Alternative ABC Connections Utilizing UHPC (Feasibility Study)

Final Report

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ABSTRACT

Accelerated Bridge Construction (ABC) is a method of bridge construction designed to reduce traffic interruption, minimize onsite construction time, and increase work zone safety. One solution to accelerate and facilitate the construction is by utilizing precast elements; however, connections between prefabricated elements could be a challenging issue, especially in seismic regions. Ultra-High Performance Concrete (UHPC) has several superior characteristics that can assist in the implementation of ABC in practice, such as rapid early age strength gain, and allow anchorage of reinforcement over a very short length.

This paper proposes an innovative connection between a precast column and cap beam, which can potentially satisfy constructability requirements and expected seismic performance. The main characteristics of the proposed connection are desired plastic hinge location and large construction tolerances. An experimental test was performed to evaluate the seismic performance of the proposed connection. The observed displacement ductility exceeds those implied by design specifications. The most interesting aspect of the proposed detail is the certainty associated with where plastic hinge forms and its length. This objective is achieved by sandwiching a certain length of the column, using normal strength concrete (plastic hinge region) in between two layers of UHPC. The behavior of the proposed connection has been further investigated by using nonlinear finite element analysis. Comparison between simulated model and experimental results shows good correlation.

1 INTRODUCTION

The bridge infrastructure in the United States consists of almost 600,000 bridges, majorities of which were constructed in the middle of the 20th century. Although concrete has shown to be a viable building material for bridges, material durability problems have been encountered. In addition, the lower design standards used during their construction have made the maintainability of these bridges, a challenge. Many bridges are now structurally deficient or functionally obsolete and are consequently in need of replacement or repair.

Replacing or retrofitting of any bridge that includes demolition and construction activities has a significant impact on traffic and safety. Minimizing the amount of construction performed in the field not only decreases detour time and traffic jams, but also increases safety for workers, vehicles, and the public. Accelerated Bridge Construction (ABC) offers a new solution for reducing construction time while building, replacing, or retrofitting bridges. The most common form of ABC uses pre-fabricated modular bridge systems. These systems require joints between elements and some agencies have expressed concern regarding the durability and structural integrity of the joints [1]. It has been recognized that column-to-cap beam or footing connection plays a critical role in the behavior of the bridge in seismic regions. Various methods for joining column-to-cap beam have been developed, but none of these methods have entirely solved construction issues. In light of the challenges inherent in connecting precast concrete elements, exploring new materials such as Ultra-High Performance Concrete (UHPC) can be beneficial. Superior characteristics of UHPC, such as the short development length and the ability to gain strength during a short time, can assist ABC construction. Implementation of ABC for bridge columns in moderate and high seismic regions has been limited due to a lack of performance data pertaining to column connections, which must be able to transfer seismic forces while allowing the column to have large inelastic deformations.

Several studies have been conducted on the methods of connection between column and cap beam, which are commonly used in practice. In 2011, comprehensive experimental research was performed under NCHRP project 12-74 on several precast bent cap details including cast-in-place, grouted duct, cap pocket, and hybrid connections. Design flowcharts were recommended based on the results for seven cap beam specimens [2]. Another research study evaluated cap beam-column connections for seismic and non-seismic areas. The study characterized precast or prefabricated elements in bridges in seismic areas into two categories, energy-dissipating and capacity-protected, and concluded that there is a significant gap in the knowledge about energy-dissipating connections [3]. In another study, Tazarv and Saiidi experimentally investigated a new column-to-footing connection using UHPC. Their results suggested the UHPC-filled duct connection could be a suitable connection for high seismic regions and the plastic hinge will form in the column without any damage in that connection [4].

The Federal Highway Administration (FHWA) summarizes connection details for prefabricated bridge elements [5]. On-going research projects are being conducted to investigate durable prefabricated connections with good behavior [3]. Moreover, in seismic areas, connections between bridge elements are designed to resist the maximum force demand and cyclic effects. In case a bridge is designed for a large seismic event, it should exhibit ductile plastic hinging in the column based on AASHTO [6].

Several methods have been proposed for connecting prefabricated cap beams to the column. In most cases, the connections have been set in a way to transfer lateral seismic forces from the superstructure to the column. Among the methods are bar couplers, grouted ducts, pocket connections, member socket connections, hybrid connections, integral connections, and mechanical connections. In addition, some of these connections were explored in an actual bridge in the United States for connecting precast elements [7].

In seismic regions, the scheme of a column should eventually lead to forming plastic hinges and dissipating seismic forces. The ends of a typical column where it connects to the footing and pier caps, are the parts with the highest demand [6].

In one study, research performed quasi-static tests on precast piers with pre-stressing bars and steel tubes tied to each other. Specimens from the outcome illustrated the flexural failure mode in the plastic hinge region; furthermore, the failure pattern appeared similar to that of normal Reinforcement Concrete (RC) columns [8].

