Self-Centering Bridge Piers with Structural Fuses

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ABSTRACT

An innovative structural system for pier columns is currently being investigated through a series of laboratory experiments. The columns and connections under investigation are comprised of precast concrete segments to accelerate construction. In addition, some of the columns being investigated employ elastic elements to self-center the columns against lateral loads and structural fuses to control large lateral deflections and expedite repair in the event of a catastrophic loading event.

At the time of publication, two cantilever columns with varying component materials and connection details have been tested in the laboratory and two more are in preparation for testing. The columns are subjected to axial and cyclic, quasi-static lateral loads. After sustaining significant damage, the self-centering columns are repaired by replacing the structural fuses and retested to failure to investigate the effectiveness of the repair.

Of the columns tested to date, the first with a socket connection at the base and no intermediate joints or post-tensioning behaved similarly to a conventional concrete column as expected. The second, a segmented column tested with elastic post-tensioning and structural fuses, experienced a premature failure due to cracking of a weld at a steel collar and subsequent bolt pull-out. Alternate detailing to avoid this failure mechanism is planned for the remaining tests.

Key words: post tensioning—precast concrete columns—rapid bridge construction—structural fuses

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PROBLEM STATEMENT

The objective of this research is to accelerate bridge pier construction through the use of precast columns in order to reduce construction costs, decrease traffic delays, improve work zone safety, and minimize environmental impacts.

Furthermore, it aims to develop a pier system that could endure an extreme loading event such as an impact, severe wind storm, flood, blast, or earthquake. The pier should be able to sustain large deformations, be tough and durable, and be easily repaired.

Precast substructures have been used around the country with varying degrees of success over the past two decades. A concise overview of the existing technology is given in “State-of-the-Art Report on Precast Concrete Systems for Rapid Construction of Bridges” by Hieber et al. (2005) The system examined in this research offers a different design approach and details that have the potential to reduce the construction time and improve structural performance. The basic proposed pier assembly is illustrated in Figure 1. Key features include steel collars at the ends of segments (Figure 2), external reinforcement of segment joints which have bolted connections (Figure 3), and bearing plates between segments to avoid labor-intensive grouting procedures (Figure 4).

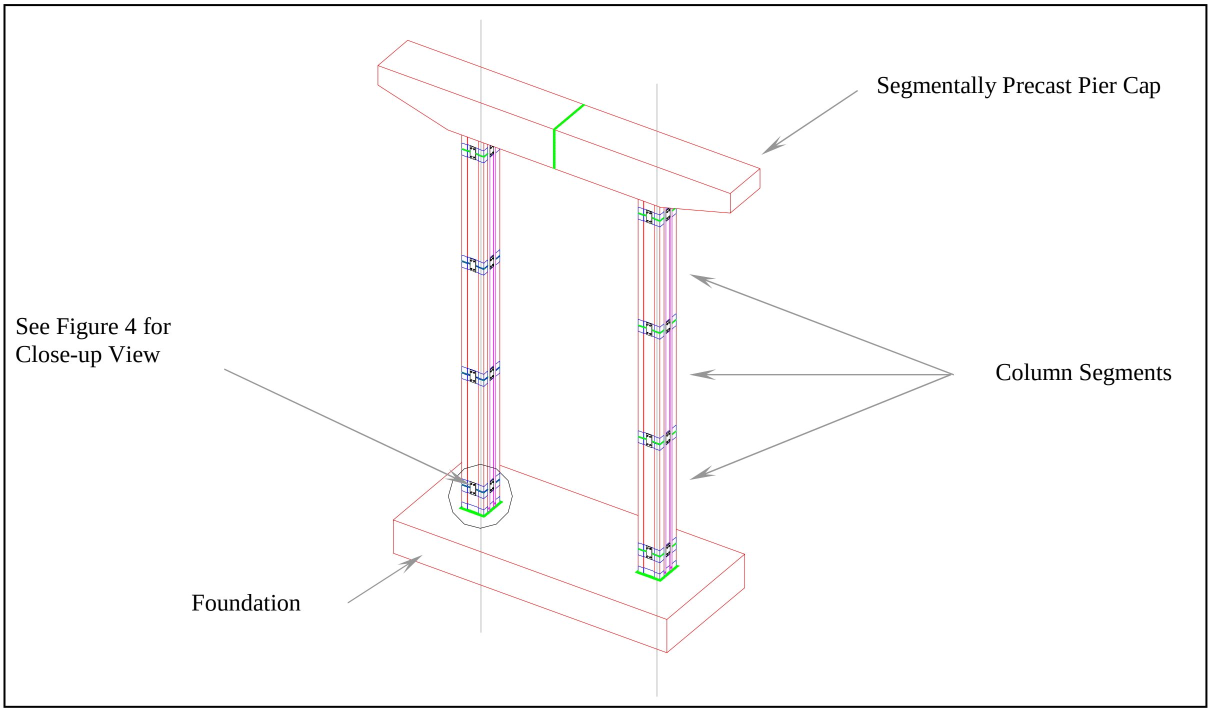


Figure 1. Basic pier assembly (isometric view)

The steel collar assembly to be cast at the ends of each column segment, as illustrated in Figure 2, serves three purposes: reinforcement of the concrete corners to prevent damage during shipping and erection, confinement of the concrete at the ends of the segments to provide additional concrete strength and ductility, and a convenient and aesthetically pleasing means for attaching the exterior reinforcement plates.

A single column segment is illustrated in Figure 3 with steel collars at each end. Both the connections between the foundation and column shaft and the connection between column segments are illustrated in Figure 4. The first (lowest) column segment fits into a socket formed in a cast-in-place or precast pile cap or a spread footing. The annular space at this location is filled with a flowable epoxy grout. The column segments are separated by bearing plates.

