SEISMIC PERFORMANCE OF CONCRETE SPECIAL BOUNDARY ELEMENT

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ABSTRACT

In February of 2010, a magnitude 8.8 earthquake struck south central Chile. Concrete construction dominates the building industry in Chile even for multi-story to high rise structures. Of these multi-story reinforced concrete structures, fifty were severely damaged, four of which were partially or totally collapsed. Chilean concrete design uses shear walls as the main lateral force resisting system, and they were a critical part in the failure of these buildings. However, the failure mechanism stumped engineers. One proposed theory suggested that because of commonly used T-shaped shear walls, the boundary elements of the web section were subject to both tension and compression with reversals of the ground motion. The intention of this research was to provide evidence of the effect tensioning a boundary element has on buckling failure. Two specimens were tested representing concrete boundary elements. One specimen was subjected to tension up to 4% strain then compressed until failure, and the other was only put in compression. Comparing the compression capacities of the two specimens, a pre-tensioned boundary element can take less than a third of the load a compression-only boundary element can handle. Therefore, tensioning a boundary element has devastating effects on the compression strength.

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1 INTRODUCTION

1.1 Background

1.1.1 Description of earthquake

In February of 2010, a magnitude 8.8 earthquake struck south central Chile. 521 deaths occurred out of a population of 3 million. Overall most buildings performed successfully. Concrete construction dominates the building industry in Chile even for multi-story to high rise structures. Of these multi-story reinforced concrete structures, fifty were severely damaged, four of which were partially or totally collapsed. Figure 1 shows typical failure of shear walls after the February earthquake.

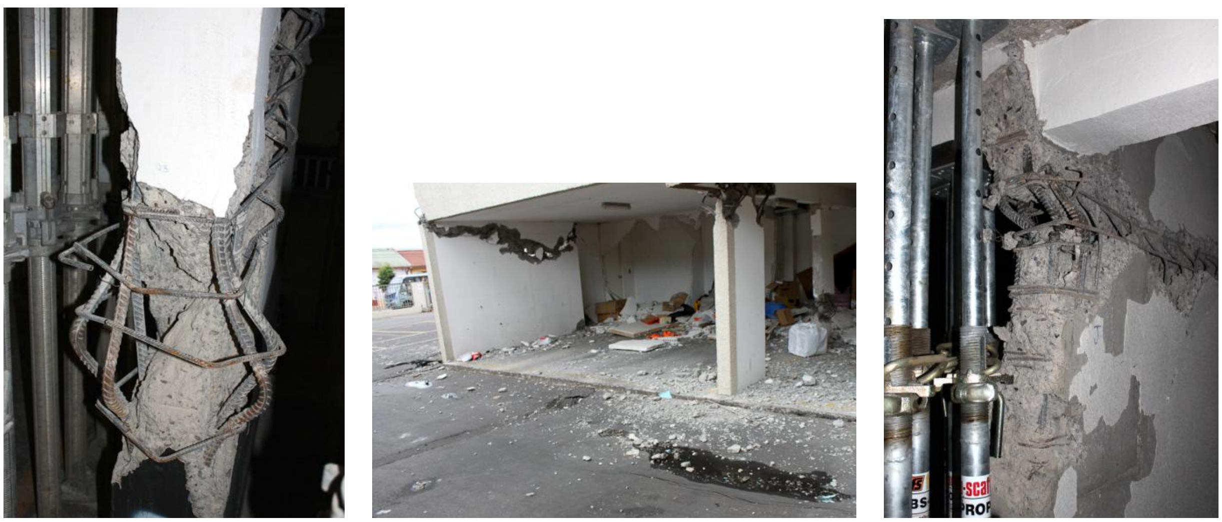

Figure 1: Typical failure seen after February earthquake (credit: Moehle)

1.1.2 Research motivation

Chilean concrete design employs shear walls as the main lateral force resisting system, and they were a critical part of the failure of these buildings. However, the failure mechanism stumped engineers. One proposed theory suggested that because of commonly used T-shaped shear walls, the boundary elements of the web section were subject to both tension and compression with reversals of the ground motion. Damage observed after earthquakes are relevant to the US because they have adopted the same building codes with minor changes and loosely enforced.

1.2 Objective and Scope

The intention of this research was to provide evidence of the effect tensioning a boundary element has on buckling failure and load carrying capacity. In order to demonstrate this, two specimens were tested representing concrete boundary elements. Specimen 1 was tensioned to 4% strain then compressed until failure, and Specimen 2 was only compressed.

