Alternative ABC Connection for Connecting Precast Column to Cap Beam Using UHPC

Submitted by
Mohamadreza Shafieifar

FINAL REPORT

Department of Civil and Environmental Engineering
Florida International University
Miami, Florida

Submitted to
Atorod Azizinamini
Director, ABC-UTC
June 2018
ABC-UTC

ABSTRACT

Accelerated Bridge Construction (ABC) is a paradigm change in delivery of bridges. ABC minimizes the traffic interruption, enhances safety to public and workers by significantly reducing onsite construction activities and results in longer lasting bridges. The use of precast elements is gaining attention owing to inherent benefits of accelerated construction. Designing an economical connection is one of the main concern for these structures. New improved materials such as Ultra High Performance Concrete (UHPC) with superior characteristics can provide solutions for joining precast concrete elements.

In this paper two types of column to cap beam connection using UHPC are proposed for seismic and non-seismic regions. Among the merits of the proposed details, large tolerances in construction and simplicity of the connection can be highlighted which facilitates and accelerates the on-site construction time.

The experimental program was carried out to evaluate the performance and structural behavior of the proposed connections. Four specimens were subjected to constant axial compressive loads and cyclic lateral loading. Results of the experiment showed that the displacement ductility of the specimens, incorporating suggested details, demonstrated adequate levels of displacement ductility. More importantly, the proposed connections prevented the damage into capacity protected element- in this case the cap beam. Analytical and nonlinear finite element analysis on the specimens were carried out to better comprehend the behavior of the proposed connections.

Keywords: Precast Column-to-cap Beam Connection, Innovative Connection, Accelerated Bridge Construction, UHPC, Seismic Performance

INTRODUCTION

Accelerated construction is in high demand in the bridge construction industry to minimize on-site construction time, reduce traffic interruption, and increase work zone safety. 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 elements. These elements require joints between elements and there are concerns regarding the durability and structural behavior of the connections [1]. The behavior of the structure with precast concrete elements is ruled by the connections and the performance of the connection is influenced by various parameters. Therefore, simple connection detail results in predictable structural behavior. A simple detail can increase construction speed and precision of fabrication.

Various studies on connection between precast concrete elements in the bridges have been carried out [2-6]. The Federal Highway Administration (FHWA) summarizes common connection details for prefabricated bridge elements [7]. The column to cap beam or footing connection plays a critical role in the behavior of the bridge in seismic regions. Several details for joining column to cap beam have been proposed, but none of these methods have entirely solved construction issues. Among the methods are pocket, bar couplers, member socket, hybrid, integral, and mechanical connections.

In light of the challenges inherent in connecting precast concrete elements, exploring new materials such as Ultra High Performance Concrete (UHPC) can be beneficial. UHPC is a cementitious material with a typical compressive strength exceeding 21 ksi (150 MPa), improved toughness and increased damage tolerance making it well-suited for use in heavily loaded structural components [8]. UHPC 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. UHPC exhibits considerably large strain and tension capacity, compared to the regular concrete, which can be employed for ductile structures in seismic regions. Superior characteristics of UHPC, such as the short development length and the ability to gain strength during a short time can assist ABC construction. Several studies are available for UHPC material properties and its application in bridges [9-11].

Ductility of a structure is expressed as the capacity of the structure to absorb and dissipate energy during earthquake, by slight loss of strength and serviceability. Formation of plastic hinge in the columns can provide this inelastic behavior of the bridges. The properly detailed and closely spaced transverse reinforcement in conventional reinforced concrete columns can ensure ductile behavior during earthquakes. Transverse reinforcement plays an important role in confining the concrete within the core region of columns, resulting in improvements of strength and ductility. Consequently, to ensure ductile behavior, modern design codes impose stringent requirements for the detailing of transverse reinforcement in columns. According to the current seismic design methods, forming the plastic hinge in the column should provide the ductility to the bridge in seismic regions.