UHPC is a new class of concrete that includes a mix of Portland cement, fine sand, silica fume, ground quartz, superplasticizer accelerator, steel fibers, and water that provides properties such as high strength and durability. Graybeal investigated material properties of UHPC with different ages and curing situations. The results showed that UHPC results in significantly higher compressive strain and tensile strength in comparison with conventional concrete [9].

The objective of the research is to develop a new UHPC based connection between the cap beam and column which can potentially be used in ABC. The connection, made primarily with UHPC, is designed for a target plastic hinge location. It is expected that the outcomes of the study will aid in development of suitable connection details in high seismic zones.

2 DESCRIPTION OF THE PROPOSED ABC CONNECTION

UHPC is used to join the precast cap beam and the column in the field. It was anticipated that the major damage will occur near the connection. Therefore to prevent spreading of the damage to the cap beam, another layer of UHPC was used as a column capital. The use of UHPC in two locations guaranteed the formation of the plastic hinge at the desired location of the column. The use of UHPC in the splice region allows the development of reinforcing bars over a shorter length. Therefore, the length of the gap to be filled by UHPC in the field is relatively small. Details of the proposed connection are shown in Figure 1.

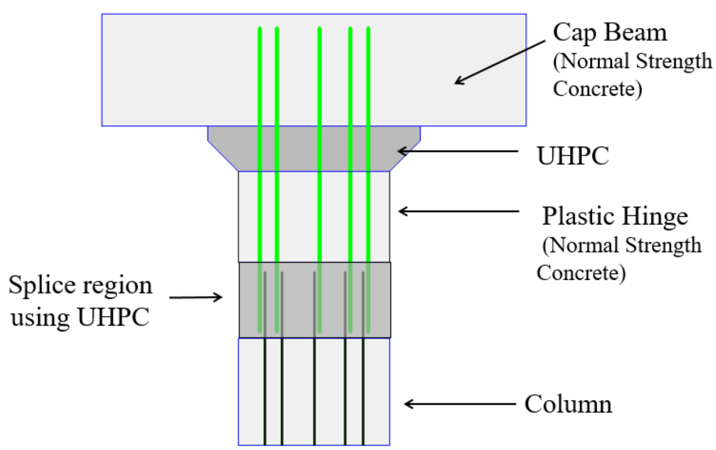

FIGURE 1 Details of the concept of the proposed connection.

The system consists of two parts; the cap beam and UHPC column capital and a part of column where the plastic hinge forms constitute one part, and the column is the second part. These two parts are connected in the field using UHPC. The connection is proposed as suitable for precast column-to-cap beam or footing connections located in high seismic zones. The merits of the proposed connection system are suitable tolerance for construction, containing damage within plastic hinge, and preventing rebar yield in the cap beam in the case of a large seismic event. The utility of this connection detail can be a feasible option for ABC located in high seismic zones. In order to investigate the real behavior of this connection, a set of experiments are considered.

3 EXPERIMENTAL PROGRAM

3.1 Description of the Test Specimen

A half scale connection between a precast column and cap beam was constructed and tested at Florida International University (FIU) under combined axial compression and reversed cyclic loading. The column was designed similar to a conventional cast-in-place column based on the AASHTO *LRFD Bridge Design Specifications* [6] and the Caltrans Seismic Design Criteria [10]. Figure 2 shows the specimen dimensions in detail. The column length was 10.5 ft (3.2 m) with a square cross section of 20x20 in. (508x508 mm). In order to limit, the potential localized failure in form of rebar yielding and concrete crushing to the cap beam, the section height at the top of the column was increased to 28 in. (711 mm) The column was reinforced longitudinally with 16-No. 6 bars and transversely with No. 4 stirrups at a 3-in. (76 mm) spacing. Clear cover of the rebars considered 3 in. (76 mm). Longitudinal and transverse reinforcement ratios were 1.95% and 1.00% respectively. The axial load of the column considered 200 kips (890 kN) which resulted in approximately 10% of the pure axial load capacity of the column section. The lap splice length of rebar in UHPC considered was 9 in., however, based on previous studies [11], only 5.25 in. (133 mm) could transfer the force between rebars.

