer para reforço do comportamento dos pilares à flexão, quer pelo uso de cintas em espiral contínua de barras de titânio.

Os resultados experimentais demonstram que os pilares reforçados apresentam um melhor comportamento quer em termos de ductilidade, quer em termos de capacidade de dissipação de energia. A resposta experimental dos pilares reforçados cumpre com os requisitos de dimensionamento exigidos nos regulamentos atuais.

Palavras-chave: Emplames / Pilares / Betão armado / Reforço sísmico / Barras de titânio

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1 Introduction

According to recent paleoseismic research, the Pacific Northwest region of the United States, including the States of Washington and Oregon, as well as Northern California, has a 15 percent probability of experiencing a M9.0 or greater earthquake originating from the Cascadia Subduction Zone within the next 50 years (Goldfinger *et al.* 2012). However, the Cascadia Subduction Zone was recognized at the end of the last century and bridge design codes were only changed to reflect the hazard in the early 1990s. As a result, bridges built in the region prior to this period were not adequately designed for the current level of expected seismic hazard. Upon review of vintage Oregon bridge designs, it was observed that many bridges built in 1950s and 1960s used square reinforced concrete columns with insufficient flexural and shear reinforcement to resist the expected seismic demands. In addition, the columns have poor detailing of the lap-splices at the base of the column above the footing, creating a bond-slip failure mode that significantly reduces the strength, stiffness, and displacement capacity of the columns (Cairns and Arthur 1979; ElGawady *et al*. 2010; Girard and Bastien 2002; Lukose *et al*. 1982; Melek and Wallace 2004; Paulay 1982). Similar details exist worldwide in regions with long return period seismicity.

This paper presents results of full-scale laboratory tests using a novel solution for retrofitting seismically vulnerable square reinforced concrete columns using externally mounted titanium alloy bars (TiABs). The use of TiABs expands the options available to designers for improving the seismic performance of older reinforced concrete structures that do not meet modern design requirements. TiABs can be both economical and efficient for this purpose and possess highly desirable qualities: impervious to corrosion, low stiffness, high ductility, well-defined material properties with high strength and minimal inelastic strain hardening, and a coefficient of thermal expansion that is closer to concrete than reinforcing steel.

Retrofitting aging or deficient reinforced concrete structures using TiABs has been proposed in the past. Recently, they were applied to bridge girders with inadequate flexural and shear reinforcement in a laboratory setting and subjected to monotonic loading at Oregon State University (Amneus 2014; Barker 2014). This was done using a construction method called Near-Surface Mounting (NSM) in which grooves are saw-cut into the concrete surface and the TiABs are bonded into the grooves with structural epoxy. These studies demonstrated promising results, inspiring the application of TiABs to RC columns for seismic rehabilitation.

Experimental results from three column tests are presented in this paper: a control specimen (conventionally reinforced with detailing representative of vintage column designs that did not consider seismic provisions), and two retrofitted specimens with externally mounted TiABs acting as flexural ligaments and with a continuous TiAB spiral reinforced confining shell. The conventionally reinforced columns had lap splices with a length of 0.90 m (3 feet) above the top of the footing. The two retrofitted specimens included: (1) a strengthened zone is extended 0.60 m (2-feet) above the lap splice, and (2) a strengthened zone extending 0.40 m (1 foot-4 inches) above the lap splice. All retrofitted specimens included externally mounted hooked titanium alloy bars, which were embedded into epoxy-filled drilled holes in the footing and columns. The longitudinal TiABs provide alternative load paths for the flexural demands. Due to inadequate transverse reinforcing in the existing columns, a titanium alloy spiral reinforced shell was formed using no concrete cover over the spiral and filled with ordinary strength concrete. The titanium spiral shell provided confinement of the column and bracing of the titanium alloy ligaments which were unbonded along their length. These combined effects were intended to produce rocking column behaviour to improve ductility and deformation capacity while precisely controlling the column flexural strength to preclude other undesirable failure modes. These features can produce more resilient bridges that are able to meet modern-day performance requirements to achieve larger deformation demands without loss of gravity load capacity.

