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RECTANGULAR CONCRETE COLUMNS RETROFITTED BY EXTERNAL PRESTRESSING FOR SEISMIC SHEAR RESISTANCE

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SUMMARY

Experimental investigation was conducted to study the shear mode of failure in rectangular reinforced concrete bridge columns, built before the enactment of modern seismic codes, and to establish the effectiveness of external prestressing as a seismic retrofit methodology. The retrofit technique involves external prestressing of concrete columns in transverse direction to improve their seismic performance in terms of strength and deformability. Typical rectangular columns with substandard shear design and details have been tested with and without seismic retrofitting. The test results indicate that the seismic retrofit technique can be highly effective in controlling diagonal tension cracks caused by shear, thereby improving their shear resistance significantly. Premature shear failure of columns was prevented and the mode of behaviour was changed from shear to flexure. External prestressing also improved the confinement of compression concrete in flexural-compression and diagonal strut regions, thereby improving deformability of concrete. The technique also resulted in substantial improvements in lateral drift and energy dissipation capacities of columns. Column drift capacity was improved up to approximately 4%. Test data is presented along with a design methodology for seismic retrofit of such columns.

INTRODUCTION

Performance of concrete structures during recent earthquakes demonstrated the vulnerability of columns to structural damage. In particular, it was observed that building and bridge columns erected prior to 1970's lacked proper seismic design and detailing practices, leading to complete structural collapse, as shown in Figure 1. The cause of failure of six of the seven bridge structures that failed in the 1994 Northridge earthquake has been attributed to column shear failure [1]. Bridge columns prior to the 1971 San Fernando earthquake were commonly designed to have very little transverse reinforcement, often consisting of No. 3 (9.5 mm diameter) or No.4 (12.7 mm diameter) ties at 12 in. (305mm) on centres,

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regardless of the cross-sectional dimensions or shear force. Column ties had short extensions and 90-degree bends simply overlapping at the ends without sufficient anchorage in concrete against opening beyond the spalling of cover concrete. Crossties were rarely used. This practice resulted in shear deficient columns under lateral shear force reversals caused by strong earthquakes. Current design of similar columns requires approximately eight times the transverse reinforcing steel present in pre-1971 designs [2].

(a) 1995 Kobe Earthquake **(b) 1999 Taiwan Earthquake**

Figure 1 Examples of shear failures in bridge columns

Older bridge piers and building columns suffer primarily from three problems; i) insufficient shear strength, ii) lack of concrete confinement and iii) improper splicing of longitudinal reinforcement. A method of column retrofit by external prestressing has been under investigation at the University of Ottawa, where a large number of circular and square columns were tested [3,4,5]. These columns were investigated for shear, concrete confinement, and reinforcement splice deficiencies. The current phase of research, which is reported in this paper, addresses seismic retrofitting shear deficient rectangular columns.

EXPERIMENTAL RESEARCH

Test Columns

Six full-scale columns, with a 350 mm by 700 mm cross-section, were designed, built and tested under simulated seismic loading. Two different specimen heights were considered to create shear-critical and flexure-critical columns. The specimens represented a segment of a bridge column between the footing and point of inflection. The columns were built in pairs with one column in each pair reflecting as-built conditions while the other column retrofitted for improved seismic resistance. They were tested under a constant axial load and incrementally increasing lateral deformation reversals. The paper focuses on the results of shear dominant column pair, which also lacked proper confinement for inelastic deformability in the flexure mode.

Two companion columns with 1225 mm height were constructed, which had a shear span of 1500 mm after the installation of the top loading beam and the shear span was measured to the point of application of horizontal force. The resulting shear span-to-depth ratio was 2.14, placing the column in a shear dominant range. The reinforcement consisted of twelve #20 (19.5 mm diameter) longitudinal bars and #10 (11.3 mm diameter) hoops with 135º hooks at both ends. The spacing of hoops was 300 mm, starting at 75 mm above the column base. A heavily reinforced footing was prepared for each column. Figure 2 illustrates the geometric details of column specimens tested.