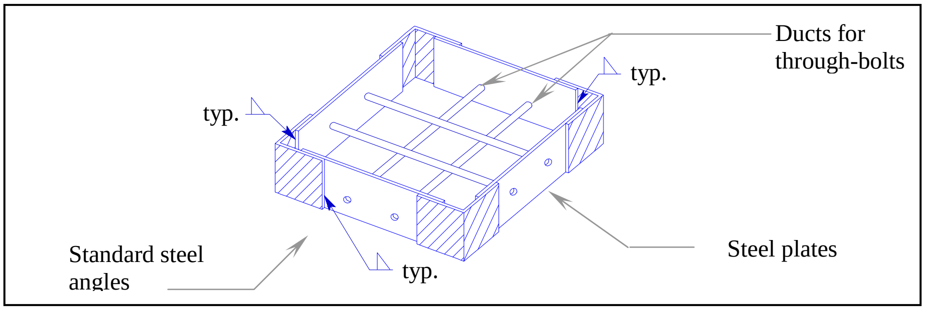


Figure 2. Steel collars at segment ends

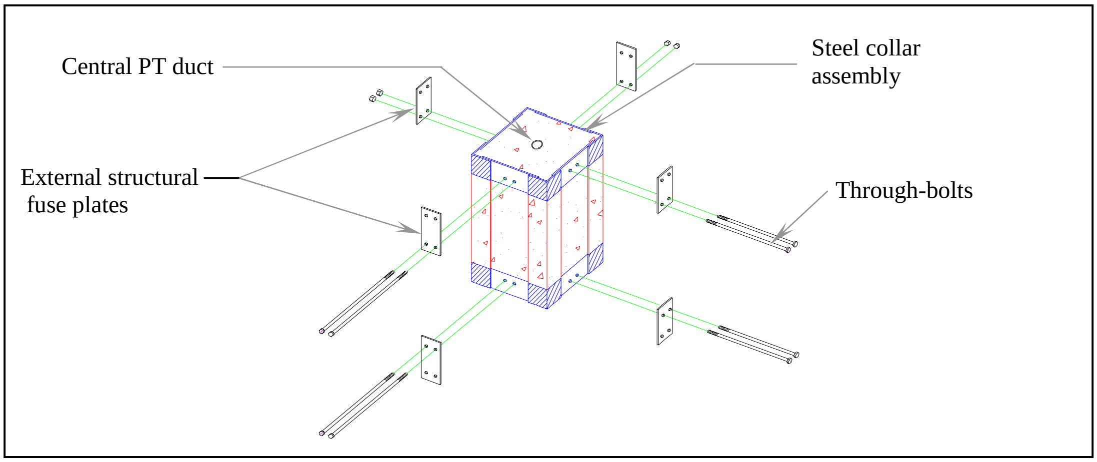


Figure 3. Single column segment with external connectors

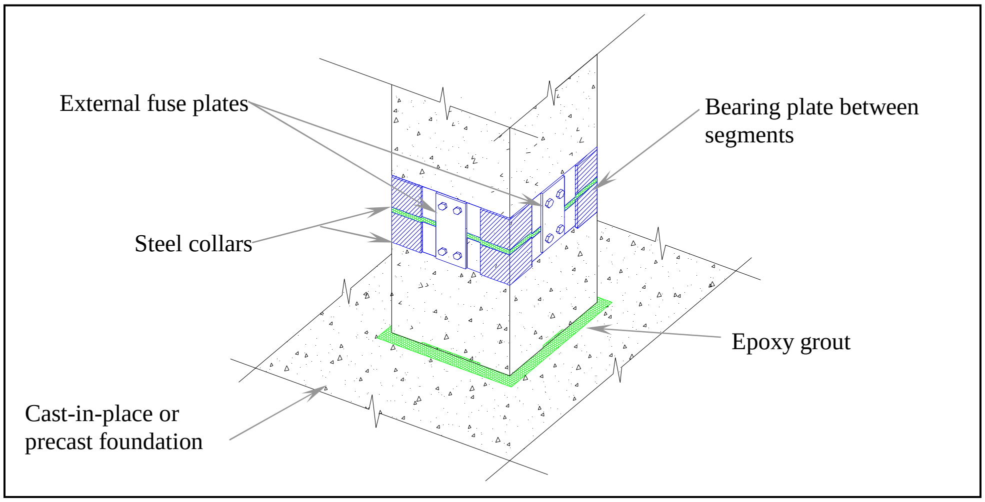


Figure 4. Close-up of typical joint

A possible construction sequence is as follows. Once piles are driven, a precast pile cap is placed or cast with sockets to receive precast column segments. A vertical alignment rod is threaded into an anchor cast into the bottom of the socket, and the first column segment is lowered into place. Once shimmed and leveled, flowable epoxy grout is poured into the annular socket space. A bearing plate is placed at the top of the last segment placed, and the next segment is subsequently lowered over the alignment rod to rest on the bearing plate. The external fuse plates previously described are then bolted into position. This procedure continues until all but the uppermost column segments are in place. The uppermost column segment is epoxied into the pier cap socket and the unit is lowered onto the columns. Once all pieces are in place, cap segments are connected, external plates are secured by fully tensioning the through-bolts, and nuts are threaded onto the top of the alignment rods to post-tension the column. The pier would then be ready to receive the superstructure. This research is placing special emphasis on the connection designs to simplify construction. Although Figures 1–4 schematically illustrate square columns, the connection details could be developed for rectangular or circular columns as well.