2 LITERATURE REVIEW

An important step on the path to improving the ACI seismic design code is travelling to post-earthquake regions and studying the structural response of affected buildings. The Chilean earthquake provided an opportunity to test multi-story concrete structures under considerable earthquake loading. More than 50 concrete structures had structural damage and 4 partially or totally collapse [EERI Special Earthquake Report, 2010]. Chile's earthquake is interesting because they have adopted parts of UBC-97 and ACI 318-95; however, after the successful building performance in the March 1985 earthquake closely spaced transverse reinforcement was not required in the vertical boundary elements [Wallace, 1992]. Despite this, some buildings had been designed to ACI specifications, and none of them showed damage. In the structural walls that had failed, the exact failure mechanism stumped engineers. These shear walls were most commonly the stem of T-shaped structural walls. These walls are capable of much less drift capacity than if they were two independent rectangular walls [Wallace, 1992]. One theory developed after reviewing the damage claimed that due to the common structural wall layout of corridor structural walls centered along the longitudinal axis with perpendicular walls framing into them caused the boundary elements to be put into extreme tension followed by extreme compression. Although failure wasn't seen in walls detailed similarly to the special boundary element section of the ACI code, no research has been done to test the effectiveness of the existing code. It is unknown whether the failure was caused by pre loading in tension or less transverse reinforcement. No research has tested both issues simultaneously.

ACI 318-08 approaches special boundary elements with the assumption that inelastic response of the wall is dominated by flexure at a critical, yielding section. The design equations follow from a displacement-based approach. The approach assumes that special boundary elements are required to confine the concrete where the strain at the extreme compression fiber of the wall exceeds a critical value when the wall is displaced to the design displacement [ACI Committee 318, 2008]. The failure behavior of the shear walls in Chile suggest a revision might be needed to increase the frequency of which shear walls are required to contain special boundary elements. Work by Chai and Elayer studied the lateral stability of a boundary element put in tension and then compression. This study confirmed the effect of the reversed axial cyclic loading, but only focused on tightly confined sections. The specimen design closely resembled that of a special boundary element. Wide cracks in the concrete developed at each transverse hoop. In addition, the wide cracks caused a critical condition where excessive out-of-plane displacement was more prone to occur [Chai, 1999]. This study demonstrated the response of ductile boundary elements, which resulted in global displacement instead of buckling or fracture of the longitudinal bars.

In addition to the creation of a critical condition, buckling failure is controlled by cyclic loading of reinforcing steel bars. Cyclic loading affects the yield plateau, growth of the curvature in cyclic stress-strain response, Bauschinger effect, low-cycle fatigue, stress relaxation, and strength degradation as shown by Heo, Zhang, Kunnath, and Xiao (2009). All these features are relevant to the process of strength degradation and softening resulting from accumulated plastic deformation [Heo et al., 2009]. This plastic deformation was found at the base of cantilever shear walls in a study by Paulay and Priestley. They concluded that inelastic buckling was governed more by wall length and previously experienced tensile strain instead of unsupported height and compressive strain [Paulay, 1993].

3 TEST PROGRAM

3.1 Test Objective

3.1.1 Overview

Two specimens will be tested: specimen 1 in tension and compression, and specimen 2 in compression only. They were detailed identically as special boundary elements per ACI 318-08 section 21.9.6.4. The intention is to determine the effect pre-tensioning the boundary element has on buckling.

3.1.2 Tension and compression value determination

The strain by which to tension the specimen simulates the maximum strain the boundary element might see of a typical Chilean T-shaped shear wall. It was determined by multiplying the code limit for the ratio of deflection to height of the wall by an estimation of the ratio of wall length to plastic hinge length:

$$\varepsilon_s = \left(\frac{\delta}{h_w}\right) * (\frac{l_w}{l_p})$$

$$\varepsilon_s = 0.02 * 2 = 0.04$$

No compression value was determined because the specimen was taken to failure.

3.1.3 Size of gross dimensions

Gross dimensions were chosen to emulate boundary elements of thin shear walls in Chile. Overall length was limited by the testing apparatus and weight constraints. Final dimensions were 6”x12” and 36” long.