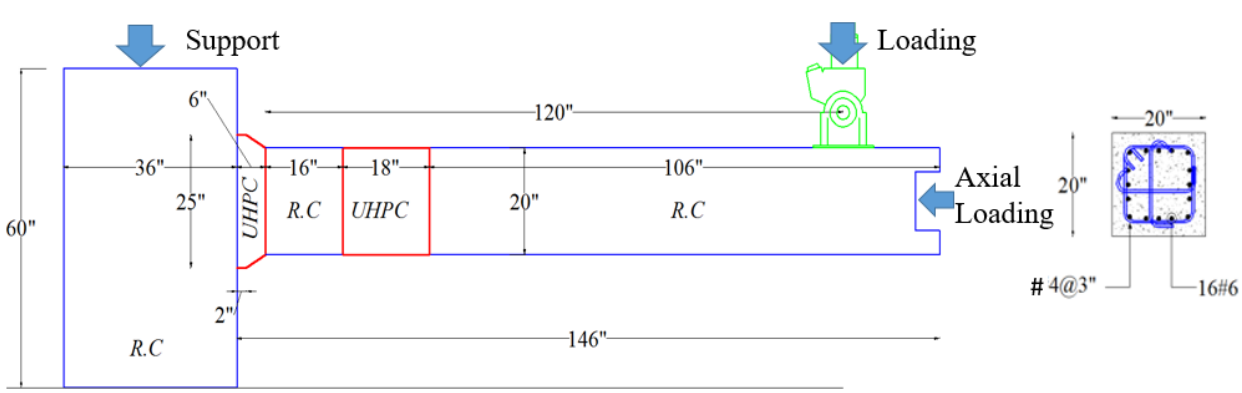

FIGURE 2 Specimen dimensions, unit: in. (NOTE: 1 in.= 25.4 mm).

3.2 **Construction of the Test Specimen**

The first step of the construction procedure was casting of the cap beam followed by casting top layer of UHPC and the plastic hinge area after one day. Joining column and the cap beam part with UHPC was carried out 28 days later. In order to minimize the cold joint issue between the Normal Concrete (NC) and UHPC, the surface of the normal concrete was roughened. Moreover, 1 in. (25 mm) of the top layer of UHPC was embedded in the support to prevent any local damage at the concrete support.

FIGURE 3 Specimen construction procedures.

3.3 **Test Setup and Loading Procedure**

A 240-kip (1068 kN) hydraulic actuator was used for displacement-controlled cyclic testing of the cantilever column. The loading setup made it more convenient to test the specimen with column placed horizontally. An axial load of 200 kips was applied using two hydraulic rams.

Initially, low displacement cycles with a rate of 0.5 in/min (13 mm/min) were applied to the column to estimate the idealized yield displacement of the column (Δ_y). The displacement rate was gradually increased to 2 in./min (50 mm/min).

The idealized bilinear load-displacement relationship is illustrated in Figure 4 in red. The linear portion of this line represents the elastic portion and the slope of the line represents the initial stiffness observed in initial loading cycles. The plastic portion is approximated by a horizontal line which had the best fit to the peak loads. The intersection of the linear and horizontal line represents the yielding displacement of the column, which corresponds to an abscissa value of about 1.8 in. (46mm). After obtaining Δ_y, the column was subjected to three cycles of $2\Delta_y$, $3\Delta_y$, $4\Delta_y$ and $5\Delta_y$ displacements. At the end of each cycle, the displacement was paused to observe the damages and map the cracks.

To monitor the behavior of the connection, the specimen was instrumented using strain gauges, potentiometers, pressure transducers, string pots, and load cell. Potentiometers were utilized to calculate column curvature at four levels in plastic hinge area. The maximum support movement during cyclic loading was 0.03 in. (0.8 mm). The compressive strength of conventional concrete for the column and plastic hinge was 7.1 ksi (49 MPa) and 6.4 ksi (44 MPa) respectively. The compressive strength of UHPC was 21.7 ksi (150 MPa). The measured yield and ultimate strength of the longitudinal reinforcement used was 68.1 ksi (470 MPa) and 112.2 ksi (774 MPa) respectively.

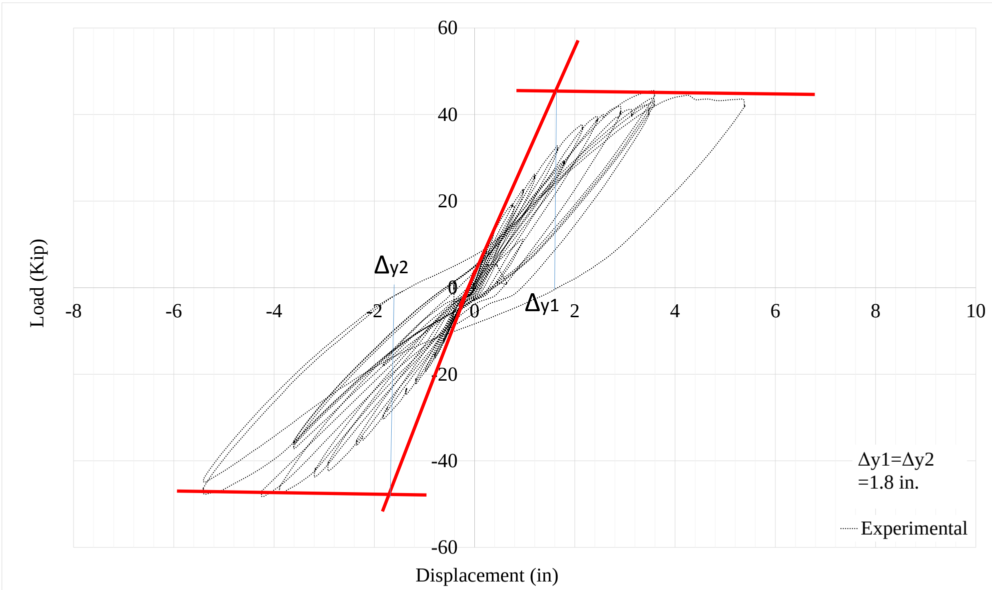

FIGURE 4 Experimental definition of yield displacement (NOTE: 1 in.= 25.4 mm and 1 kip= 4.4 kN).