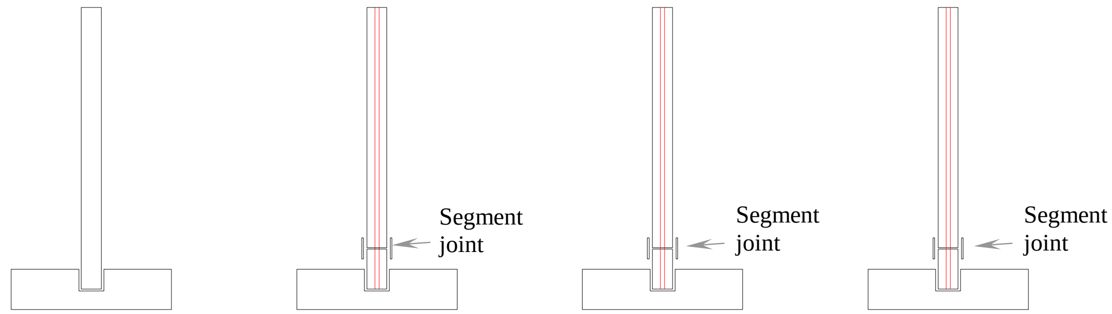
The segmentally precast pier provides the economic and aesthetic advantages usually ascribed to any precast concrete system. Because the concrete is cast at a plant rather than in the field, environmental conditions that are crucial to freshly placed concrete may be more closely monitored and controlled. The usual result is higher quality concrete that is more durable over the life of a structure. The precast pieces may be cast early in the project schedule and then be rapidly assembled in the field even during temperature extremes that normally pose problems for cast-in-place structures. Architectural finishes may also be expediently applied in the plant providing a wider range of appearances for the completed structure. By casting the pier columns in segments, shipping and handling costs will be reduced and smaller, lighter equipment will be required for field assembly.

In addition to the economic advantages of accelerated construction, the system can be designed to provide advantages in structural performance under extreme lateral loads arising from events such as impacts, severe winds, floods, blasts, or earthquakes. This behavior may be achieved by designing the easily replaceable external plates as structural fuse plates. The structural fuse plates would serve to concentrate damage in the fuses themselves while leaving the remainder of the structure relatively undamaged. Continuous elastic elements, such as a post-tensioning rod may also be incorporated in the members to provide a self-centering force and minimize residual deformations. By achieving minimal residual deformations, the bridge may remain in service immediately after an extreme loading event. Full repair of the structure, if required, may then be accomplished simply by replacing the structural fuse elements. It is anticipated that the incremental costs may be minimal because such fuse elements can serve the dual purpose of expediting construction and improving structural performance.

RESEARCH METHODOLOGY

Key Variables

Because the greatest uncertainties in this system are associated with the behavior of the segment joints, cantilever column specimens were designed to focus on the behavior of a single segment joint. Test variables to be investigated include the types of bearing plates, fuse plates, and amount of post-tensioning force applied. Figure 5 shows the variables selected for each test. One column was cast as a single unit to examine the behavior of the socket-type base connection and serve as a basis for comparison with the subsequent jointed columns. Columns 1 and 2 have been tested, and Columns 3 and 4 have not yet been tested.



Column 1:	**Column 2**:	**Column 3**	**Column 4**
·Precast concrete column ·No PT ·No intermediate Joint	·Segmental column ·A36 steel fuse plates ·Epoxy grout bearing plate ·PT= 44 kips	·Segmental column ·1018 steel fuse plates ·Polymer concrete bearing plate ·PT= 66 kips	·Segmental column ·Shape memory alloy fuse plates ·FRP bearing plate ·PT= 66 kips

Figure 5. Test columns

Bearing Plates

The bearing plate in the precast pier system is one of the variables being investigated. The bearing plate must be compliant enough to transfer axial stress uniformly between segments but strong enough to withstand high shear, axial, and bending stresses. Several different types of plates were considered with desired properties that included high compressive, tensile, and shear strengths, a modulus of elasticity roughly half that of concrete, high resistance to corrosion, low creep, and relatively low cost. Materials considered were an epoxy grout plate, a polymer concrete plate with a steel reinforcement, a fiber-reinforced polymer (FRP) plate, a lead plate, a neoprene pad, and a cotton duck pad. The following were selected for testing.

- **Epoxy Grout.** Sikadur 32, a high modulus, flowable epoxy was mixed with Grade 37 silica sand in a 1:1 ratio for Column 2. The modulus of elasticity was roughly 1,500 ksi, while the compressive strength of the epoxy-grout was roughly 11.5 ksi. When tested in compression, this material exhibited large plastic strains without cracking.

- **Polymer Concrete.** A steel-reinforced polymer concrete plate is planned in the third column test. In polymer concrete, a high-strength, corrosion-resistant, thermosetting resin acts as the binding agent. Three-eighths in. aggregate will be used in the plate, and a two inch by two in. welded wire grid will be placed in the plate to reinforce against splitting. The modulus of elasticity of polymer concrete is approximately 2,400 ksi, while its compressive strength is approximately 10 ksi.

- **Fiber-Reinforced Polymer.** A carbon fiber-reinforced polymer (CFRP) plate is planned for the fourth column test. An epoxy resin acts as the binding agent for CFRP, with a 90-90 fiber orientation. The elastic modulus for the CFRP bearing plate perpendicular to the fibers is expected to be approximately 2,000 ksi.

Fuse Plates

The fuse plate in the precast pier system is another variable under investigation. Desired properties include low yield stress, large plastic strain region, high resistance to corrosion, large ultimate elongation (~20 to 25%), high toughness, and low cost. Materials being considered include A36 steel, 1018 carbon steel specifically manufactured to yield between 30 and 36 ksi, shape memory alloy, A242 steel, and A588 steel. The following were selected for testing.

- **A36 Steel.** A36 steel plates were used in the second test column. From laboratory tests, the plates were determined to have a yield stress of 42 ksi with an ultimate elongation near 20%.

- **1018 Carbon Steel.** 1018 carbon steel will be used for the third test column. It has a yield stress of 30 to 36 ksi and an ultimate elongation similar to that of the A36 plates.