3.2 Design of Concrete Specimen

The thin middle section is the boundary element with a tension head on each side that was designed to support the tension test mechanism. Headed deformed rebar donated by Erico were used so develop the bars when subjected to tension. Clear cover and transverse spacing requirements follow ACI 318-08. Figure 2 shows the specimen design; note the tight transverse reinforcement spacing. Figure 3 and 4 are cross sections of the specimen and tension head respectively.

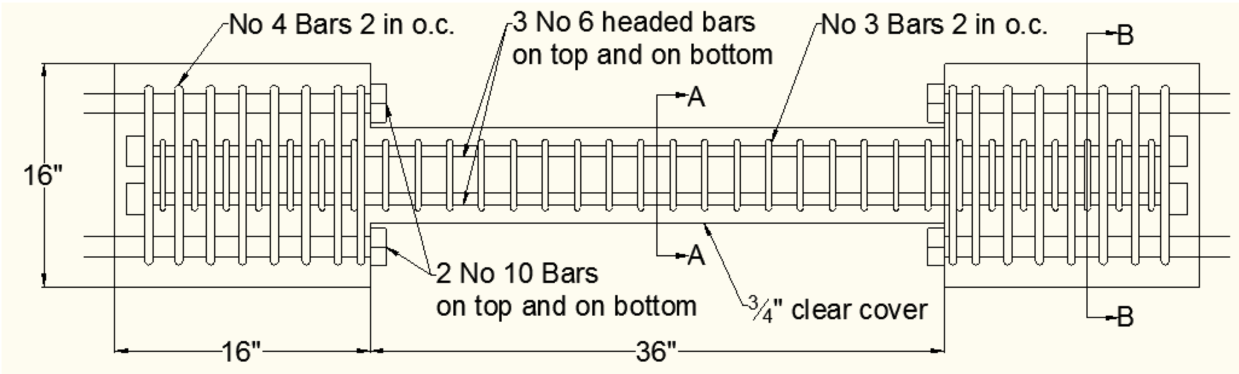

Figure 2: Plan view of concrete specimen design

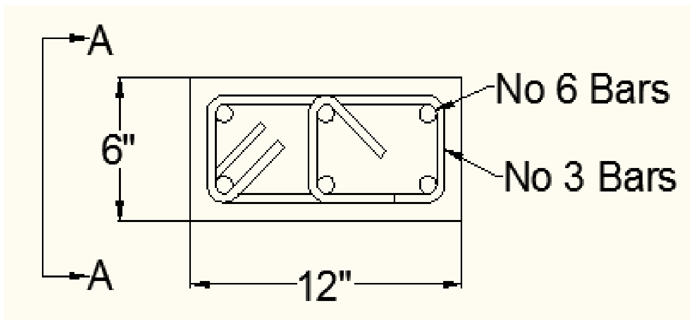

Figure 3: Cross-section of boundary element

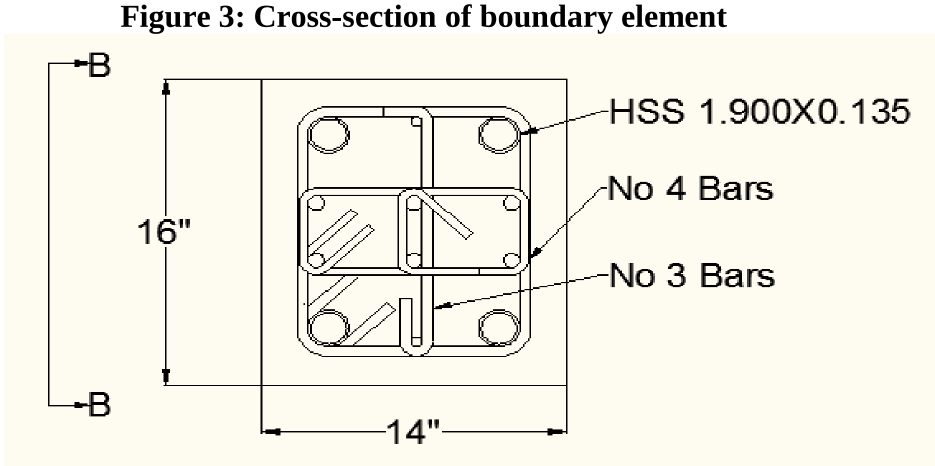

Figure 4: Cross-section of tension head

3.2.1 Transverse reinforcement spacing

The boundary elements tested were detailed to satisfy the requirements for special boundary elements in ACI 318-08 section 21.9.6.4.