The specimens were cast using ready mixed concrete. Standard concrete cylinders were tested to establish the strength of concrete. The average concrete strengths on the day of column tests were 42 MPa and 37 MPa for unretrofitted and retrofitted columns, respectively. Concrete casting was done in two stages to simulate the actual construction practice in industry. First the footings were cast, with column reinforcement in place. The columns were cast approximately a week after.

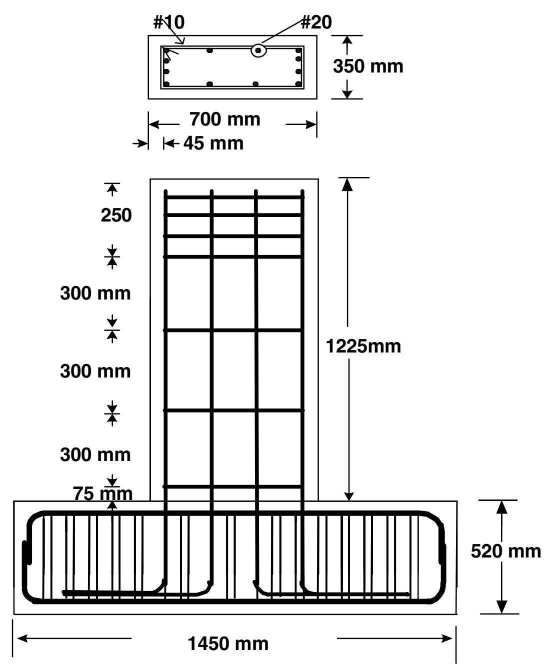

Figure 2 Details of test columns

Tension coupon tests were performed on steel re-bars to establish stress-strain relationships for reinforcing steel bars used as longitudinal and transverse reinforcement. The yield strength for #10 and #20 bars was established as 400 MPa. Seven-wire prestressing strands used for retrofitting the columns had a nominal diameter of 9.53 mm and ultimate strength of 1860 MPa.

Retrofit Technique Employed

One of the columns in the pair was retrofitted by transverse prestressing. This was done by means of 7-wire strands, which were placed at every 150 mm, starting at 75 mm from the column base. Hollow steel sections (HSS 31.8x31.8x6.35mm) were placed on each face with three semi-circular disks welded on them, forming a frame around the column at each strand location. The height and location of semi-circular raiser disks were calculated to make transverse forces as equally distributed as practically possible. A specially manufactured anchor was used to perform prestressing of each strand. The anchor was placed on one of the steel hallow sections, replacing one of the semi-circular disks, while enabling the prestressing of strands. Figure 3 illustrates the details of the anchor, as well as the raiser frames. A small piece of greased steel strip was places around each corner to prevent damage to the corners of columns and to allow the strands slide freely during prestressing. Figure 4 illustrates a typical view of the retrofitted column.

The strands were prestressed to 25% of the ultimate strength of strands, resulting in approximately 465 MPa. This level of prestressing was found to be sufficient in controlling diagonal cracking in earlier tests on circular and square columns [3,4,5]. The lateral pressure generated by prestressing was distributed reasonably uniformly on column faces through the contact points created by the steel hardware.