- **Shape Memory Alloy (SMA).** Shape memory alloy will be used for the fourth test column. It has a yield stress plateau similar to A36 steel, with much less permanent elongation. The yield stress plateau is approximately 50 ksi. Although more expensive than conventional steel, the SMA plates provide the advantages of helping to self-center the column and are less likely to require replacement.

Post-tensioning

The post-tensioning force applied to the columns is the final variable being investigated. The forces applied to each test specimen were noted previously in Figure 5. Higher initial PT force increases the lateral stiffness of the column, increases self-centering force, and reduces cracking in the concrete at the expense of axial capacity of the column.

Specimen Design and Construction

Laboratory size constraints restricted the height of the columns, which in turn established column gross cross section dimensions. Fuse plate and bearing plate dimensions and material properties were then selected. Design dead load and post-tensioning forces were also determined based on these limiting criteria. For simplicity of design and analysis, fuse plates were only used on the tension and compression sides of the columns.

An analytical approach using reinforced concrete design techniques was used for the design and theoretical analysis of the columns and foundation blocks. Force and moment equilibrium and strain compatibility were used to establish predicted values for load-deflection behavior, stresses, and strains in various elements of the columns.

Four identical foundation blocks were cast to provide a base connection for each column specimen. The foundation blocks were designed to restrict rotation at the base of the columns. A socket was used to provide a connection between the foundation block and the test columns. Five ksi concrete and mild reinforcing steel were used for the blocks. Foundation block dimensions are shown in Figure 6.

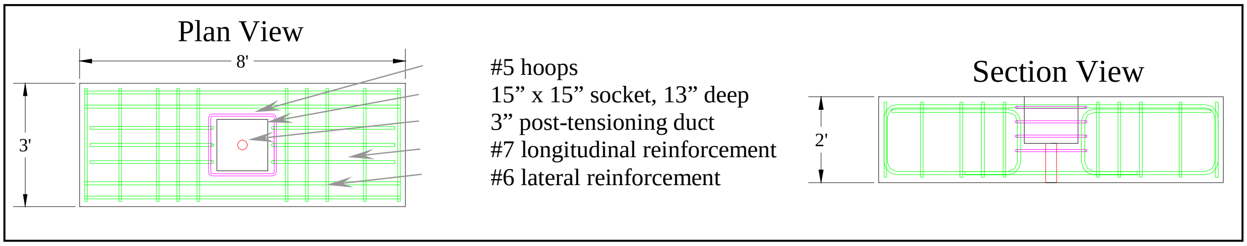
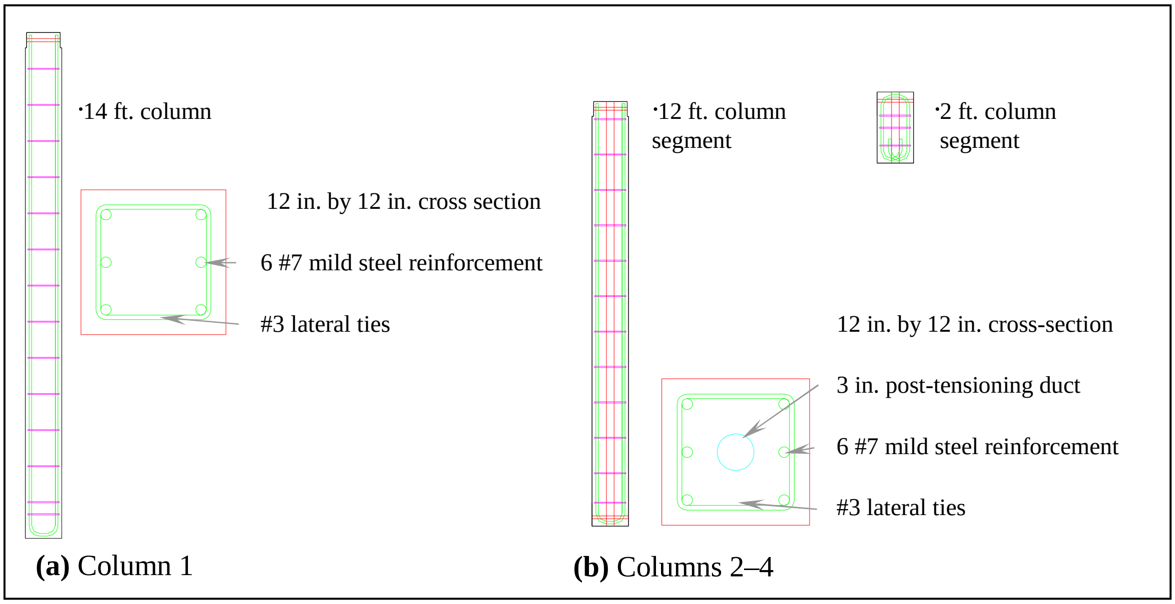


Figure 6. Foundation blocks

Four test columns were cast. The first column was cast as one continuous reinforced concrete element 14 ft. in length. Six #7 mild reinforcing bars were used for longitudinal reinforcement and #3 mild reinforcing bars were used as lateral ties spaced on 12 in. centers. The three remaining columns were cast in two segments. Each column consisted of a two ft. reinforced concrete segment and a 12 ft. reinforced concrete segment. Reinforcement for all segments consisted #7 mild steel bars spaced as in the first column.

The segments of the jointed columns were cast with steel collars on one end. The collars were fabricated with one in. diameter through-ducts for bolts to connect the fuse plates to both column segments (see Figures 2 and 8).



(a) Column 1

(b) Columns 2–4

Figure 7. Test columns

Figure 8. Steel collar

Test Setup, Instrumentation, and Procedure

The test was designed to apply a constant, vertical dead load, as well as a cyclic, quasi-static lateral load. Dead weights were hung from a pier cap attached to the top of each column specimen. Each hanging block weighed 18.5 kips, while the pier cap weighed 7 kips for a total of 44 kips. Column base fixity was achieved by grouting the column into the socket in the foundation block and post-tensioning the foundation to the lab strong floor.