3.2.2 Loading mechanism for tension test

In the tension portion of the experiment, the loading head had to be designed to take ultimate strength at which the longitudinal steel would fracture. It was determined to be 264 kips by multiplying an over estimation of ultimate strength of the steel reinforcement to the gross area of six No. 6 bar:

$$F = \sigma * A$$
$$F = 100ksi * 6 * 0.44in^2 = 264kips$$

Using this value, the tension head was designed to resist splitting forces that could develop in the center as the post-tension rods bear on either side.

3.3 Design of Test Set-up

3.3.1 Tension test

The tension test apparatus seen in Figure 5 was designed to have four post tension rods connect each tension head to a plate that bears on the wide flange beams flanking the concrete specimen. On one end a 300 kip capacity hydraulic jack pushes the plate outwards creating tension in the boundary element.

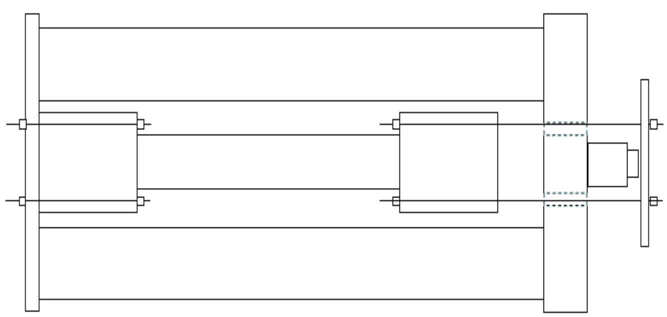

Figure 5: Tension test apparatus

3.3.2 Compression test

The compression test was performed by the "Big Press" in the NEES@Berkeley laboratory shown in Figure 6. Loading capacity is 4,000 kips with best control maintained in tests less than 10 minutes in duration.

Figure 6: 4,000 kip capacity compression machine

3.4 Construction

Figure 7 shows one of two identical reinforcement cages constructed of No. 6 longitudinal bars and No. 3 transverse bars.

Figure 7: Reinforcement cage

Formwork as seen in Figure 8 was constructed by nailing sections together and then assembling units with screws. One was made for each specimen. All the 2x4's were recycled from previous experiments at the NEES lab.

Figure 8: Concrete formwork

Concrete was poured and finished off. Six cylinders per specimen were cast so that compression tests could be done to determine strength before experimentation. The specimens cured by covering with plastic and wet burlap for 25 days. Figure 9 shows the finished concrete.

Figure 9: Finished concrete

After formwork removal, honey-combing was found on the right side of Specimen 1 about 3 ½” in diameter. The hole was grouted before testing, and appeared to show no significant effect on the results. Figure 10 shows the most severe honey-combing before grouting.

Figure 10: Honey-combing found in Specimen 1

3.5 Instrumentation

Longitudinal displacements created as the specimen was tensioned were measured by Novatechnik transducers attached to brackets placed on the top and bottom of each specimen as shown in Figure 11 and 12. Two to measure the strain of the 36” boundary element and two at each loading head to monitor the displacement between the tension head-boundary element interface, so that slippage could be monitored. The transducers were connected to the data acquisition system so that the displacements could be recorded as well as available during the test to determine when 4% strain was achieved.