3.4 Experimental results

3.4.1 Observed Damage

The connection damage in the plastic hinge location is shown in Figure 5 at different displacement levels. The First cracks formed in the plastic hinge area followed by development of cracks in the column and then in the cap beam face. The spalling of the cover concrete was first observed at $3\Delta_y$. A small portion of the cap beam around the column failed, which can be attributed to cold joint effect. Damage was limited to the plastic hinge zone, which consisted of normal strength concrete. No spalling of concrete was observed in UHPC portions of the test specimen and observed cracking in UHPC portions of the test specimen was very limited. The damage in the column was primarily located in the region of normal strength concrete. Bond failure between normal concrete and UHPC was not observed, which indicated appropriate bond between the materials.

FIGURE 5 Plastic hinge damage at different displacement levels.

3.4.2 Mode of Failure

Longitudinal bar buckling in the plastic hinge region of the test specimen followed by bar fracture was the mode of failure that occurred at $5\Delta_y$. During the first cycle of $5\Delta_y$ a loud sound was heard, which was attributed to the first rebar rupture resulting in a load drop, and the testing was immediately stopped. The failure was marked by the rupture of three more longitudinal bars at the 4^{th} cycle of $5\Delta_y$ with strength degradation of more than 25%. Three of the fractured rebars were located at the bottom and one in the top face of the column. Fracture of all rebars was located around the middle of the plastic hinge. Spalling of concrete at $5\Delta_y$ developed inside the core of the concrete.

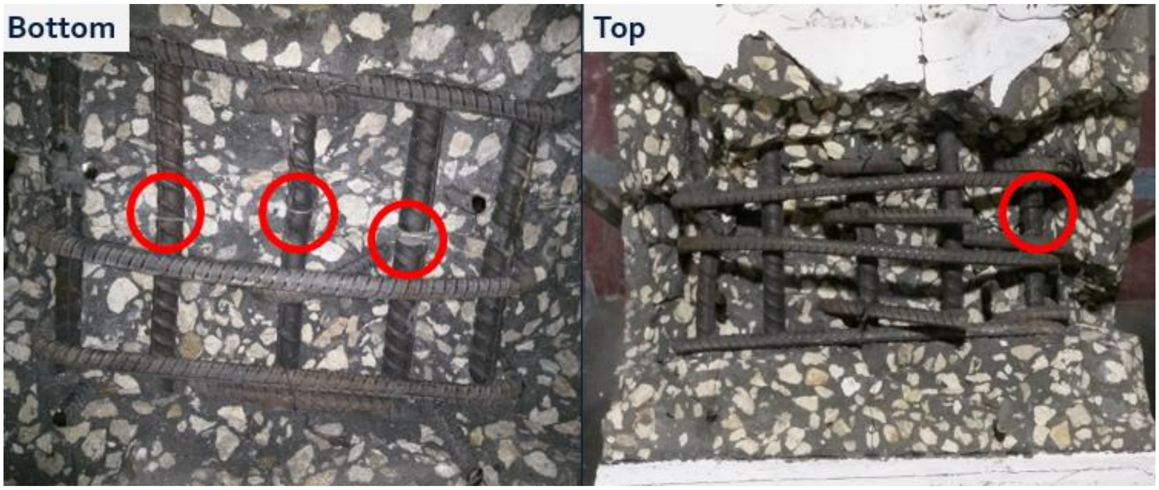

FIGURE 6 Fracture in bars located in the middle of the plastic hinge.

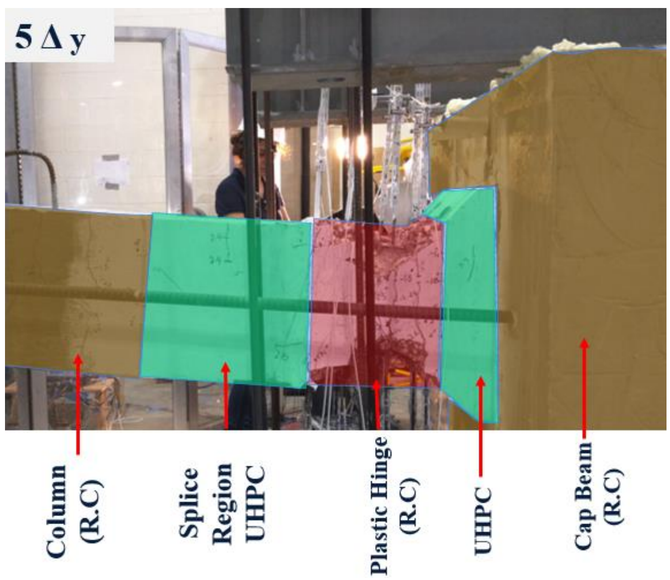

FIGURE 7 Failure mode and location of the plastic hinge.