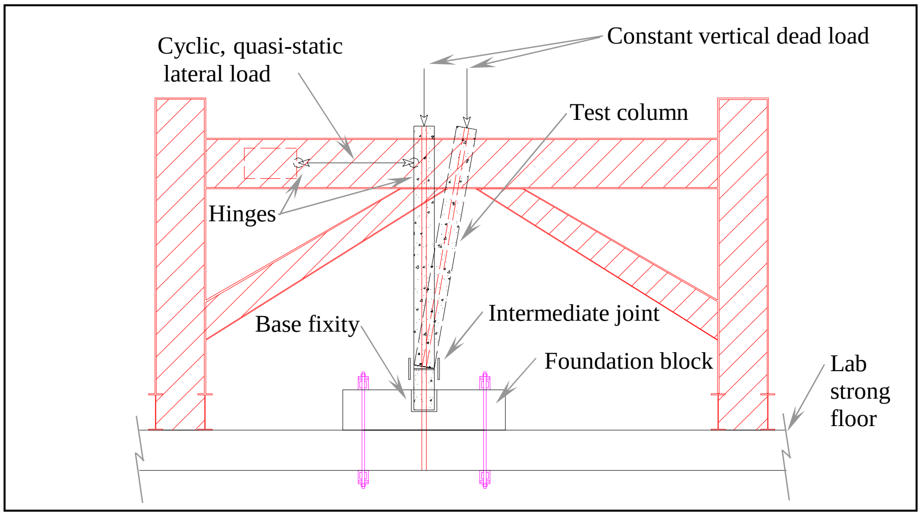


Figure 9. Test setup schematic

For the second test column, fuse plates stabilized the column segments before post-tensioning or dead weight was applied. Oversize holes in the fuse plates allowed for alignment. Once the post-tensioning was applied, the column was stable without the fuse plates. The fuse plates were then set into their final position. Four one in. diameter A490X bolts were used to connect the fuse plates to the column segments. A slip-critical connection was designed for the joint, so direct-tension-indicator washers were used to indicate when the bolts were adequately tensioned. Figure 10 shows the final test setup after the dead weights were applied.

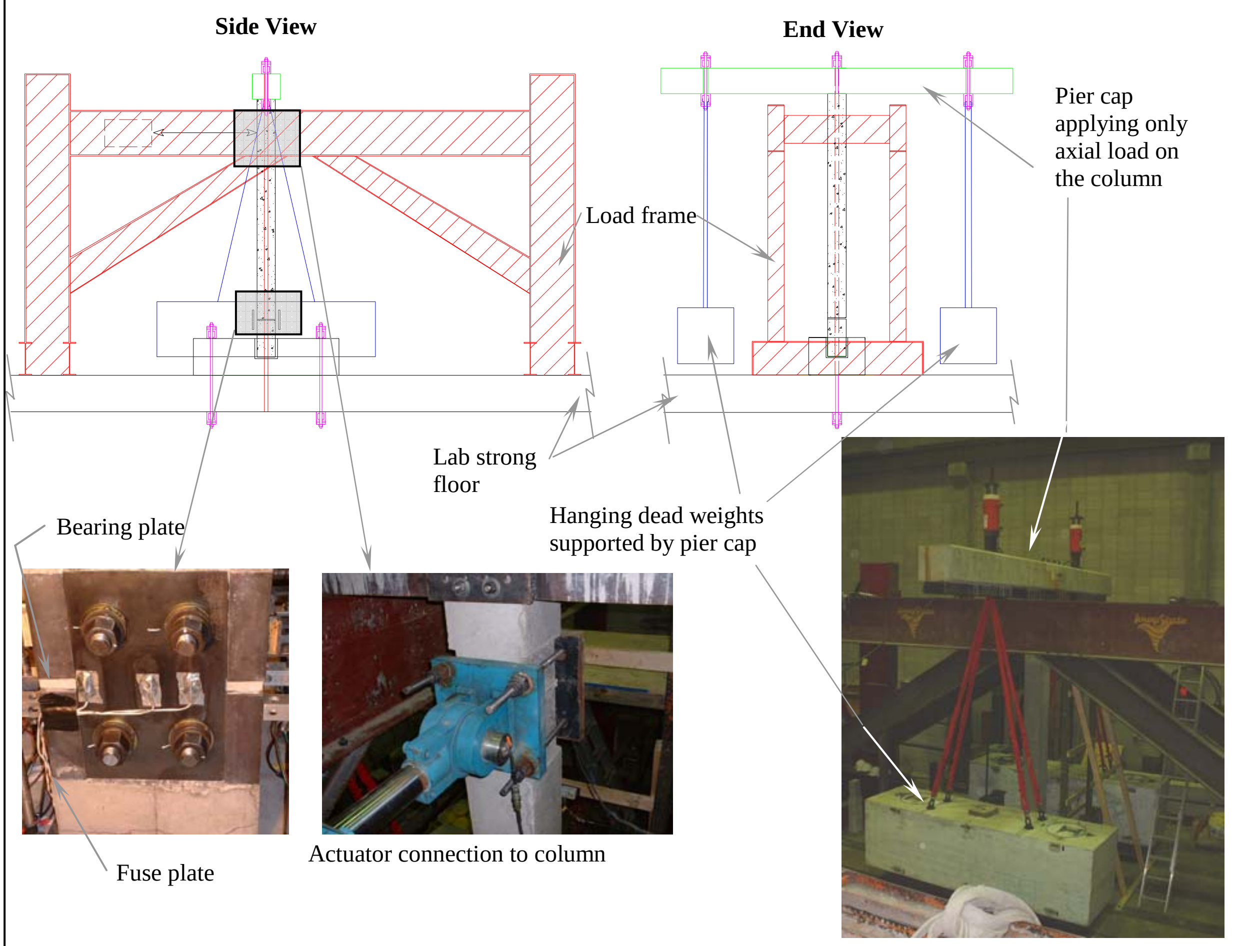


Figure 10. Test setup

Instrumentation

Several types of instrumentation were used to acquire data from the tests. Load cells were used to measure the post-tensioning force and lateral load. Lateral displacement transducers, located 7 in., 15.75 in., 30 in., 60 in., 96 in., and 134 in. (height of actuator) from the base were used to measure column lateral deflections. Tilt of the column was measured by inclinometers located 1 in., 7 in. and 15.75 in. from the base connection. Displacement transducers were used to measure deformations in the segment joint. Four were mounted on both faces perpendicular to the fuse plates, as shown in Figure 11. Strain gauges were bonded on mild reinforcing bars near the base of each column to measure strain. Strain gauges were also used to measure strains in the fuse plates.