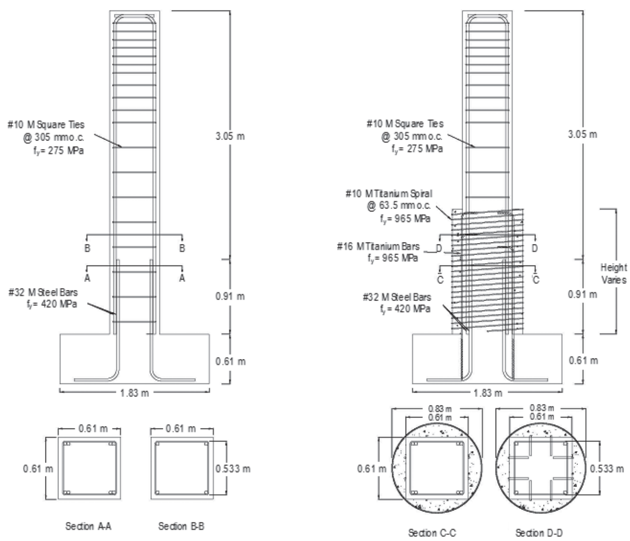
2 Experimental program

An experimental program was developed to evaluate the performance of RC columns retrofitted with TiABs subjected to reversed cyclic lateral loading. The program consisted of three full-scale square RC bridge columns constructed in the laboratory. The dimensions and loading of the tested columns were selected after analysis of geometrical information of the elements in a database of bridges produced for a state transportation agency in the US. The column specimens were tested as cantilever columns.

2.1 Specimen design

2.1.1 As-built specimen design

The specimens are representative of typical square reinforced concrete bridge columns designed according to pre-1970's design standards. The overall geometry of the columns was 3.96 m (13 feet) tall with a 0.61 m × 0.61 m (24 in.× 24 in.) square cross-section. The columns rest on a 1.83 m × 1.83 m (6 ft. x 6 ft.) footing that was 0.61 m (24 in.) tall. The total height of the column specimens was 4.57 m (15 ft.), measured from the top of strong floor to the uppermost point of the column. Total weight, including the column and footing, was 8.44 metric tonnes (18.6 kips). The longitudinal reinforcement in the columns consisted of four 32 mm (#10) reinforcement bars placed at the corners of the columns. Transverse reinforcement consisted of 10 mm (#3) square ties, having 90º hooks and spaced at 300 mm (12 in.) on-center. The lowermost tie was located 150 mm (6 in.) above the top of footing elevation. Tie spacing is decreased in the upper 1.4 m (4.5 ft.) of the column for additional shear reinforcement near the load point of the column. Each of the columns contained lap splices, which consisted of a 90º hooked foundation bar that extended 0.9 meters (3 ft.) out of the footing. The longitudinal column bars were tied to the footing bars using mild steel tie wire at three locations evenly spaced along the length of the splice. The concrete cover was 38 mm (1.5 in.). Nominal yield strength of the footing and column longitudinal reinforcement was 420 MPa (60 ksi). The column transverse reinforcing steel, had a nominal yield strength of 275 MPa (40 ksi). The lower yield stress for the ties corresponds to ASTM A305 Intermediate Grade reinforcing steel used during the era of construction considered and provides similar transverse strength and stiffness as the in-service columns. Grade 40 is not available for large dimeter reinforcing bars and thus higher grade steel was used which required smaller bar diameters. The higher strength but smaller diameter bars provide similar strength and development length but lower dowel resistance and flexural stiffness than larger diameter lower strength bars. The overall specimen geometric details, dimensions, and cross-sections are shown in Figure 1.