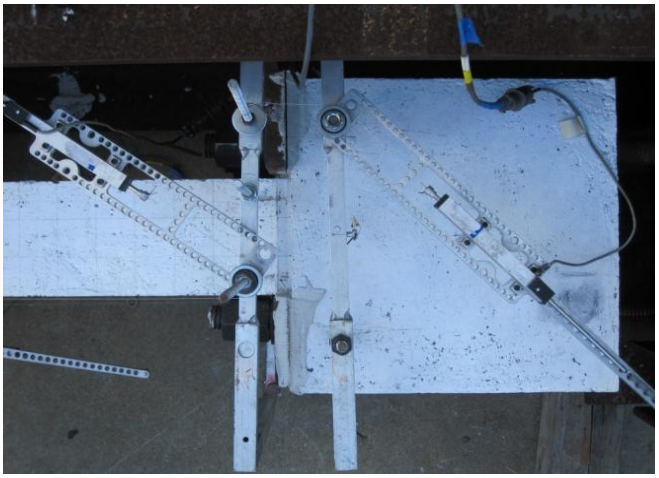

Figure 11: Aerial view of Novatechnik transducers used in tension test

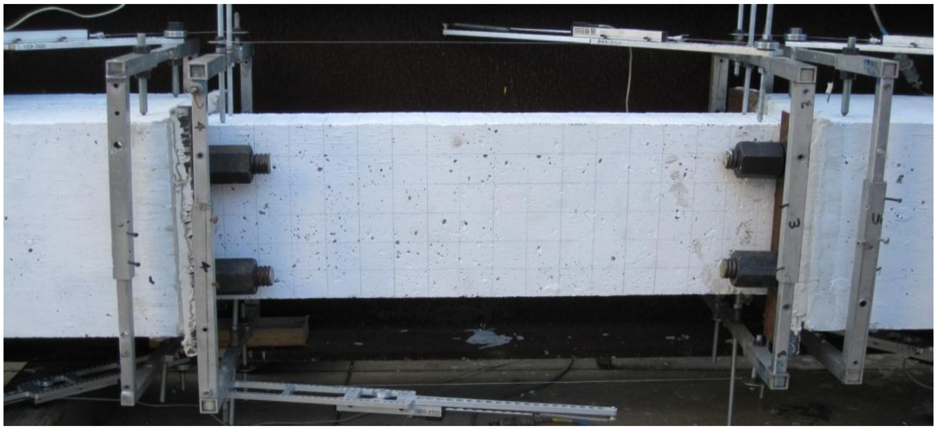

Figure 12: Side view of brackets and Novatechnik locations in tension test

Figure 13 shows the Novatechnik transducers that were also used to record global displacements at the center of the boundary element in the weak axis, which was anticipated to be the plane of bending. In order to measure longitudinal compression strain, wire potentiometers held to the floor were attached to the face of the loading machine.

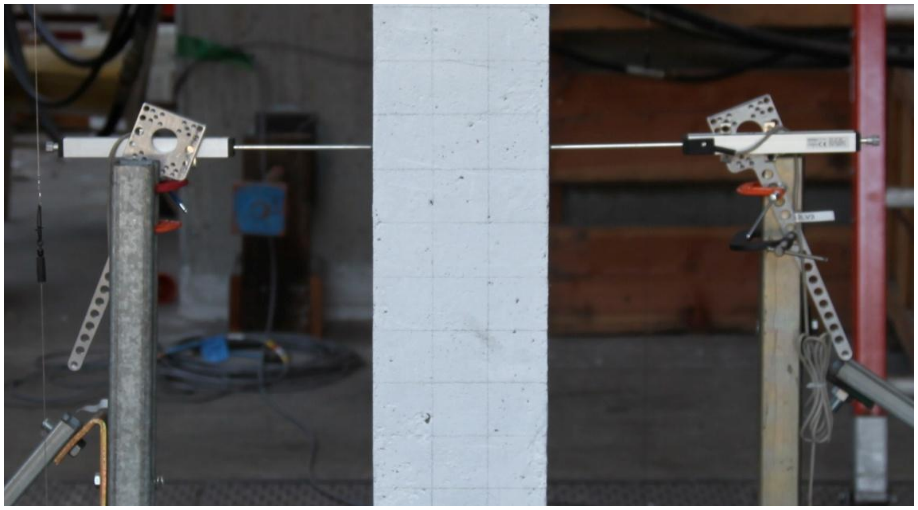

Figure 13: Novatechnik orientations in compression test

3.6 Testing Procedure

3.6.1 Tension test

The hydraulic jack was loaded in increments of 20 kips until yielding was seen in the longitudinal steel at 183 kips. From there, loading continued to 1.5%, 2%, 3%, and 4% strain. At the pause between each increment, cracks were marked and attention was paid to the tension heads to ensure they were performing well. Figure 14 shows the tension test set up without the front wide flange.