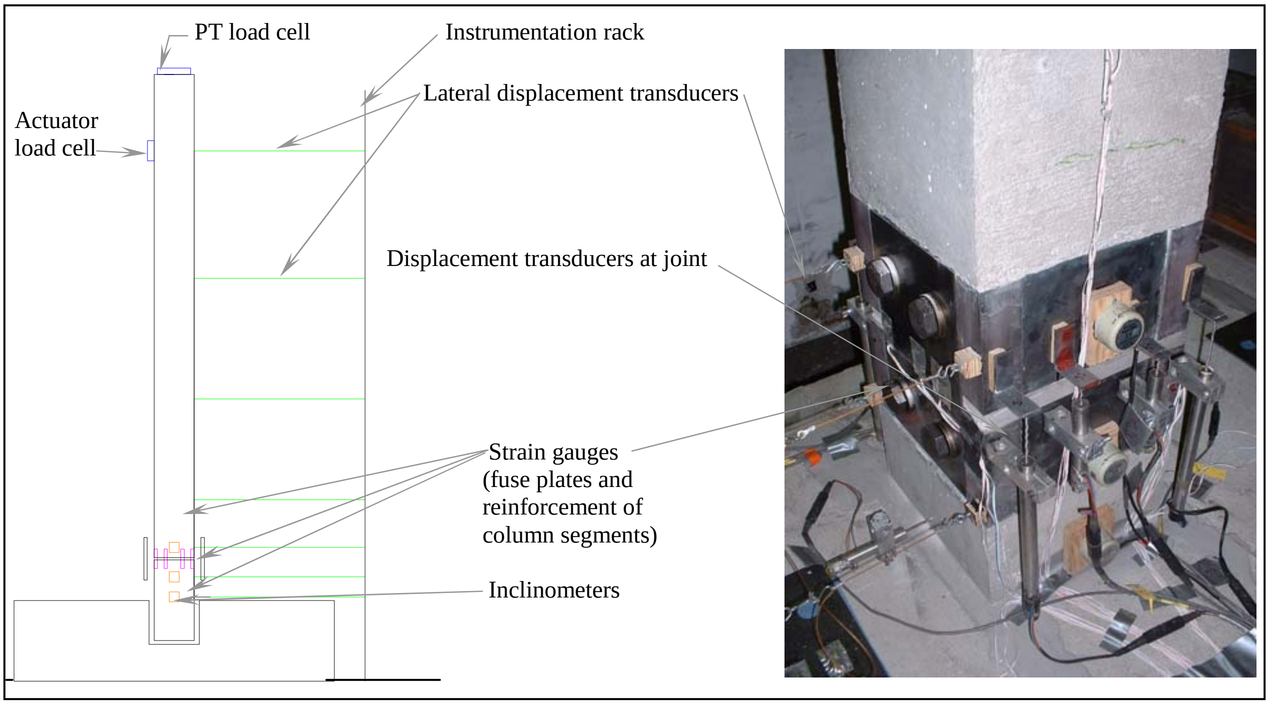


Figure 11. Instrumentation

Testing Procedure

A cyclic, quasi-static lateral load was applied to each test specimen. Loading on the column was displacement-controlled and increased incrementally until failure. Test data were taken at each displacement increment from each of the strain gauges, inclinometers, and displacement transducers using a data acquisition system (DAS). The specimen was examined for damage at each peak displacement of each cycle, and photographs were taken every few cycles. The displacement regimen used in the tests is shown in Figure 12.

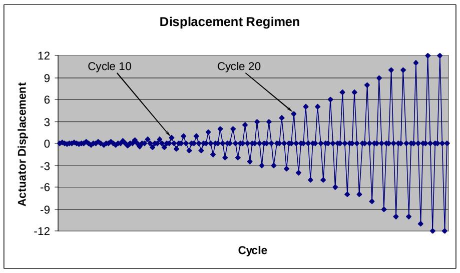


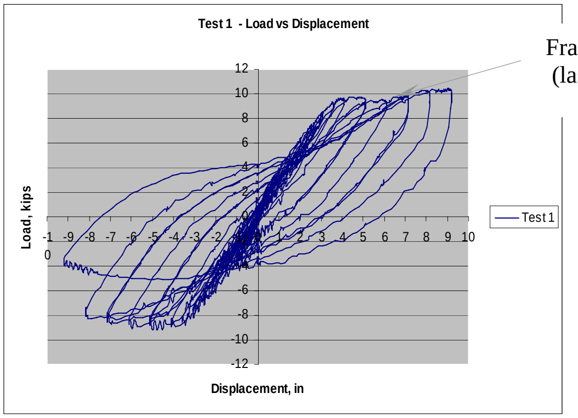
Figure 12. Displacement regimen

The second test column followed a similar procedure with one major modification. Once the column had been displaced 2.5 in., yielding and buckling of the fuse plates was apparent. At this point, the fuse plates were replaced with new plates and the loading regimen was started over and continued until the column failed.