(a) (b)

Figure 1 Overall elevation view and cross sections of column specimens, including geometry and reinforcing details: (a) Specimen C1 = as-built configuration and (b) Specimens C2 and C3 = retrofitted configurations

2.1.2 Retrofit specimen design

Two of the columns (specimens C2 and C3) were retrofitted with TiABs. The titanium alloy is designated as Ti 6-4 which has 6% aluminium and 4% vanadium as alloying elements. Each retrofit consisted of eight vertical TiABS (two bars anchored to each column face, spaced at third-points) and a continuous circular spiral TiAB that was wrapped around the lower portion of the column with the ends anchored into the column faces at the top and bottom spiral. Three different lengths of vertical TiABs were used on each column. The three variable length bars allow a transition of longitudinal force around the column perimeter rather than a single location which reduces the concentration of stresses. Each longitudinal TiAB was fabricated with a 190 mm-long (7.5 inches) 90° threaded extension on the hook. The lower end of the bars were fabricated with 508 mm (20 in.) long threads and were placed into holes that were hammer-drilled into the footing to a depth of 560 mm (22 inches). The hooked ends were placed into holes that were hammer-drilled into the column face to a depth of 200 mm (8 in.). The remaining length of the vertical TiABs were smooth where they extend above the footing and below the 90º hooks. The vertical TiAB nominal diameter was 15.9 mm (5/8 inches) and the bend diameter of the

90º hook was 130 mm (5 in.). The diameter of the TiAB spiral was 9.5 mm (3/8 inches) and the wrap diameter is 865 mm (34 inches). The material properties and production method (dead lay) of the TiAB spiral allows the spiral to easily open and be wound around the column without permanent deformation and the coil naturally contacts the corners of the column. The spiral can be wrapped around the column by a single person without exertion. No other concrete preparation is required other than drilling eight (8) holes in the column face and eight (8) holes in the footing to anchor the ends of the vertical TiABs and two (2) holes to anchor the TiAB spiral. Nominal yield strength of all TiABs was 965 MPa (140 ksi) which was used in the design of the specimens.

Two different retrofit heights were considered in this study. The height of the retrofit for specimen C2 was 1.52 m (5 ft.) tall, which extended 0.61 m (2 feet) above the top of the column reinforcing steel lap splice. It consisted of vertical TiABs of lengths 1.93 m (76 inches; 3 each), 1.78 m (70 inches; 3 each), and 1.63 m (64 inches; 2 each). After installation of the vertical TiABs, a TiABs spiral was manually wrapped around the column base and placed at a pitch of 63.5 mm (2.5 inches). The retrofit for specimen C3 was 1.32 m (4.33 feet) tall, which extended 0.41 m (16 inches) above the lap splice. It consisted of vertical TiABs of lengths 1.78 m (70 inches; 3 each), 1.63 m (64 inches; 3 each), and 1.47 m (58 inches; 2 each). The corresponding TiAB spiral was placed at a pitch of 38 mm (1.5 inches) along the upper 405 mm (16 inches) over the region containing the vertical bar hook anchorages and at a pitch of 63.5 mm (2.5 inches) over the remaining height. The volume within the spiral was filled with concrete and intentionally results in no cover over the TiAB spiral. The concrete fill was isolated from the column concrete to prevent it from bonding and becoming composite with the underlying concrete by wrapping the column with plastic sheeting before casting the shell. The shell was cast directly against the footing for specimen C2 but for specimen C3, a 25 mm (1 in.) thick foam insulation board was placed between the shell and the top of the foot to isolate these elements.

2.2 Specimen construction

The specimens were constructed using two concrete placements for the conventional column, C1, and three concrete placements for the retrofitted specimens, C2 and C3. Firstly, the footing was cast and then the column was cast after the footing had cured. For the retrofitted specimens, the TiAB reinforced shell was cast after the column had cured. The footings for columns C1 and C2 were cast on the same date and footing for column C3 was cast at a later date. All columns were cast at different dates, as were each of the retrofit shells. The concrete mix for the footings and columns were designed to provide properties that are consistent with concrete proportions and mechanical properties from the age of construction and considering long-term strength gains over time in service. The concrete mix contained 9.5 mm (0.75 inch) maximum aggregate size and had a 28-day nominal compressive strength of 21 MPa (3 ksi). The use of the high-strength titanium spiral with close pitch over the lap splice region allows the shell concrete to consist of conventional strength material to improve economy and field applicability. The concrete used to fill the TiAB reinforced concrete shell contained 4.75 mm (0.375 in.) maximum aggregate size and had a 28-day nominal compressive strength of 28 MPa (4 ksi). A summary of the day-of-test compression and tensile strengths based on cylinder tests for all elements (footing, column, retrofit) is provided in Table I. The compression test cylinders for Column C1 were defective so the compressive strength was estimated based on the average correlation observed between tensile and compression strengths for specimens C2 and C3 and the tensile strength measured for C1. The controlling failure mode for specimen C1 is highly dependent on the concrete tensile strength which was measured using split cylinder tests.