3.4.3 Moment–Displacement Relationship

As mentioned previously, to calculate the idealized yield displacement of the column (Δ_y), several cycles were applied to the specimen. Based on first cycles, Δ_y was measured 1.8 in. (46 mm). The moment–displacement hysteretic curves for the specimen are shown in Figure 8. The measured moment includes moment due to self-weight of the column including additional moments caused due to P-Δ effect. Based on this figure, the maximum capacity of the system was 6500 kip.in. (734 kN.m). However, the nominal capacity of the column considering 200 kips (890 kN) axial load, was estimated 5840 kip.in. (660 kN.m).

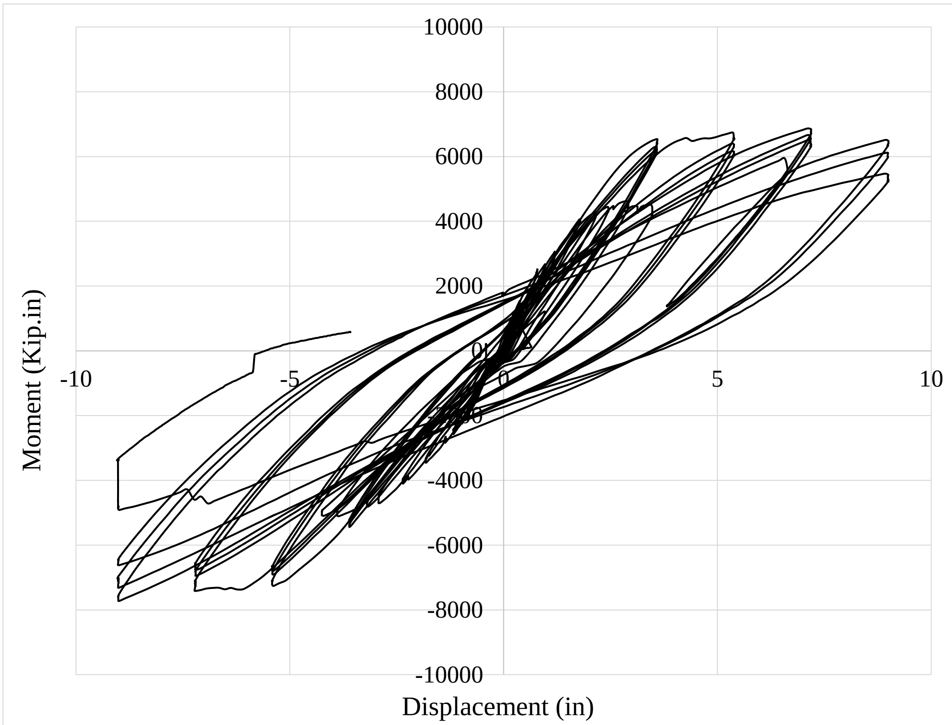

FIGURE 8 Experimental moment-displacement result.

3.4.4 Measured Curvatures

The specimen was instrumented with two rows of 8 potentiometers to measure the curvature of the column. The potentiometers were mounted on steel rods that passed through the column at four different distances from the cap beam face. The maximum curvature profile at each displacement versus the column height is presented in Figure 9. Results show that the plastic hinge was placed at the desired location and curvature had uniform distribution in 2 and $3\Delta_y$ displacement level. At $4\Delta_y$ displacement level, the maximum curvature was measured between the second and third rows of the potentiometers, which were located approximately at the middle of the plastic hinge zone. The potentiometers stopped working around $5\Delta_y$ and further measurement of curvature was not possible.

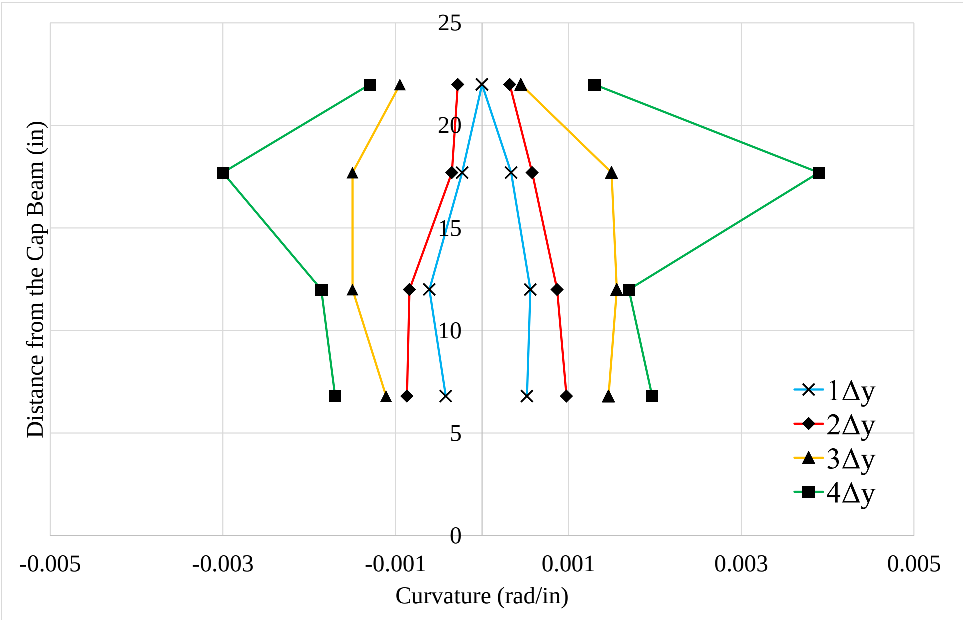

FIGURE 9 Plastic hinge curvature profiles for each displacement.