The objective of this research is to develop a new UHPC based connection between the cap beam and column which can potentially be used in ABC application for both seismic and non-seismic regions. The connection, made primarily with UHPC, is designed for a target plastic hinge location. The feasibility study of this project was done in [12]. In the current research, four scaled columns with the proposed detail were tested under cyclic loading and the structural behavior of the specimens was highlighted. The results of the tests and numerical studies will be used in future for development of design guidelines.

DESCRIPTION OF THE PROPOSED ABC CONNECTION

In the proposed connection, UHPC is used to join the precast elements in the field. Two details are proposed: seismic and non-seismic details. In the seismic detail, two layer of UHPC are employed. As shown in Figure 1 (a), in the absence of top layer of UHPC, it was anticipated that the major damage will occur near the cap beam. 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 layers 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 short length. Therefore, the length of the gap to be filled by UHPC in the field is relatively small. For the non-seismic detail (Figure 1 (b)), the cap beam and the column are simply joined with a layer of UHPC.

For the seismic detail, the system consists of two parts. First part includes the cap beam, UHPC column capital and a part of column where the plastic hinge forms while the second part is the column. These two parts will be connected in the field using UHPC. The connection is proposed for precast column to cap beam or footing connections located in high seismic zones. Gaining the strength of UHPC over a short time and large tolerances for construction of the proposed details, can provide a rapid method of bridge construction. The merits of the proposed connection system are large tolerance for construction, confinement of damage within plastic hinge, and prevention of rebar yield in the cap beam in the case of a large seismic event. In order to investigate the structural behavior of this connection, a set of experiments were conducted.

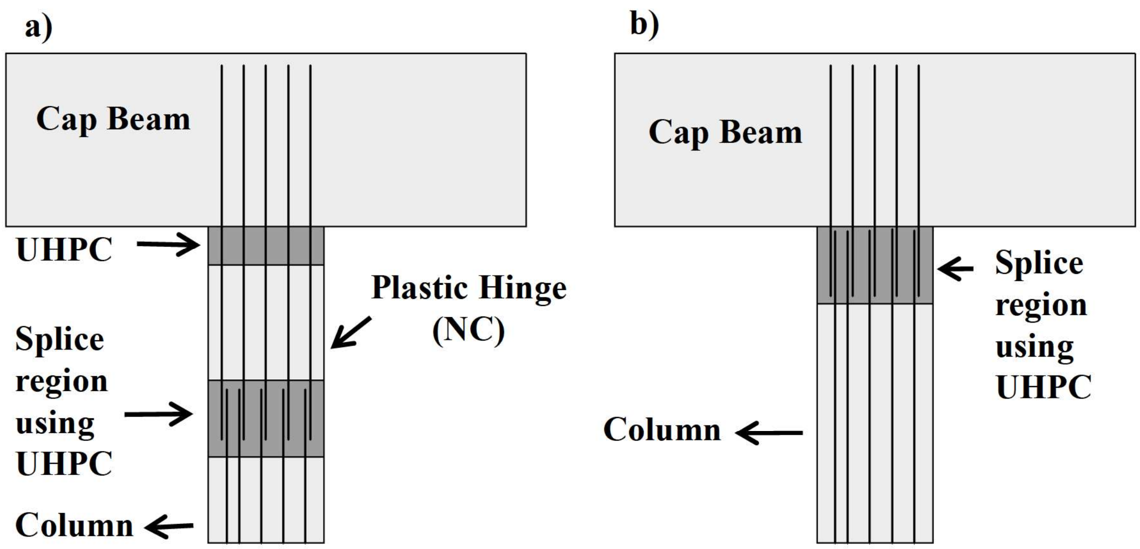

FIGURE 1 Details of the concept of the proposed connection a) Seismic detail b) Non-Seismic detail.