Tabela I Material properties

Specimen	Column concrete tensile strength (day-of-test)	Column concrete compressive strength (day-of-test)	Retrofit shell concrete compressive strength (day-of-test)
C1	3.6 MPa (520 psi)	30.6 MPa (4440 psi)*	N/A
C2	3.4 MPa (490 psi)	29.0 MPa (4200 psi)	23.6 MPa (3420 psi)
C3	6.0 MPa (870 psi)	25.6 MPa (3700 psi)	34.8 MPa (5050 psi)

*Estimated based on split tensile strength relationship

2.3 Experimental setup and methodology

Following the construction of the footing and column (and retrofit, where applicable), the specimen was anchored to the laboratory strong floor, the horizontal actuator between the strong wall and column load point was attached, and then the axial load system was connected. A hydraulic jack was used to produce axial force in the specimens. The force applied by the jack was measured with a 2225 kN (500 kip) capacity load cell. The jack was placed at top of the column and a 3.2 mm (1/8 in.) thick copper plate was placed between the column and jack to accommodate surface imperfections and enable more uniform pressure distribution to the column. The axial load was distributed to a spreader beam and the force was self-reacted through the footing using two Dywidag bars, one on each side of the column and anchored with spherical nuts to permit rotation of the bars as the column sways under the lateral load. The applied axial load was 900 kN (200 kips), which corresponds to 10% of the nominal axial compressive capacity of the column. Fluctuations in axial load during testing due to column drift were monitored and the jack pressure was adjusted to maintain the compression force on the column during reversed cyclic testing. Lateral force was applied near the column top using a servo-hydraulic controlled actuator in displacement control. The loading protocol consisted of reversed cyclic loading. Each predetermined drift displacement level consisted of three full cycles (six peaks), with each cycle beginning with the column in the neutral position (zero displacement) and then displaced in the positive direction (north, pushing) to the target displacement amplitude, then in the negative direction (south, pulling) to the same displacement amplitudes and then returned to the neutral displacement. The experimental setup for the lateral and axial loading systems are illustrated in Figure 2.

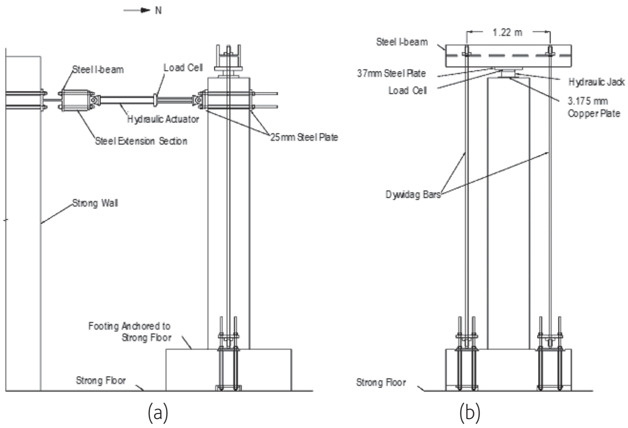


Figure 2 Experimental setup: (a) lateral loading system; (b) axial force loading system

2.4 Instrumentation

Each test specimen was instrumented to quantify the local structural behaviors during testing. The instrumentation plan for the strain gages applied to the column reinforcing steel bars is illustrated in Figure 3. The instrumentation plan for the strain gages applied to the TiABs is illustrated in Figure 4. For all specimens, 22 strain gages were placed on the longitudinal steel reinforcement in the column, including both the footing bars and column bars, twelve (12) strain gages were placed on the transverse reinforcement ties in the column (two strain gages on each tie of the six (6) bottom-most ties). For the retrofitted column specimens, 24 strain gages were placed on the longitudinal TiABs (three (3) on each bar) and an additional ten (10) on the shorter retrofit or twelve (12) on the taller retrofit were placed on the TiAB spiral (two (2) gages each at five (5) or six (6) elevations for the short and tall retrofits, respectively).