3.4.5 Measured Strains

The column longitudinal bars were instrumented with six strain gauges at the middle of the plastic hinge. The peak tensile strain profiles of rebars were measured at different displacement levels. No yielding of the shear reinforcement was observed even at $5\Delta_y$. By losing strain gauges at $5\Delta_y$, measuring strains of the bars was not feasible (Figure 10).

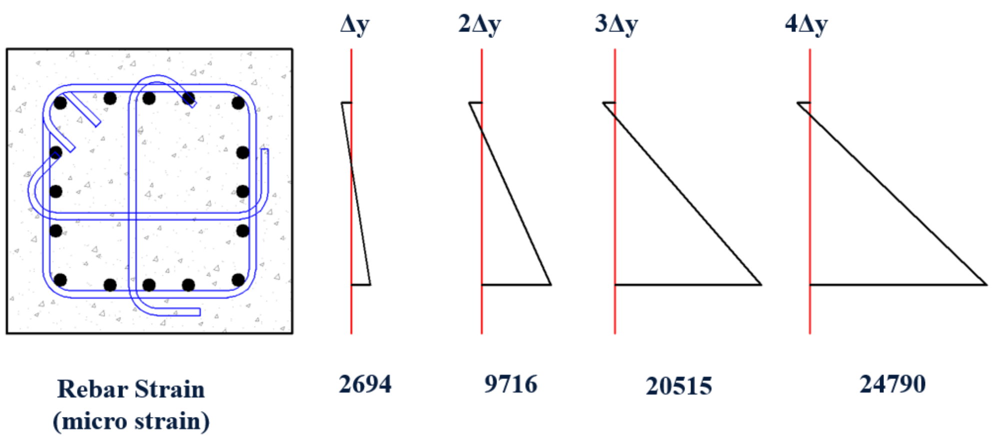

FIGURE 10 Peak tensile strain profiles of bars measured in the middle plastic hinge.

4 NUMERICAL INVESTIGATION

The numerical analyses were performed using Abaqus finite element (FE) software. Dimensionally, the simulated model was the same as the test specimen. An eight-node solid element was utilized for both UHPC and concrete, and rebars were modeled as a beam element. Material strength was based on the measured compressive strength of concrete and UHPC. A monotonic displacement control loading was applied to the model. Figure 11 shows the numerical model.

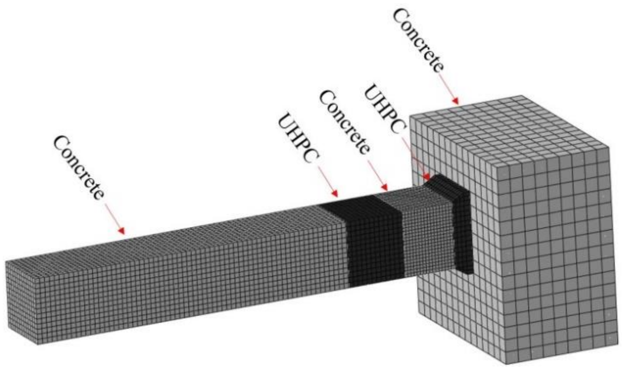

FIGURE 11 Column numerical model details.

4.1 Numerical Simulation Results

The measured hysteretic curves and calculated force displacement of the specimen are shown in Figure 12. As indicated in the figure, the initial stiffness of the system in FE model is higher than the stiffness obtained from the experimental study. This difference can be attributed to the material properties and boundary conditions such as support rigidity defined in FE model. Despite the difference, a good correlation of the maximum load capacity was observed between FE model and experiment.