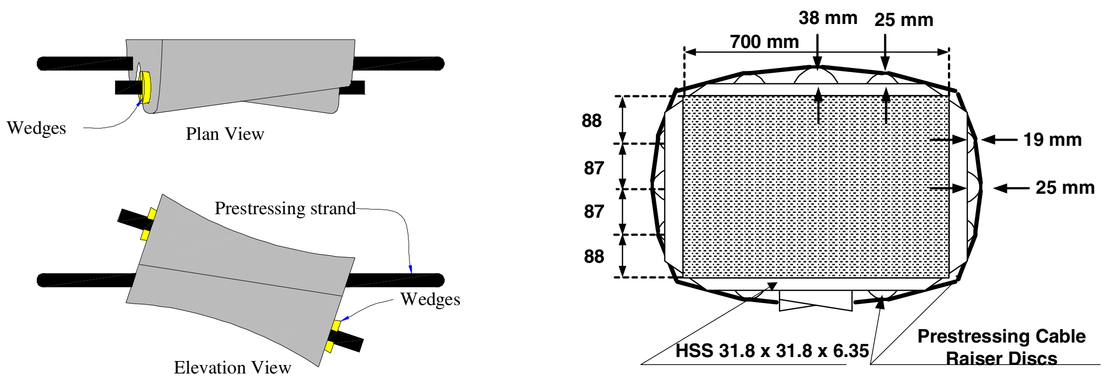

Figure 3 Details of the prestressing anchor and the raiser frames

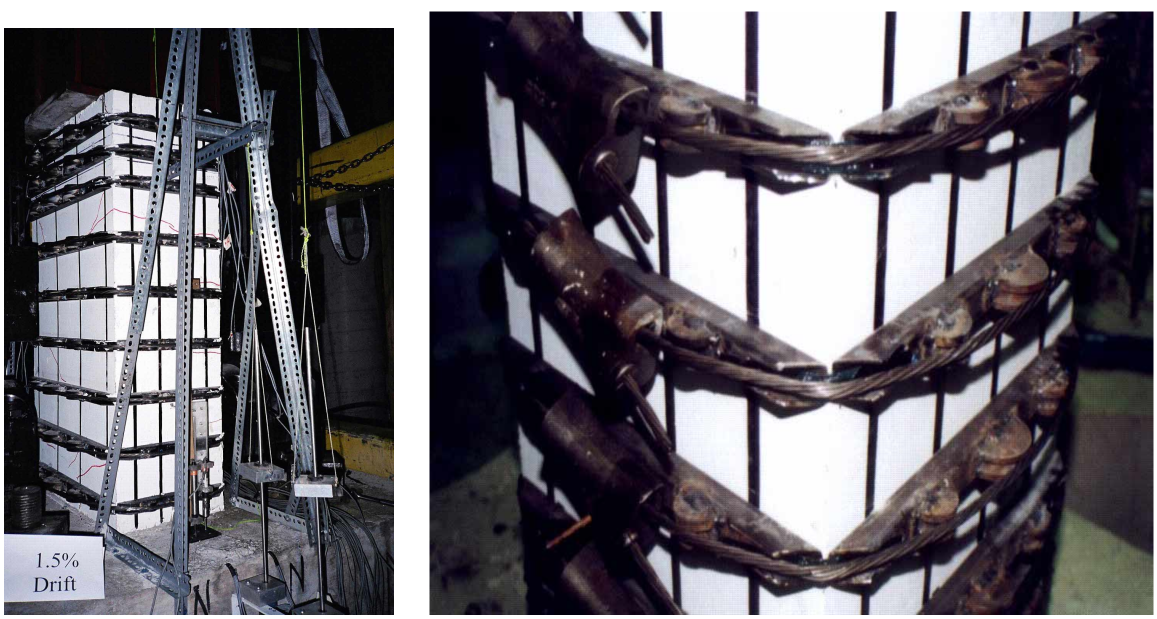

Figure 4 Views of retrofitted column

Test Setup, Instrumentation and Test Procedure

The tests were conducted using three computer servo-controlled MTS hydraulic actuators with 1000 kN capacity. The column footing was fixed to the laboratory strong floor by using four high strength bolts. The lateral deformation reversals were applied by a 1000 kN double-acting servo-hydraulic actuator, connected to a steel loading collar on the top of the column, and operated in deformation controlled mode. Two other actuators were placed vertically on either side of the columns to provide constant axial compression, simulating gravity load. The level of axial compression was equal to 15% of the column concentric capacity, representing a typical gravity load on bridge columns. This corresponded approximately to 693 kN of force per actuator, producing a total force of 1386 kN. Figure 5 illustrates the test set-up.