Strain gages on the longitudinal reinforcing steel were applied to the west face reinforcement bars, since the column was tested laterally in the north-south direction. This provided data for one bar in tension and one bar in compression due to column bending. On the transverse steel ties, the bottom six ties were instrumented with two strain gages each, one on the west face side and one on either the north or south faces. Three strain gages were applied to each of the vertical TiABs. On the TiAB spiral, strain gages were placed at elevations that corresponded to the instrument locations on the transverse steel ties with two strain gages, one on the west face and one on the north face. All surface strain gages were placed on the bars so as to minimize the influence of local bending induced strains in the bars.

Each specimen was also fitted with external instruments to measure local and overall deformations and applied loads as illustrated in Figure 5. Eleven (11) (column C1) or eight (columns C2 and C3) string potentiometers were used to measure flexural deformations (curvature and rotations). Eight (8) string potentiometers were used to measure shear deformations over the elevation of the column above the footing. A single string potentiometer was used to measure lateral column displacement at the location of the center axis of the horizontal actuator (location of applied load). Five (5) displacement sensors were used to measure footing slip, footing uplift and rotation, and deformation at the column base.

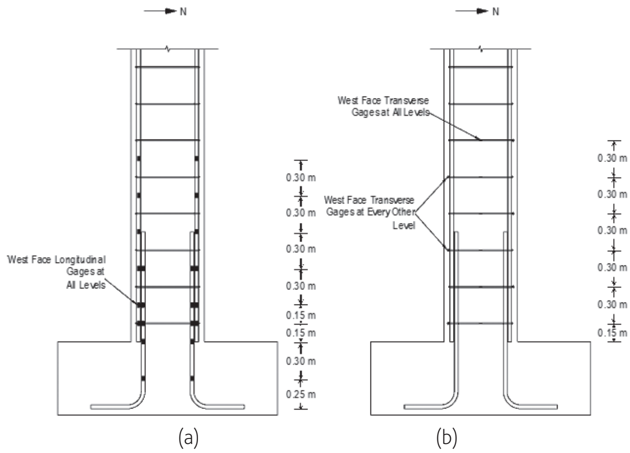


Figure 3 Strain gage locations: (a) vertical reinforcing steel bars and (b) transverse reinforcing steel ties

The string potentiometers were mounted away from the column face to minimize disruption of instrument readings from potential concrete spalling. The sensors were attached to aluminium angles that were supported by and bolted to 9.5 mm (3/8 inch) diameter threaded rods that were drilled and epoxy bonded into the column specimens. The threaded rods were anchored into the column after casting. There were twelve (12) mounting rods on column C1 (one on each north and south face at six different levels) and ten (10) mounting rods on columns C2 and C3 (one on each north and south face at five different elevations). The applied horizontal force was measured using a load cell attached to the actuator and the applied vertical load was measured using a load cell in series with the hydraulic jack at the top of the columns.