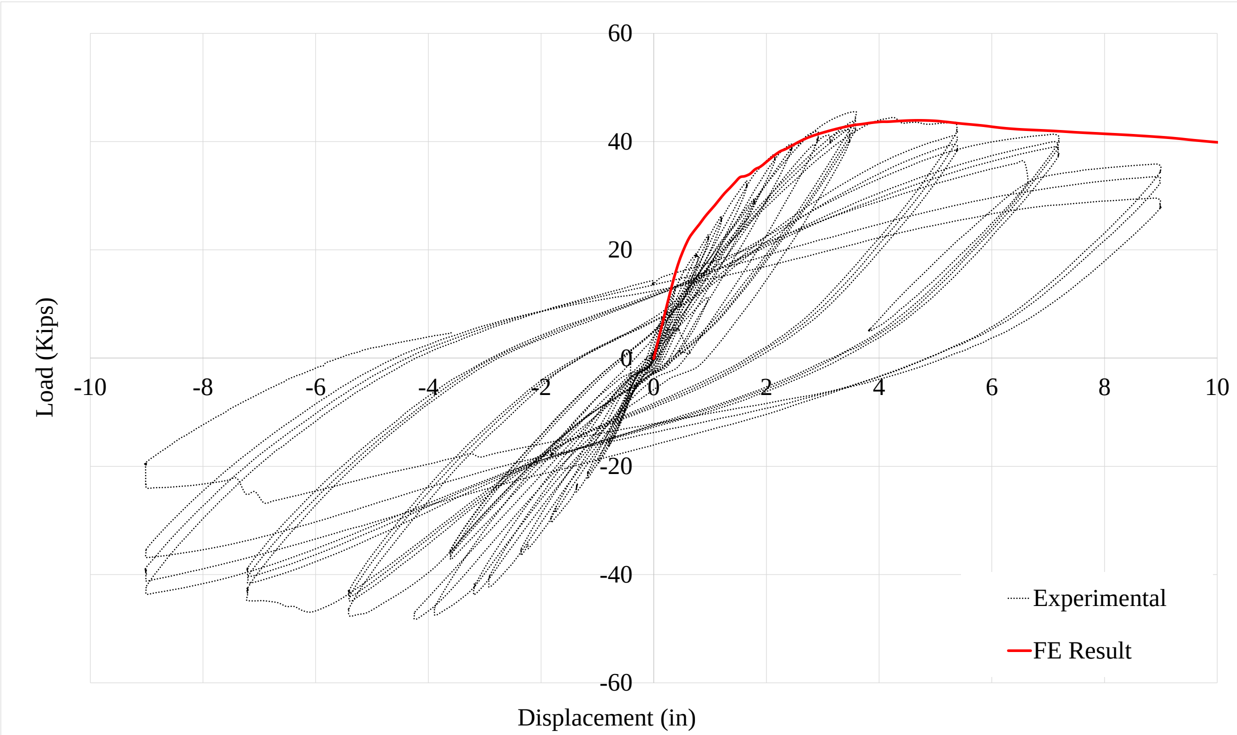

FIGURE 12 Measured hysteretic curves and calculated force displacement.

The numerical results for the crack formation and bars stresses are depicted in Figure 13. The model was able to reproduce the column behavior with reasonable accuracy. As illustrated, the results of FE analysis could predict crack formation, critical sections, and yielding development in rebars.

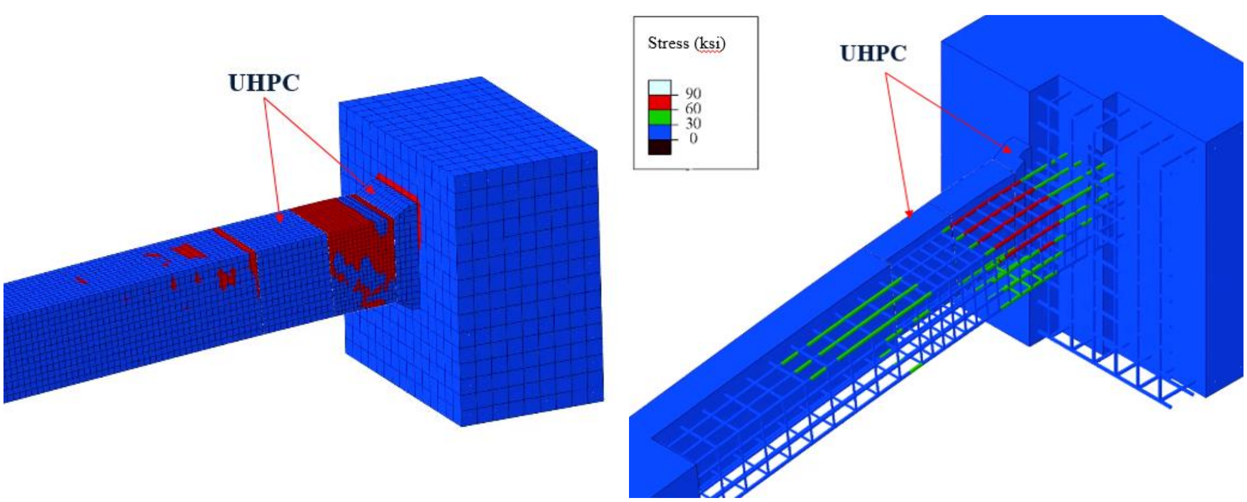

FIGURE 13 Crack formations and rebar stresses in the numerical model.

4.2 Axial Load Effect

The FE model was further used to investigate the effect of axial load on the connection behavior. The FE model was subjected to different axial load from 10 to 25 percentage of the pure axial load capacity of the column. Results show that increasing the axial load increases the ultimate capacity of the column as well as connection. Increased axial load results in decreased ductility of the system.