EXPERIMENTAL PROGRAM

In this study, four scaled connections between a precast column and cap beam were constructed and tested under combined axial and reversed cyclic loading. Three seismic and one non-seismic details were investigated. The column and cap beam were designed based on common provisions for seismic regions [13-15] with the ductility ratio of 5. The length of the splice region were large enough to ensure that slip failure in this region would not occur.

Description of the Test Specimens

Four column to cap beam connection were designed for testing under lateral loading and constant axial load, designated as specimen S-2.5-10, S-4-10, S-2.5-20 and NS-2.5-10. Details of test variables and axial load of all specimens are given in Table 1. S-2.5-10, S-4-10, S-2.5-20 had a seismic proposed detail with different axial load rate and transverse rebar ratio in splice and plastic hinge zone. The axial load varied from 10% to 20% of its pure axial capacity. In addition, specimen NS-2.5-10 was designed with non-seismic detail. Previous researches indicates that sufficient steel fibers in concrete may increase shear capacity, section confinement and provide more ductility [16]. Therefore, to study the confinement effect of UHPC, different transverse reinforcement ratios in the splice region were used.

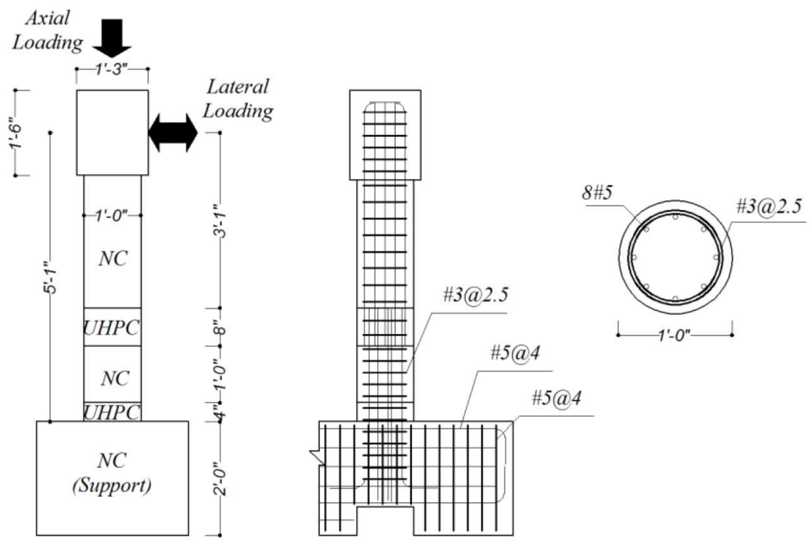

FIGURE 2 Specimens S-2.5-10, S-4-10, S-2.5-20 typical dimensions and reinforcement details.

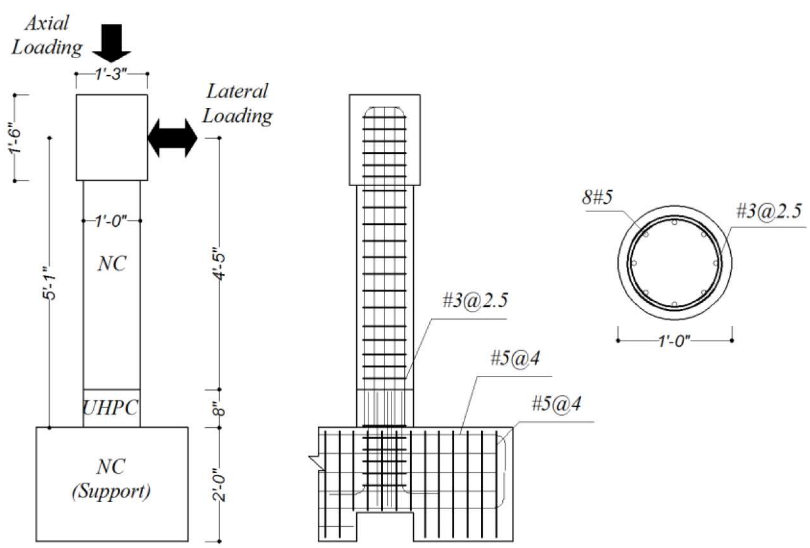

FIGURE 3 Specimen NS-2.5-10 (Non-Seismic detail) dimensions and reinforcement details (No stirrups at splice region).