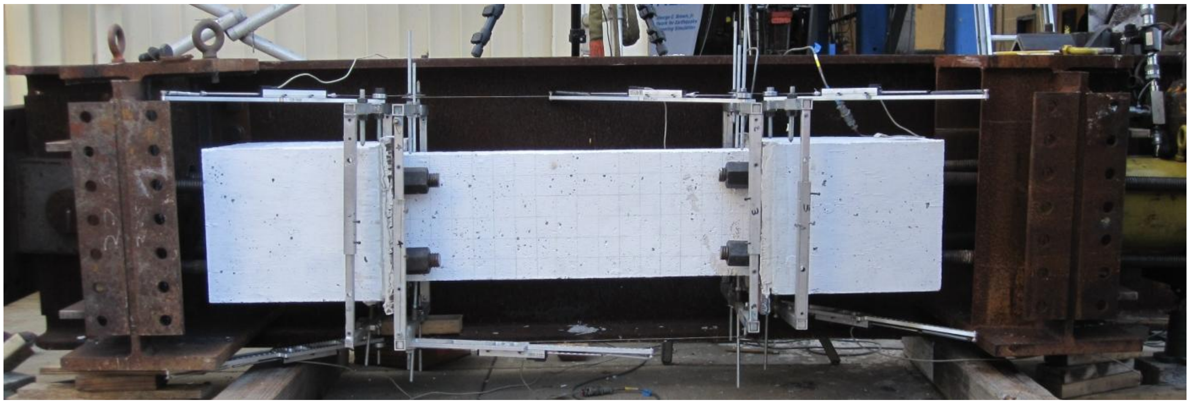

Figure 14: Tension apparatus before wide flange was attached

3.6.2 Compression test

The concrete specimens were grouted in place on the top and bottom in order to ensure an even distribution of force and alignment so that the force was purely axial. Loading began steadily at a rate of 1 kip per second until failure. This can be seen in Figure 15.

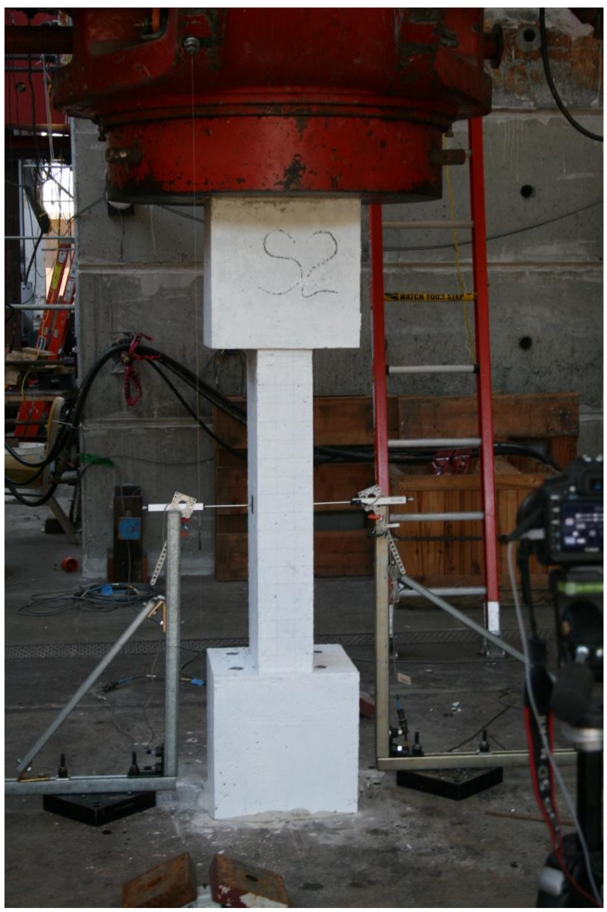

Figure 15: Compression test set up

4 TEST RESULTS

The following pictures demonstrate intermediate results from the tension portion of the experiment. Large cracking developed to ¼" in some cases, and at the very least, hairline cracking could be seen in the plane of each transverse reinforcing hoop. Figure 16 shows the specimen before testing and Figure 17 displays the specimen after the test was performed and the hydraulic jack was unloaded.