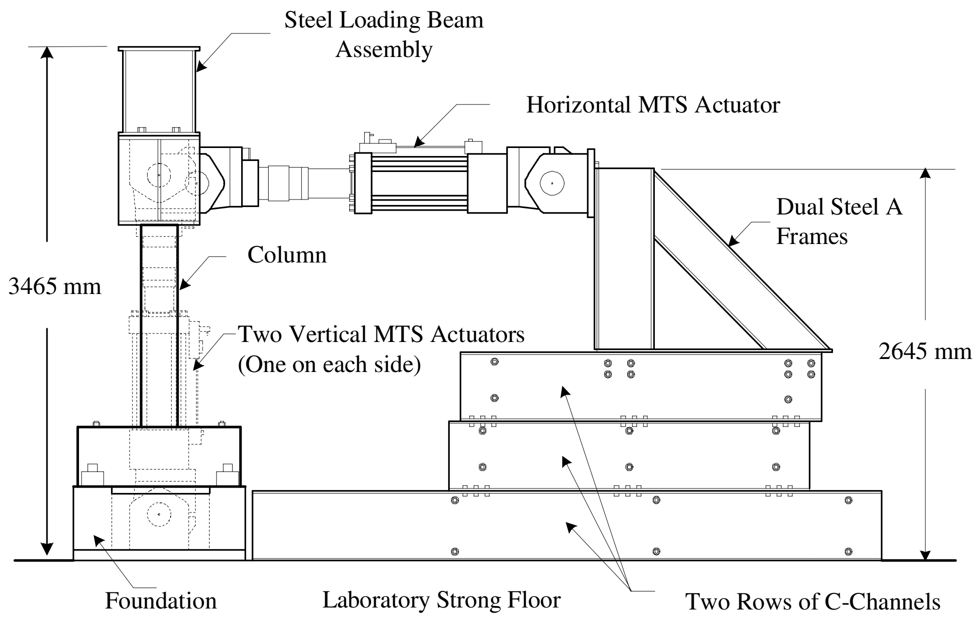

Figure 5 Test setup

A total of 14 electric resistant strain gauges were used to measure the strains in steel reinforcement. Eight strain gauges were placed on longitudinal bars and 6 on the second and third transverse bars. Two gauges were placed on each prestressing strand. Total of four LVDTs (Linear Voltage Displacement Transducer) were used to measure flexural rotation at h and h/2 from footing (where h is the cross-sectional dimension of column parallel to loading, which is 700 mm). One Temposonic LVDT was used to measure the lateral displacement. All the data collected during testing was recorded by a data acquisition system

Each column was first loaded by vertical actuators. The axial load was then kept constant during testing. The horizontal deformations were applied in three cycles at each level. Following three cycles at 0.5% drift level, simulating post-cracking elastic response, the drift ratio was increased incrementally to 1%, 1.5%, 2%, 3%, 4% etc. until a significant drop in load resistance was recorded.

Test results

The performance of each specimen was monitored during testing. Column SC-C was the control column, which reflected non-seismic design practices in existing bridge columns. Flexural cracks were observed after the first cycle of loading at 0.5% lateral drift on the North and South faces of column, which were perpendicular to the direction of lateral loading. Flexural cracks propagated into the sides in the form of inclined shear cracks during the third cycle of 0.5% drift ratio. When the load was increased to 1% lateral drift ratio, the spalling and crushing of concrete was observed near the base within the bottom 75 mm of column. The previous shear cracks propagated further to column sides, as new cracks have formed. The column reached its peak resistance during the first cycle of 1.5% drift ratio, and further cycling resulted in strength decay. Moment resistance in the direction of first load excursion (loading towards North) dropped by approximately 25% at the end of the third cycle. This level of strength drop was considered to be significant enough to label the drift capacity to be 1.0 %, since the columns could not sustain three cycles at 1.5% drift level without experiencing less than 20% strength decay. During subsequent deformation cycles the diagonal cracks as well as flexural cracks within the lower 350 mm segment widened. The crack at column-footing interface also widened as the longitudinal bars continued extending within the footing. Figure 6 illustrates the column at the end of test. Experimentally recorded moment-lateral drift hysteretic relationship for the column is shown in Figure 7. The moment values plotted were computed from recorded horizontal forces and the horizontal and vertical components of axial load, including the P-Δ moment. It is clear from the hysteretic relationship shown in Figure 7 that the column experienced severe strength degradation immediately after the cycles at 1% lateral drift. The moment resistance dropped below 50% of maximum recorded during 2% drift cycles and the test was discontinued. The hysteresis loops show some pinching, signifying shear dominant response. The failure was triggered insufficient transverse reinforcement against diagonal tension.