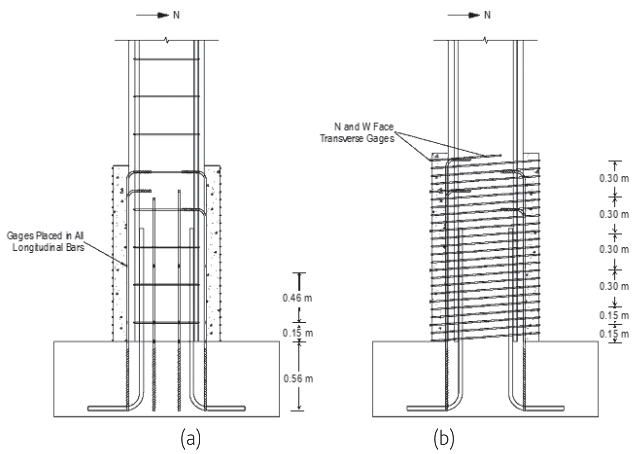


Figure 4 Strain gage locations: (a) vertical TiABs and (b) spiral TiAB

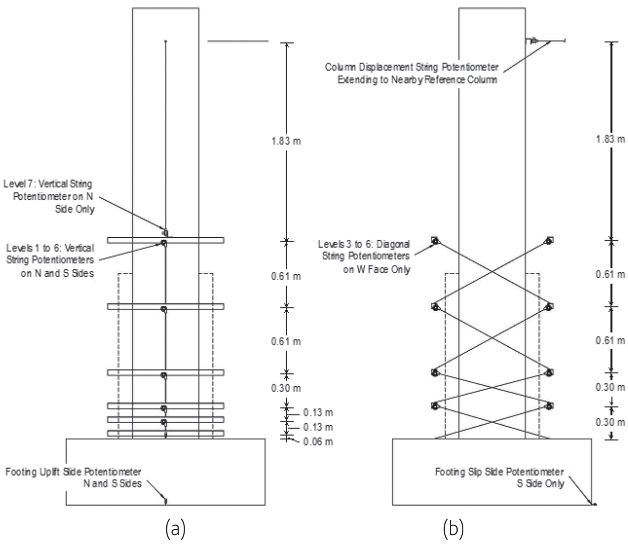


Figure 5 External instrumentation setup for: (a) vertical deformations and (b) horizontal deformations

3 Experimental results

The overall lateral force-column top displacement response for column specimen C1 is shown in Figure 6. The performance of the non-retrofitted specimen was very poor. Failure was observed at a very low drift of approximately 1.7%. This corresponds to a displacement ductility of 1.2. Flexural cracking occurred initially at the base of the column and extended to a height of approximately one-half of the overall column height. Splitting cracks along the lap lengths appeared at low drift magnitudes and eventually diagonal cracking was observed within the lap zone. Once splitting cracks extended along the entire lap length, spalling of the cover concrete began to occur, exposing the lap splice. Once the lapped bars were visible, marks were placed on the exposed bars to observe the relative movement between the bars. The observed relative movement was on the order of 25 mm (1 in.) at the end of the test. Failure of the lap splice and inability to resist flexural demand was the ultimate failure mode of the non-retrofitted specimen. The applied axial load was restorative, precluding buckling of the longitudinal steel reinforcing bars or specimen collapse.

The overall lateral force-column top displacement response from column specimens C2 and C3 are shown in Figures 7 and 8, respectively. Similar performance was observed for both specimens. The columns achieved peak lateral force of approximately 270 kN (60 kips) and maintained the upper shelf strength up to drift limit of 4.2% and 3.5% for C2 and C3, respectively. Failure of the lap splice occurred at these drift limits, where the specimens then demonstrated reduced lateral resistance of approximately one-third of the peak force in both tests. The specimens then followed a lower shelf resistance, maintaining this resistance of approximately 180 kN (40 kips) up to drift level of approximately 8.3% in both specimens. This corresponded to displacement ductility of 4.9 and 5.3 for specimens C2 and C3, respectively. This second resistance shelf was produced by the vertical TiABs combined with a diminishing contribution from the column longitudinal steel bars produced by sliding friction after bond failure. During testing, audible noise was heard from the vertical TiABs caused by localized damage to the anchorage locations in the column and footing. These tended to occur when the specimens were moving through the neutral point (zero displacement) where the vertical TiABs experienced stress-reversal. Localized damage included stable withdrawal of the hooked end from the column face and concrete spall cones forming at the TiAB anchorage to the footing. A definitive failure condition for both specimens C2 and C3 was not reached. The tests were terminated because the stroke capacity of the actuator was achieved and additional drift could not be imposed for the present setup.