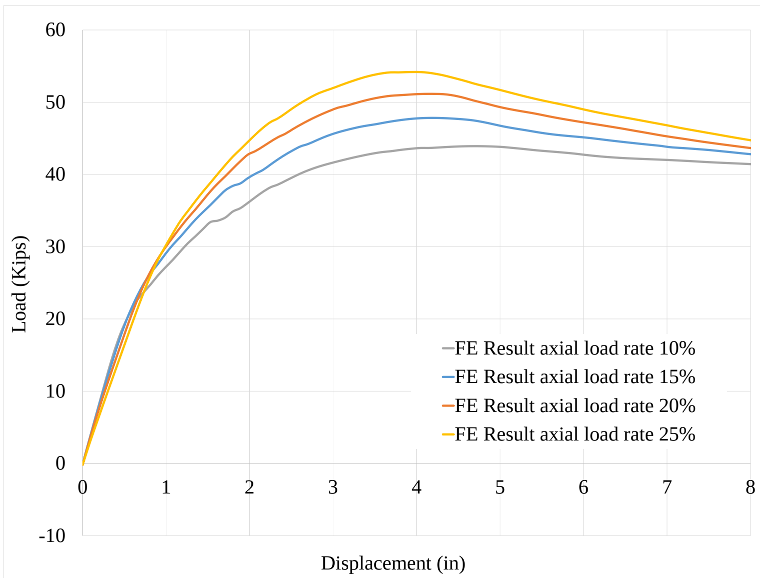

FIGURE 14 Effect of axial load on the behavior of the connection in the numerical model.

5 CONCLUSIONS

A layer of UHPC was used to connect a half-scaled precast concrete column to the cap beam. In order to force forming plastic hinge in the desired location in the column, another layer of UHPC was added below the cap beam. A cyclic testing on cantilever configuration of the column was carried out. Results of the test led to the following conclusions:

1. This connection is proposed as a suitable precast column-to-cap beam or footing connection for high seismic areas, which can potentially satisfy constructability requirements and large construction tolerances.

2. No UHPC part damage such as bar pullout or crushing was observed. Therefore, in comparison with conventional concrete, short lap splice of bars in UHPC can transfer forces between spliced bars, which leads to a decrease in casting volume and saves time in the field. Because

of the high workability of the UHPC and available tolerance of bars, attaching two precast parts can be accelerated.

3. Based on the curvature of the column, the plastic hinge is located in the desired location as guaranteed. The UHPC layer prevents development of yielding into the cap beam. However, limiting plastic hinge in short length can lead to less ductility.

4. Good correlation was found between the experimental results and FE analysis. Therefore, the model developed in this research can be used for more accurate follow-up studies of the proposed system responses in the future. It is expected that results of FE analysis will enable prediction of crack formation, critical sections, and yielding development in rebars.

REFERENCES

1. Culmo, Michael P. Accelerated Bridge Construction-Experience in Design. *Fabrication and Erection of Prefabricated Bridge Elements and Systems*, 2011.

2. Restrepo, J. I., M. J. Tobolski, and E. E. Matsumoto. "NCHRP Report 681: Development of a Precast Bent Cap System for Seismic Regions." *Transportation Research Board of the National Academies*, Washington, DC (2011)

3. Marsh ML. Application of Accelerated Bridge Construction ConnePctions in Moderate-to-High Seismic Regions. In *Transportation Research Board 90th Annual Meeting*. CD-ROM. National Academies Press, Washington D.C., 2011.

4. Tazarv M. and Saiidi MS. UHPC-filled Duct Connections for Accelerated Bridge Construction of RC Columns in High Seismic Zones. *Eng Struct*. No.99, pp.413-22, 2015.

5. Culmo, Michael P. Connection Details for Prefabricated Bridge Elements and Systems. *No. FHWA-IF-09-010,* 2009.

6. AASHTO Guide Specifications for LRFD Seismic Bridge Design. Washington, DC: *American Association of State Highway and Transportation Officials;* 2012.

7. Khaleghi, Bijan, Eric Schultz, Stephen Seguirant, Lee Marsh, Olafur Haraldsson, Marc Eberhard, and John Stanton. Accelerated Bridge Construction in Washington State. *PCI journal* 57, No. 4, pp.34-49, 2012:

8. Shim, Chang Su, Chul-Hun Chung, and Hyun Ho Kim. Experimental Evaluation of Seismic Performance of Precast Segmental Bridge Piers with a Circular Solid Section. *Engineering Structures* 30, No. 12, pp. 3782-3792, 2008.

9. Graybeal BA. Material Property Characterization of Ultra-high Performance Concrete. 2006.

10. Caltrans. Seismic Design Criteria (SDC). *CA: California Department of Transportation, Sacramento*; 2010.

11. Graybeal, Ben. Design and Construction of Field-Cast UHPC Connections. *No. FHWA-HRT-14-084*. 2014.