Figure 16: Specimen 1 before tension test

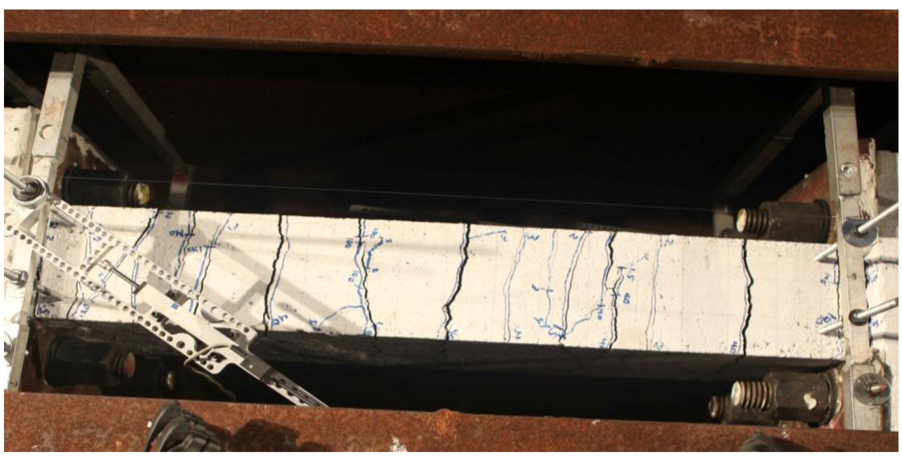

Figure 17: Specimen 2 after tension test

Figures 18 and 19 show a before, middle, and after picture to explain the sequencing of failure. The initial condition of the pre-tensioned specimen consisted of relaxed strain due to unloading without crack closure. As loading began, the concrete cracked up the middle of the cross-section and global buckling was seen immediately. A ductile curve shaped developed before completely giving way. Loading capacity topped at 170 kips.

Comparatively, specimen 2 saw different results. The failure demonstrated brittle response. As the concrete spalled near the top of the boundary element, global buckling occurred instantaneously. All strength was lost immediately. However, specimen maintained loading until 600 kips.

Figure 18: Specimen 1 compression test results

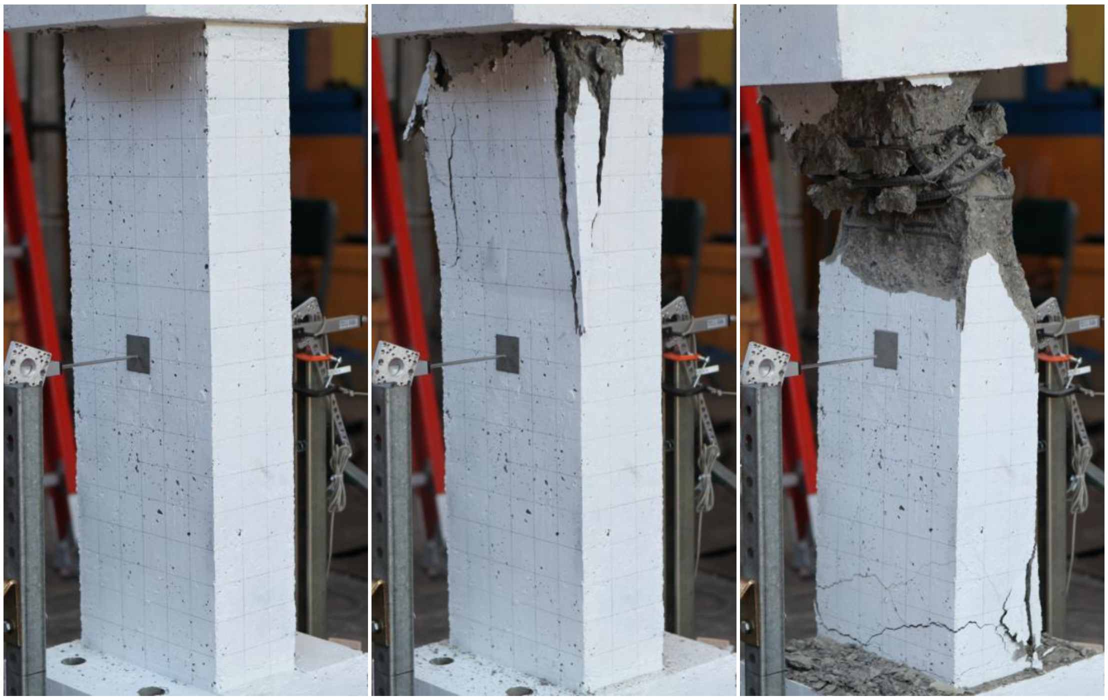

Figure 19: Specimen 2 compression test results

5 ANALYSIS

The results show how dramatic the effect of tension is on the loading capacity of a boundary element. Specimen 1 took less than a third of the loading of Specimen 2 before failure. This can be seen in Figure 20. Positive values denote compression while negative values show tension. Notice how quickly strength is lost in Specimen 2.