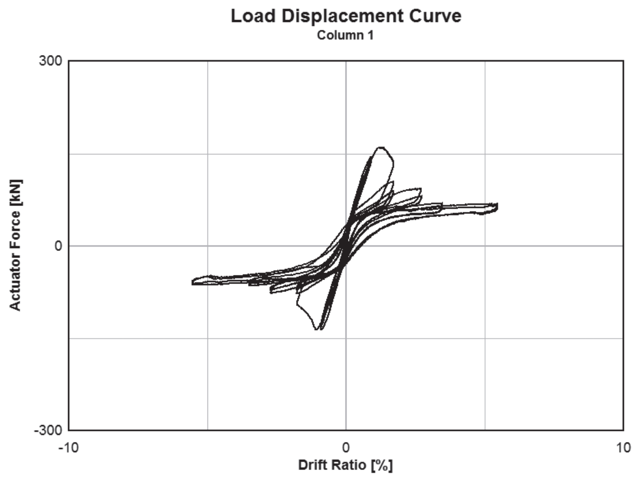


Figure 6 Overall lateral force-column top displacement response for specimen C1

The final condition of the columns showed damage to the concrete in the retrofit shell at the column corners where the shell concrete is only as thick as the TiAB spiral. Spalling of the concrete shell in specimen C2 occurred at fairly low drift levels because the segment of the concrete shell on the compression face was able to bear against on the footing and induced shearing force which cracked the thin shell at the column corner locations. This did not affect the structural performance of the column but to reduce this effect, 25 mm (1 in.) thick insulation foam board was added to specimen C3 to prevent this force transfer mechanism (described in section 2.1). Cracking of the shell in the concrete corners was still observed for specimen C3 but was reduced and was observed to initiate at higher drift levels than did specimen C2.

Backbone response curves are shown in Figure 9 and the energy dissipated at different drift levels is shown in Figure 10 for all specimens. As seen in Fig. 9, the average initial elastic stiffness of the TiAB retrofitted specimens were larger than the control specimen. Both specimens contained the same area of vertical and hoop TiABs, thus the length of the retrofit and bearing of the concrete shell on the footing produced changes in stiffness. The longer length retrofit with shell bearing (specimen C2) increased the average initial elastic stiffness 70% compared to the control specimen while the shorter retrofit without direct shell bearing (specimen C3) increased the stiffness 52%. The influence of the length of the retrofit appears to be more influential on stiffness than bearing of the shell on the footing due to purposely debonding the shell from the concrete column. The retrofitted specimens were able to achieve larger lateral loads about 60% higher at drifts about 4 times that of the unretrofitted specimen. Significantly, as seen in Figure 9, the TiAB retrofitted specimens were able to dissipate more energy with about 110 kN-m at peak force (corresponding to about 3.5% drift) compared to control specimen C1 (only 3 kN-m at peak load occurring at 1.7% drift). The total energy dissipated by both specimens C2 and C3 were similar over the duration of the tests.