Table 1 Details of the specimen.

Specimen ID	Geometry detail	Transverse Reinforcement detail	Axial Load Ratio
S-2.5-10 (Reference)	As shown in Fig 2 (seismic detail)	#3@2.5 in. in plastic hinge and splice region	10%
S-4-10	As shown in Fig 2 (seismic detail)	#3@4 in. in plastic hinge and no strips splice region	10%
S-2.5-20	As shown in Fig 2 (seismic detail)	#3@2.5 in. in plastic hinge and one stirrups at splice region	20%
NS-2.5-10	As shown in Fig 3 (non-seismic detail)	#3@2.5 in. in plastic hinge and no stirrups at splice region	10%

Figure 2-3 shows the specimens dimensions in detail. The column height was 61 in. (155 cm) with a circular cross-section having diameter of 12 in. (30.5 cm). The column was typically reinforced longitudinally with 8-No. 5 rebars (M16). Longitudinal reinforcement ratio was 2.19% for all specimens. Clear cover of the rebars was 1.5 in. (38 mm). The axial load of the reference specimen (S-2.5-10) was 56 kips (249 kN) which resulted in approximately 10% of the pure axial load capacity of the column section. The lap splice length of rebar in UHPC was 8 in. (200 mm), equals to 13 times of the rebar diameter; although, previous studies have shown that shorter lap splice lengths can be used if adequate cover concrete is provided [17].

Construction of the Test Specimens

For the seismic details, the first step of the construction procedure was casting of the cap beam (support) followed by casting the first layer of UHPC and plastic hinge part. Casting splice region and column part was carried out 11 days later. In order to minimize the cold joint between the Normal Concrete (NC) and UHPC, the surface of the concrete was textured. Moreover, around 1 in. (25 mm) of the first layer of UHPC was embedded in the support to prevent any local damage at the concrete support.

FIGURE 4 Specimen construction procedures a) Formwork of the support, b) Casting support, c) Casting first layer of UHPC, d) Casting plastic hinge, e) Reinforcement of the column, f) Casting splice region with UHPC and g) Casting column with N.C.

Test Setup and Loading Procedure

The experimental test was carried out six months after casting the specimens. Quasi-static test was considered to evaluate the cyclic performance of the connection. A 150-kip (490 kN) hydraulic ram was used for cyclic testing of the cantilever column. A constant predetermined axial load was applied using two hydraulic rams. Initially, low displacement cycles were applied to the column to estimate the idealized yield displacement of the column (Δ_y).

FIGURE 5 Loading setup overview.

The yielding displacement (Δ_y) was defined by assuming the bilinear model, equivalent elasto-plastic system with the same elastic stiffness and ultimate load as the real system, by following the procedure suggested by R. Park [18]. After obtaining Δ_y, the column was subjected to three cycles of $2\Delta_y$, $3\Delta_y$, $4\Delta_y$ and etc. At the end of each cycle, the displacement was paused to observe the damages and trace the cracks. Yielding displacement of the seismic specimens (S-2.5-10, S-4-10 and S-2.5-20) was 0.65 in. (17 mm). The yielding displacement of non-seismic specimen (NS-2.5-10) was calculated 0.8 in. (20 mm). To evaluate the behavior of the specimens, the test setup was instrumented with string pots, load cells, strain gauges, potentiometers and pressure transducers. To calculate the curvature of the column, potentiometers were employed at four levels in plastic hinge regions. Cylinder test was used to measure the compressive strength of normal concrete and UHPC a day after the experimental test. The actual material properties is reported in Table 2.