Figure 6 SC-C at 2% drift

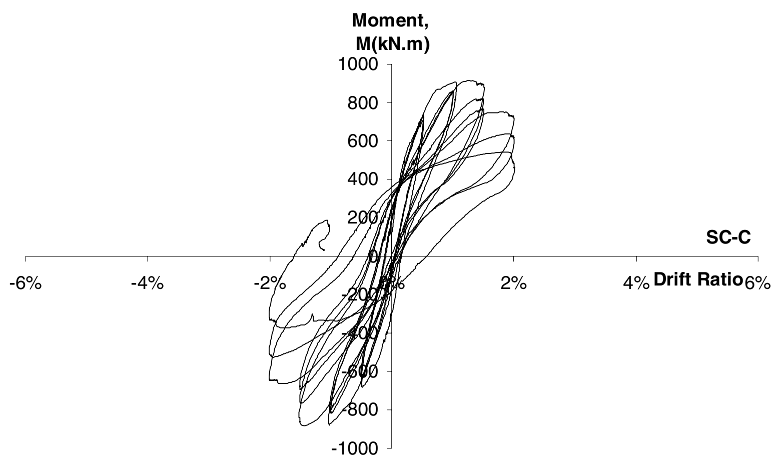

Figure 7 Hysteretic moment-drift relationship for SC-C

Column SC-R was the companion column to SC-C, with identical geometric and material properties. However, this column was retrofitted by external prestressing in transverse direction, as explained earlier. The same loading scheme that was used for the previous column was also used for this column. The first flexural hairline crack was observed during the third cycle at 1%. The number of flexural cracks increased during subsequent cycles of the same deformation level. A crack formed at column-footin interface on the south side during the third cycle at 1% drift. The first set of shear cracks were observed on both side faces, during 2nd cycle of 1.5% drift when the lateral force was at 590 kN. The first cycle at 2% drift resulted in additional hairline flexural cracks on the North and South faces. The crack at column-footing interface widened and became longer during the 2nd cycle at 2% drift. Some crushing of cover concrete was observed near the base during the third cycle of the same deformation level. Increased diagonal shear cracks were observed during 3% drift cycles with widening of one of the cracks. However, the crack was well under control due to external prestressing. The column maintained its strength until 4% lateral drift and experienced some strength decay beyond this deformation level. Figure 8 illustrates the column at the end of 3% drift cycles, with virtually no damage. The hysteretic moment-lateral drift relationship is plotted in Figure 9, showing stable loops until after 4% lateral drift.

When the moment-deformation hysteretic relationships are compared, it can be seen that Column SC-R, with seismic retrofitting was able to sustain about 4 times the lateral drift sustained by the control column SC-C. This is attributed to the improvement in shear resistance. Transverse prestressing counteracted diagonal tension and improved concrete shear resistance by delaying the formation of diagonal cracks. Beyond diagonal cracking, the prestressing continued to control cracking, improving aggregate interlock at higher inelastic deformations and providing additional transverse shear reinforcement. This resulted in a significant overall increase in column shear resistance, changing the mode of mode from shear to flexure. The column then behaved in ductile flexure mode, exhibiting improved deformability. Further improvement in deformability was achieved due to the confinement of compression concrete, which delayed concrete crushing in flexure.