TEST RESULTS

Specimen 1

Lateral loads were applied to a maximum 9 in. displacement in the positive and negative directions. Cracking began near the base and progressed upward beginning on cycle seven (0.40 in. displacement, 2.29 kip load) at an applied base moment of 25.68 ft.-kips. The predicted load for initial cracking was 2.32 kip. The column had the most significant damage on the southwest corner when the mild reinforcement began to buckle. This caused the column to tilt slightly, perpendicular to the axis of loading. When positive displacement was induced, the pier cap began to impinge on the loading frame during the last four cycles causing a significant increase in lateral load. This can be seen in the hysteresis for the Column 1 test in Figure 13.



Frame impingement
(last 4 cycles)

Figure 13. Test 1 hysteresis

Strain gauge data indicated that the longitudinal reinforcement yielded in tension on cycle 15 (displacement 2.5 in., 7.54 kip load) at an applied base moment of 84.20 ft.-kips. The mild reinforcement buckled roughly nine in. above the base. Confinement at the base provided by the socket in the foundation and by a lateral tie three inches above the base prevented buckling from occurring at the base.

LVDT and inclinometer data indicated minimal rotation at the column base. At peak lateral loads the rotation at the base was measured at 0.67 degrees and -0.78 degrees for push and pull cycles, respectively. The epoxy at the socket connection showed no signs of cracking during the test. See Figure 17(a) and (c) for the deflected shape of Specimen 1.

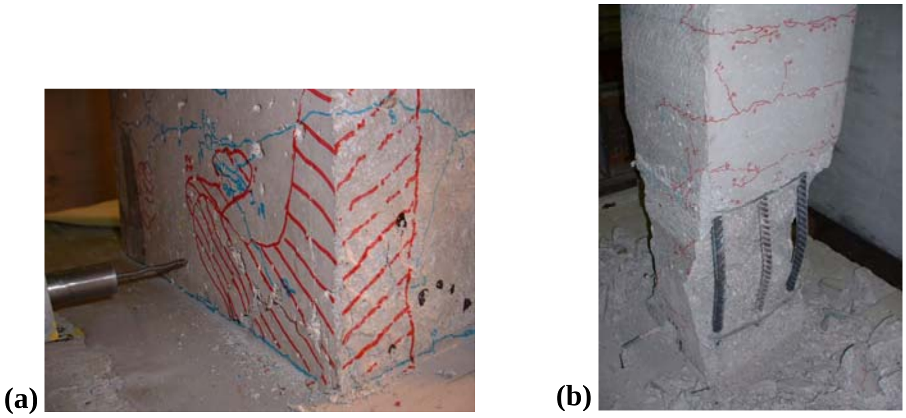


Figure 14. (a) Spall initiation at southwest corner, (b) Column 1 after testing

Test 2 Results

Test 2 consisted of two loading regimens. The first test (referred to as 2a) included cycles to a displacement of 2.5 in., when significant damage had been achieved in the fuse plates. After replacing the fuse plates, the loading regimen restarted and continued until failure of the column (test referred to as 2b).

Cracking at the base of the column began on cycle ten (0.75 in. displacement, 4.45 kip load) at an applied moment of 49.69 ft.-kips. This specimen resisted cracking for three cycles (0.35 in.) more than Column 1 due to the post-tensioning. Minor cracking progressed up the column during the following cycles up to a displacement of 2.5 in., when the plates were replaced.

Yielding of both fuse plates was achieved on cycle 14 (2.0 in. displacement, 6.66 kip load). Noticeable plate buckling also began on cycle 14. At this point, an unforeseen failure mechanism developed when a weld cracked in the collar and the through bolts began to pull out of the upper column segment. After replacing the fuse plates, the loading cycle was repeated up to +/- 6 in. displacement. Only minor cracking occurred in the column during the remainder of the test. Neither segment experienced significant damage or spalling other than bolt pull-out. No mild reinforcement reached the yield stress in the second test.

The hysteresis for test Column 2 (see Figure 15), shows the relatively small residual deflections for the column. Column 2 had an initially stiffer response to loading than Column 1. The evident changes in slope occurred when the fuse plates yielded, and load declined when the collar cracked.

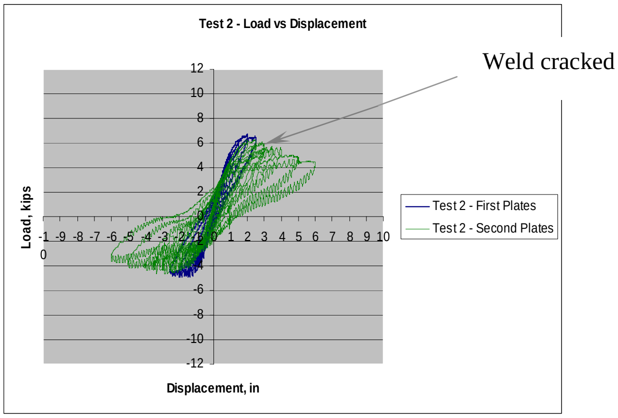


Figure 15. Test 2a and 2b hysteresis

The epoxy grout plate exhibited no evidence of damage until the final cycle, when a small crack was noticed on the northwest corner of the plate. Pinging sounds noticed in the bolts may have indicated some slippage in the slip-critical connection; however, no bearing on the holes was evident after the plates were removed.

In the test on Column 2 there was negligible rotation at its base, as indicated by the LVDT and inclinometer data. These data showed rotation at the base at peak loads of 0.1 degrees and -0.07 degrees for push and pull cycles, respectively. The epoxy at the socket connection showed no signs of cracking during the test. See Figure 17(b) and (d) for the deflected shape of Column 2.