Table 2 Actual material properties of the material.

Concrete Material	
Concrete of Supports and Plastic hinge location	f_c=7.1 ksi (49 MPa)
Concrete of the columns	f_c=6.4 ksi (44 MPa)
UHPC of the first layer	f_c= 20.1 ksi (139 MPa)
UHPC of the splice region	f_c= 24.9 ksi (172 MPa)
Steel Bars	
f_y= 68 ksi (468 MPa)	f_u= 113 ksi (780 MPa)

EXPERIMENTAL RESULTS

Observed Damage

The development of cracks and crushing of the concrete in the plastic hinge region at different displacement levels are shown in Figure 6 for each specimen. For all seismic details (S-2.5-10, S-4-10 and S-2.5-20), the first cracks formed in the plastic hinge area followed by development of limited cracks in the column. For these specimens the spalling of the cover concrete was first observed at $2\Delta_y$ and damage were limited to the plastic hinge zone, which consisted of normal strength concrete. No spalling or cracking was observed in UHPC portions of all specimens. No bond failure between normal concrete and UHPC was observed, which indicates the sufficient bond between UHPC and normal concrete. The spalling of the cover concrete of non-seismic specimen (NS-2.5-10) was first observed at the first cycle of $2\Delta_y$. Limited concrete cracking was found on the surface of the cap beam of the specimen.