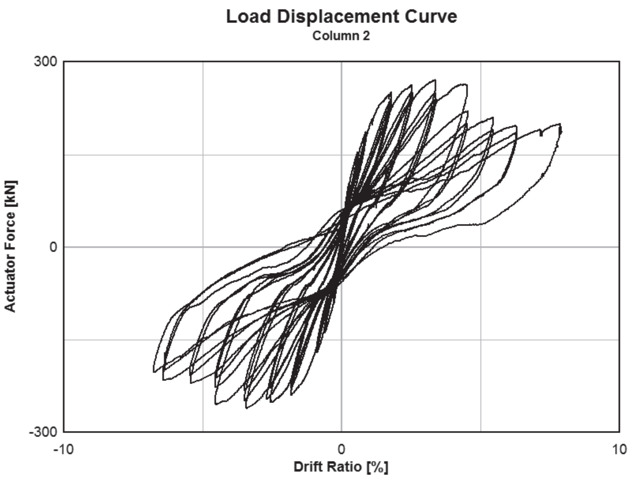


Figure 7 Overall lateral force-column top displacement response for specimen C2

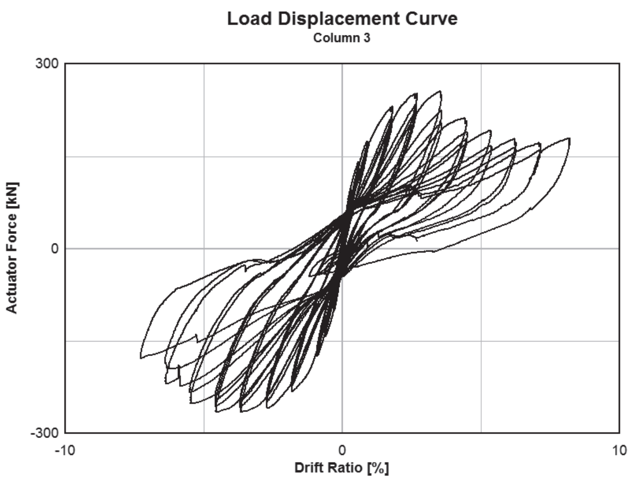


Figure 8 Overall lateral force-column top displacement response for specimen C3

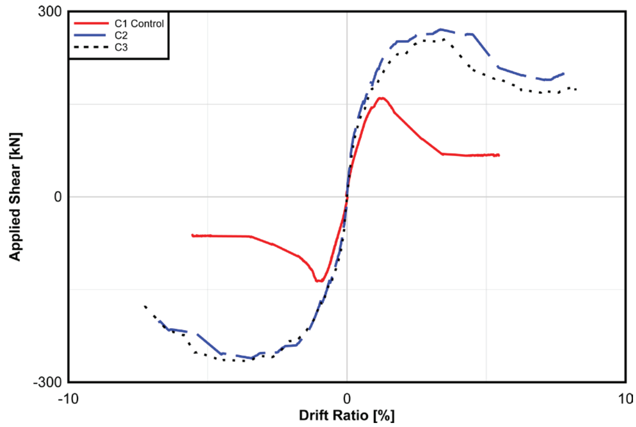


Figure 9 Backbone shear-drift responses for all specimens

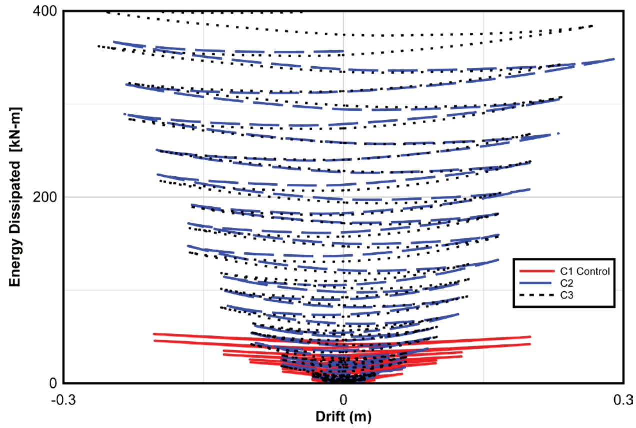


Figure 10 Energy dissipated at drift amplitude for all specimens

4 Conclusions

Three full-size square reinforced concrete columns were tested under axial compression combined with reversed cyclic lateral load. The specimens were constructed to reflect the overall size, proportion, materials, and details of vintage bridge columns according to US practice prior to 1970. The specimens contained widely spaced ties and short lap splices of the longitudinal reinforcing steel in the column above the footing. Performance of the retrofitted columns was greatly improved compared to the baseline non-retrofitted specimen. The non-retrofitted column failed at very low displacement ductility levels due to failure of the longitudinal reinforcing steel lap splice in the column above the footing. The retrofitted columns achieved an initial higher resistance due to the actions of the internal flexural steel and vertical TiABs until the contribution of the flexural steel diminished due to loss of bond at the lap splice. After the lap splice failure, the vertical TiABs continued to provide flexural resistance. The TiAB spiral provided confinement of the concrete column and allowed the lap splice to continue to contribute resistance to higher drifts than the baseline non-retrofitted specimen. It further prevented buckling of the vertical TiABs. The combined effects of a confined core and long unbonded vertical TiABs provided rocking column behavior which did not exhibit an obvious failure point or significant strength degradation (after achieving the lower resistance shelf following failure of the steel lap splice). The test results show great potential for the use of TiABs for retrofitting vintage bridge columns to produce desirable and predictable seismic response that can achieve the required performance levels of modern designs and materials.

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