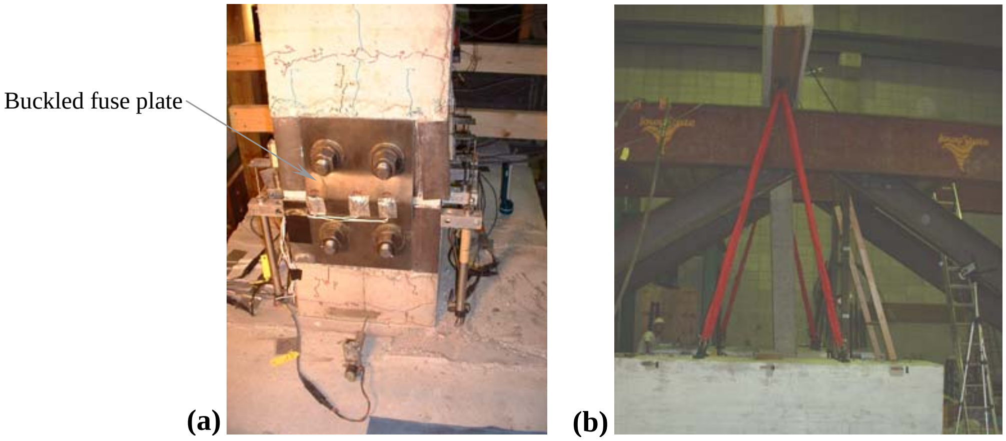
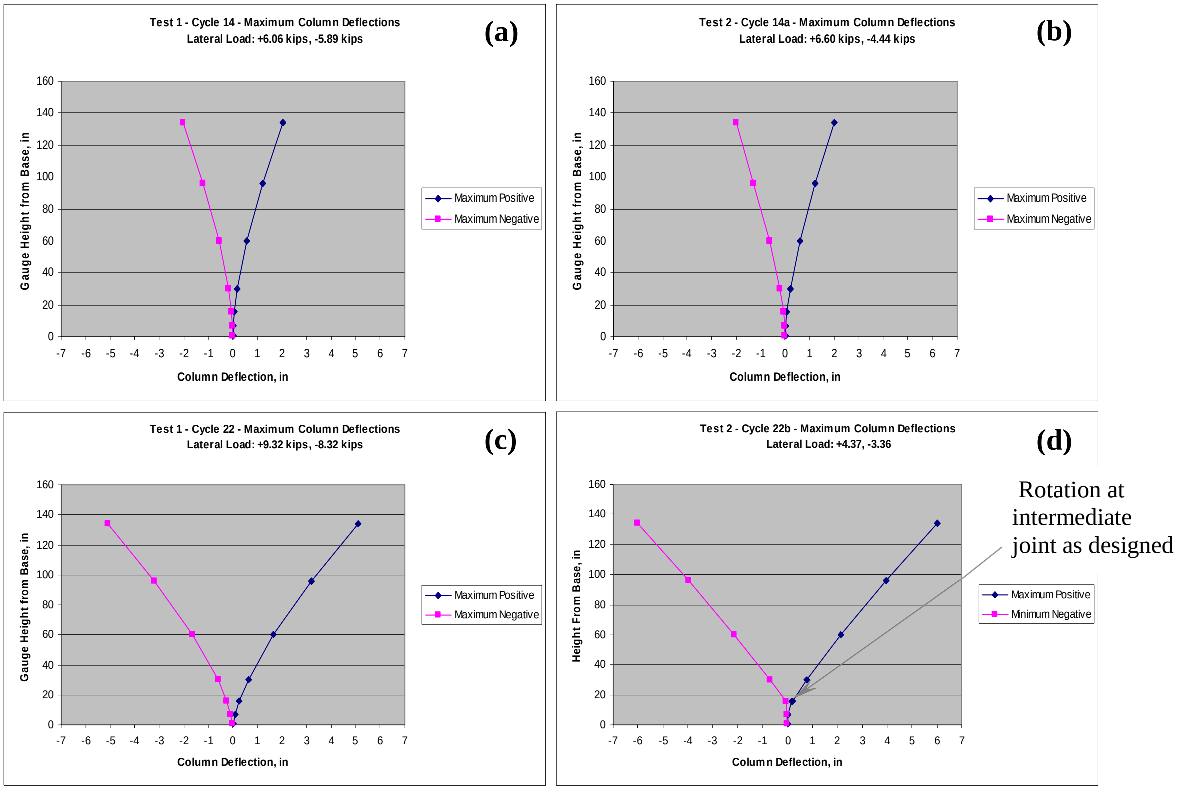


Figure 16. (a) Buckled fuse plate with cracks in column, (b) Displaced column



Rotation at intermediate joint as designed

Figure 17. (a) Test 1 deflected shape, cycle 14, (b) Test 2a deflected shape, cycle 14, (c) Test 1 deflected shape, cycle 22, (d) Test 2b deflected shape, cycle 22

SUMMARY

The unjointed Column 1 behaved, as expected, similarly to a conventional concrete column. Predictions of first cracking load, load at yielding of reinforcement, and ultimate lateral load capacity closely matched conventional computations for reinforced concrete columns. Lateral displacements were slightly larger than predicted due to the elastic deformation of the epoxy grout at the socket. The rotations measured at the socket, however, were small, and the connection showed no indication of damage during the test. Furthermore, the socket connection detail was easy to construct with the greatest additional time cost in forming the socket itself.

The Test 2 segmented column displayed much lower lateral load and deflection capacity than Column 1 due to the unforeseen failure of the welded collar and subsequent bolt pull-out. This failure mechanism may be addressed by using perpendicular through-bolts at the collar, specifying heavier welds, or by slight modification of the reinforcement detail at the end of the column. Subsequent tests will address this issue.

Despite the premature failure of the column, initial lateral stiffness was greater and residual deformations smaller than those observed in Column 1. Yielding and buckling of the fuse plates occurred as predicted until cracking of the weld. Cracking in the column was significantly reduced relative to Column 1 since curvature was concentrated in the intermediate joint as designed.

ACKNOWLEDGMENTS

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