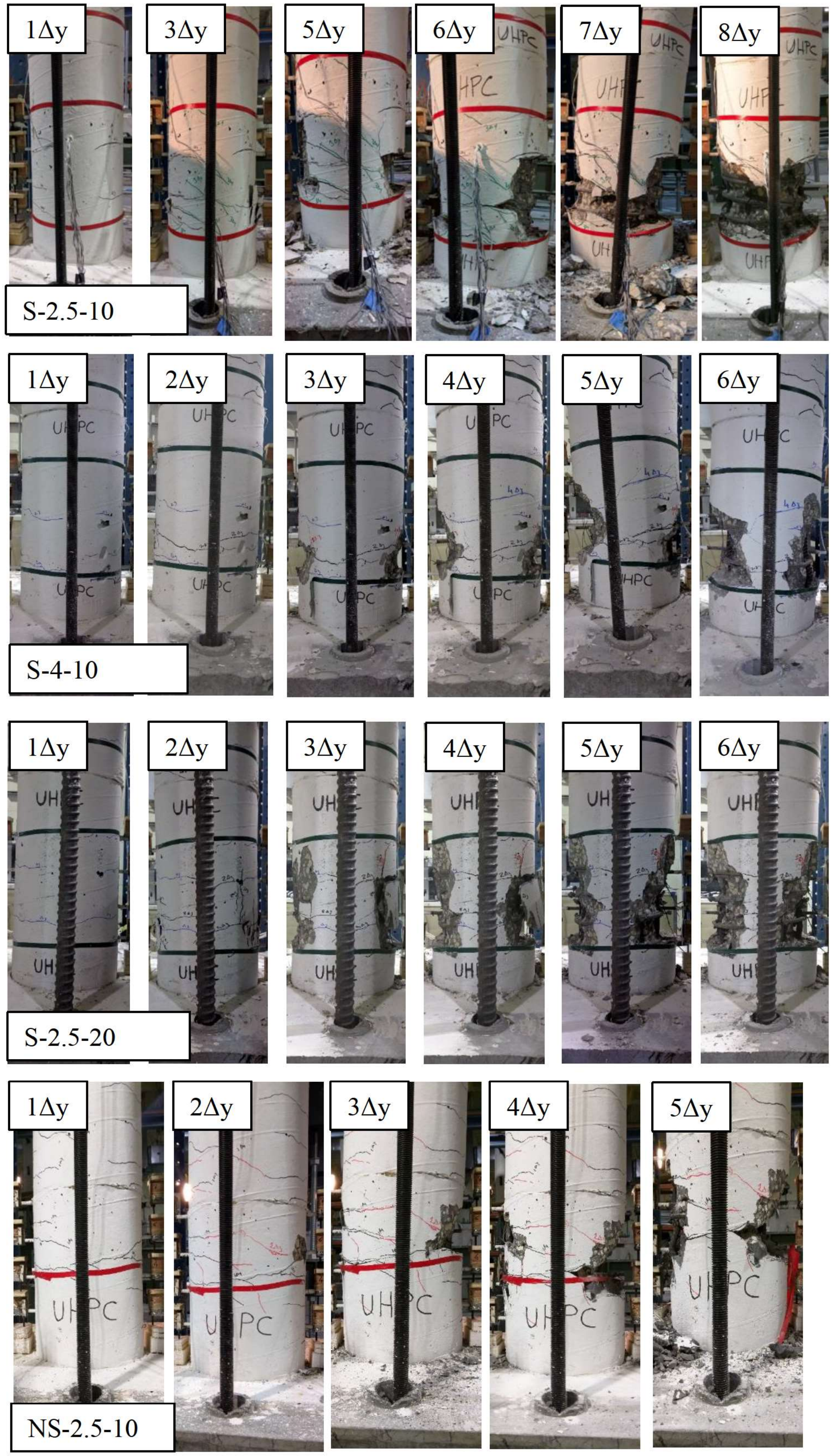

FIGURE 6 Plastic hinge damage at different displacement levels for all specimens.

Mode of Failure

Specimens failed when one of the longitudinal bars ruptured after development of buckling between stirrups due to spalling of the concrete cover. The concrete cover in the plastic hinge zone spalled off and then the core concrete crushed. Fracture of the rebar was located around 2 inches above the UHPC layer for seismic details. The displacement ductility of the specimens are shown in Table 3 indicating their performance in seismic zones based on common seismic provisions. Results of the test show that by increasing the stirrup spacing or axial load the ductility will decrease. By comparing reference specimen (S-2.5-10) with specimens S-4-2.5 (with less transverse reinforcement ratio) and S-2.5-20 (higher axial load), it was concluded that the ductility of the specimens was most influenced by the space between the stirrups in plastic hinge region.