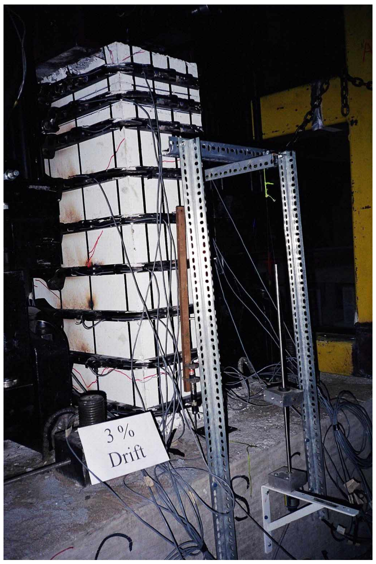

Figure 8 SC-R at 3% drift

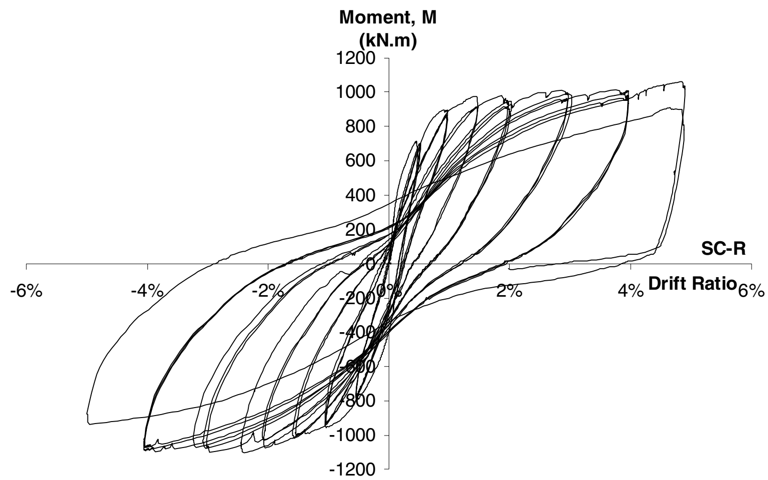

Figure 9 Hysteretic moment-drift relationship for SC-R

RETROFIT DESIGN FOR SHEAR

Transverse prestressing improves both the concrete and reinforcement contributions to shear. The required level of enhancement for each component depends on the performance level expected from the structure. If the damage after a major earthquake is to be eliminated completely to a point where no repair due to shear damage is required, the design criterion should be to prove sufficiently high strength enhancement against diagonal tension so that the widths of diagonal cracks are controlled and the deterioration of concrete under stress reversals is not permitted. Transverse prestressing to eliminate diagonal cracking completely may not be necessary, as some tension in concrete can be tolerated for an acceptable level of performance. The column tests reported in the current investigation, as well as those tested earlier [3] indicate that the damage on concrete could be controlled if the transverse strain is limited to 0.15% to 0.20%, depending on the lateral drift demand. In most cases 0.20% transverse strain provide acceptable level of crack control up to approximately 4% lateral drift. At this strain level, both concrete and internal reinforcement could be relied on for shear resistance. Requirements that satisfy this level of performance were developed by Saatcioglu and Yalcin [3] and are given below.

$$V_e \leq \phi V_c + \phi V_s + \phi V_p \tag{1}$$

$$V_c = 0.2\sqrt{f'_c}bd \tag{2}$$

$$V_s = A_v f_s \frac{d}{s} \tag{3}$$

$$f_s = 0.002E_s \ \leq f_y \tag{4}$$

$$V_p = 2A_{ps}(f_{pi} + 0.002E_p)\frac{h}{s_p} \tag{5}$$

$$A_{ps} = \left(\frac{\frac{V_e}{\phi} - 0.2\sqrt{f'_c}bd - A_v f_s \frac{d}{s}}{2(f_{pi} + 0.002E_p)} \right) \frac{s_p}{h} \tag{6}$$

$$50MPa < f_{pi} \leq 0.5 f_{pu} \tag{7}$$

$$s_p \leq \frac{h}{4} \tag{8}$$

The above requirements are intended to maintain the integrity of concrete during seismic response while providing reserve shear capacity beyond the development of effective prestress, ($f_{pe} = f_{pi} + 0.002E_p$). The contribution of axial compression to concrete shear resistance, V_c, is neglected conservatively. Where the factored axial force results in net tension, V_c may not be relied on, and the concrete contribution term in Eq. 2 should be dropped.

If the design performance level calls for the survival of bridge columns after the earthquake, while sustaining repairable damage, the requirements can be relaxed. In this case, the seismic resistance is provided essentially by reinforcement. This reinforcement consists of the existing ties inside the column and the external strands provided for retrofitting. Concrete contribution is conservatively neglected due to the possibility of significant damage at high levels of transverse strain. The following are the design requirements for collapse prevention performance level.

$$V_e \leq \phi V_s + \phi V_p \tag{9}$$

$$V_s = A_v f_y \frac{d}{s} \tag{10}$$

$$V_{ps} = 2A_{ps} f_{py} \frac{h}{s_p} \tag{11}$$

$$A_{ps} = \left(\frac{V_e - A_v f_y \frac{d}{s}}{2f_{py}} \right) \frac{s_p}{h} \tag{12}$$

$$s_p \leq \frac{h}{4} \tag{13}$$

For the above performance level, the prestressing hoops are required to be stressed to a minimum of 50 MPa to remain at least snug-tight around the column. While further stressing does not improve the ultimate shear capacity, it does control damage to concrete, helping the column to approach the damage control performance level.