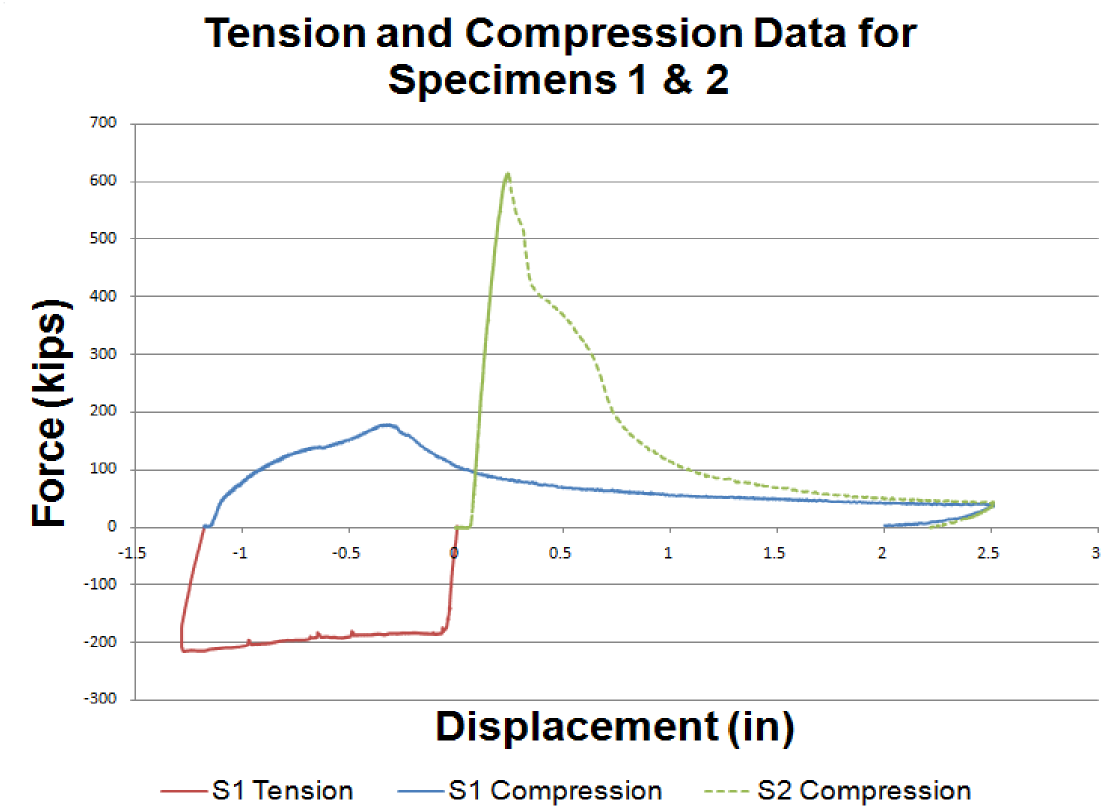

Figure 20: Force vs displacement chart of Specimens 1 and 2

Because the tension test took the longitudinal reinforcement past the point of yielding, residual strain was seen in the bars. This prevented closure of the concrete cracks developed during the tension test. Therefore, as loading increased in compression, a critical condition was created at each crack where it was susceptible to instability because the force was being resisted by only the longitudinal bars. As they began to give way to the left, the rotation caused the concrete to meet on the right side of the specimen. This crushed the concrete under the concentrated force, and all strength was lost.

Specimen 2 results were surprising because ductile response was anticipated because of the high ratio of transverse reinforcement to longitudinal. Instead, an explosive spalling of the concrete occurred and resulted in immediate loss of loading capacity. This could be attributed to small concrete cover or too thin of a minimum cross-sectional dimension.

6 CONCLUSION

A special boundary element subjected to tension prior to compression can tolerate less than a third of the load capacity of a virgin boundary element. Although a more ductile failure was seen in the pre-tensioned case, this drastic decrease in compression capacity is alarming. Continued analysis will take place with emphasis on Euler buckling and critical buckling load. With the following information, further research is needed: investigation of under what conditions boundary elements are vulnerable to extreme tension, most probable strains likely to develop, and performance under multi-cycle loading.

Comparing this work to that done simultaneously on non-special boundary elements, it was shown that he dense reinforcement detailing added little to the performance of the specimen. Because of this failure, possible code revision of section 21.9.6.4 and 21.9.6.5 will be suggested to ACI.

7 ACKNOWLEDGEMENTS

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