PROTECTION AGAINST CORROSION AND OTHER EXTERNAL ATTACKS

Once the retrofit design is completed and applied in the field, the prestressing hardware must be protected against weathering (corrosion) and other external effects. While there may be a variety of techniques to cover the steel hardware, shotcreting is one of the most practical approaches for bridge columns. Fiber-reinforced concrete jacket is another technique that has been tried and tested in laboratory on a circular column [3]. The column was subjected to simulated seismic loading and the jacket was able to maintain its integrity until after 3% lateral drift ratio. It suffered only hairline cracks and survived up to 7% lateral drift with some spalling of the jacket cover near the column base. Figure 10 illustrates the performance of concrete jacket over prestressing hardware at 1% and 7% lateral drift. Another effective technique for protection of prestressing hardware in interior applications may be fire-resistant drywall.

(a) At 1% drift **(b) At 7% drift**

Figure 10 A circular column with protective concrete jacket

CONCLUSIONS

The following conclusions can be drawn from the experimental investigation reported in this paper:

- Reinforced concrete bridge columns, designed prior to the enactment of modern seismic design codes, tend to be vulnerable to brittle shear failures. This vulnerability is attributed to the practice of using insufficient amount of column ties with large spacings. A representative rectangular column tested in the current investigation performed poorly under simulated seismic loading, exhibiting limited lateral drift capacity of 1.0 %.
- Seismic retrofitting concrete columns by transverse prestressing improves shear capacity, resulting in significant improvements in column deformability. The lateral drift capacity of the rectangular bridge column tested in the current investigation was improved from 1% to 4% through external prestressing.
- Transfer prestressing delays the formation of diagonal tension cracks, improving concrete shear resistance. It further enhances column shear capacity by providing additional transverse reinforcement, changing the mode of failure from brittle shear to ductile flexural failure. Seismic

retrofitting shear deficient concrete column by external transverse prestressing proves to be an effective technique that is also cost-effective.

- The performance-based design approach outlined in the paper can be used to retrofit existing shear deficient columns.

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NOTATIONS

A_v : Total area of transverse shear reinforcement within spacing, s, in the direction of shear force (mm^2).
A_{ps} : Area of strand used to prestress column in the transverse direction (mm^2).
b : Column cross-sectional dimension perpendicular to shear force (mm).
d : Distance from the extreme compression fiber to the centroid of longitudinal tension reinforcement in mm (for circular sections the centroid of longitudinal reinforcement in the opposite half of the member shall be used).
d_p : Diameter of prestressing strands in mm.
D_b : Bend diameter in mm
E_p : Modulus of elasticity of prestressing steel, E_p = 200000 MPa.
E_s : Modulus of elasticity of transverse steel for shear, E_s = 200000 MPa.
f'_c : Concrete compressive strength in column, in MPa.
f_{pe} : Effective prestress in MPa.
f_{pi} : Initial prestress in MPa.
f_{py} : Yield strength of prestressing strand, in MPa.
f_{pu} : Ultimate strength of prestressing strand, in MPa.
f_s : Stress in transverse shear reinforcement, MPa.
f_y : Yield strength of transverse shear reinforcement, in MPa.
H : Column cross-sectional dimension parallel to shear force, or diameter of circular section, in mm.
S : Spacing of transvers shear reinforcement, in mm.
s_p : Spacing of external prestressing hoops, in mm.
V_c : Shear force resistance provided by tensile stresses in the concrete, newtons.
V_e : Maximum seismic shear force that the column may be subjected to during earthquake, determined as the larger of the factored shear force and the shear force associated with the formation of flexural plastic hinges in the column, newtons.
V_n : Nominal shear capacity of column, newtons.
V_p : Shear strength enhancement provided by external prestressing in newtons.
V_{pc} : Shear strength enhancement in concrete, introduced by external prestressing, newtons.