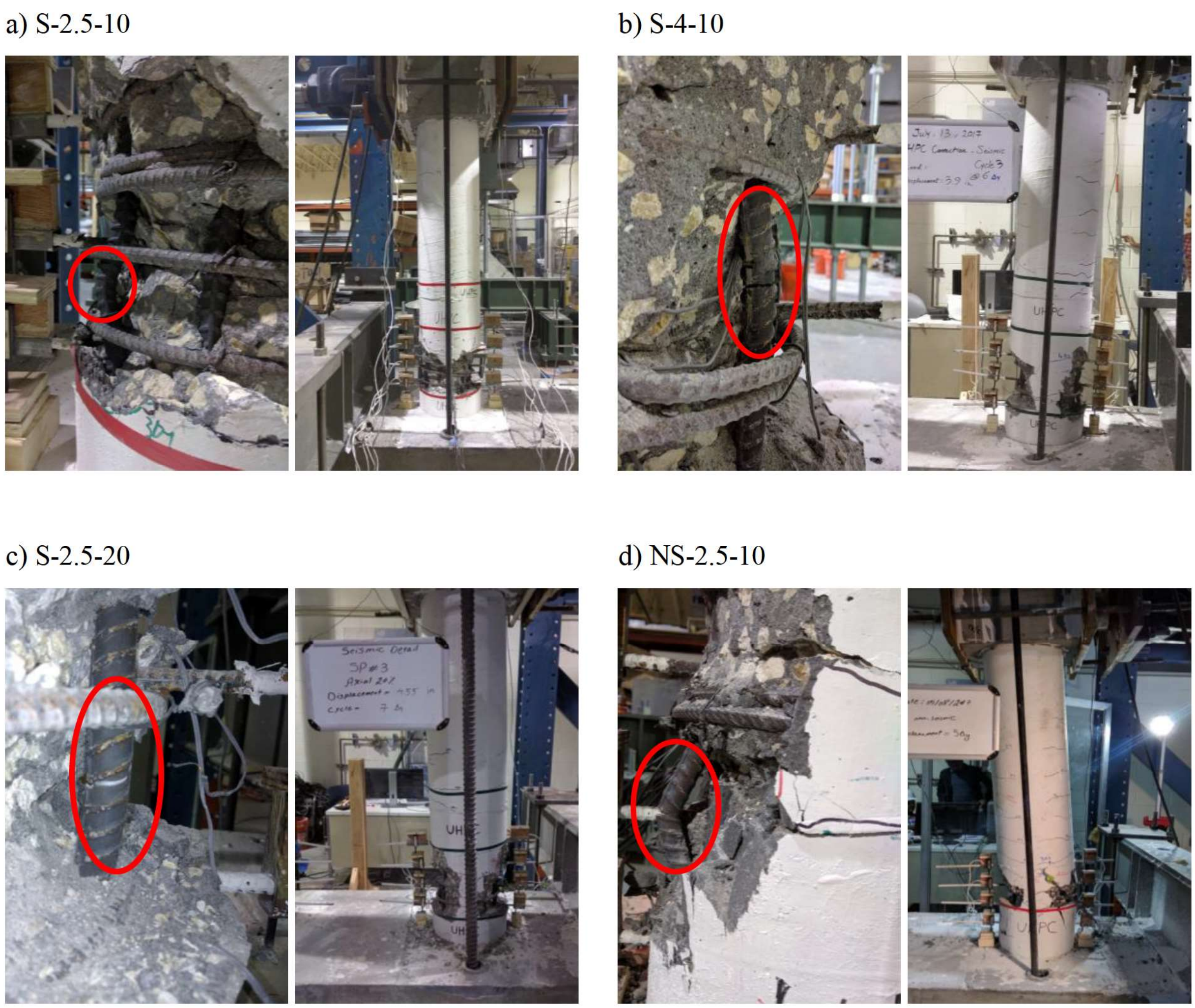

FIGURE 7 Failure pattern, longitudinal rebar fracture and location of the plastic hinge in the specimens.

Table 3 Ductility and maximum drift of the specimens.

Specimen ID	Maximum drift	Displacement ductility
S-2.5-10	8.5 %	8
S-4-10	5.3 %	5
S-2.5-20	6.4%	6
NS-2.5-10	6.5%	5

Load Displacement Curve

Table 2 shows the maximum displacement for each specimens. Even though the non-seismic detail was not designed for seismic loading, the specimen was subjected to the cyclic loading to show its capability for seismic area as well. Compared with the monotonic loading, cyclic loading can provide severe condition. Figure 8 shows the hysteresis load deflection curve of the specimens. In all specimens rebar cages were shifted approximately half inches during the casting and asymmetry of the curves can be attributed to the casting imperfection of the specimen. The lateral load decreased gradually after the peak load, indicating the spall off of cover concrete and p-Δ effects. Lateral load reduction after peak load of specimen S-4-10 (with a larger distance between stirrups) and non-seismic detail (NS-2.5-10), were more than other specimens. Specimens with higher ductility ratio had more capacity to absorb and dissipate energy under extreme loads, such as earthquakes, by showing a limited loss of serviceability. However, it should be noted that the residual drifts of the specimen S-2.5-20 (with 20% axial load) was smaller than those of columns with 10% axial load.

Due to the applied axial load, specimen S-2.5-20 had a higher load capacity compared to other seismic details. It is also worth to notice, as the distance between load and critical section in the non-seismic detail (NS-2.5-10) was less than other specimens, the specimen could carry out a higher level of lateral load.

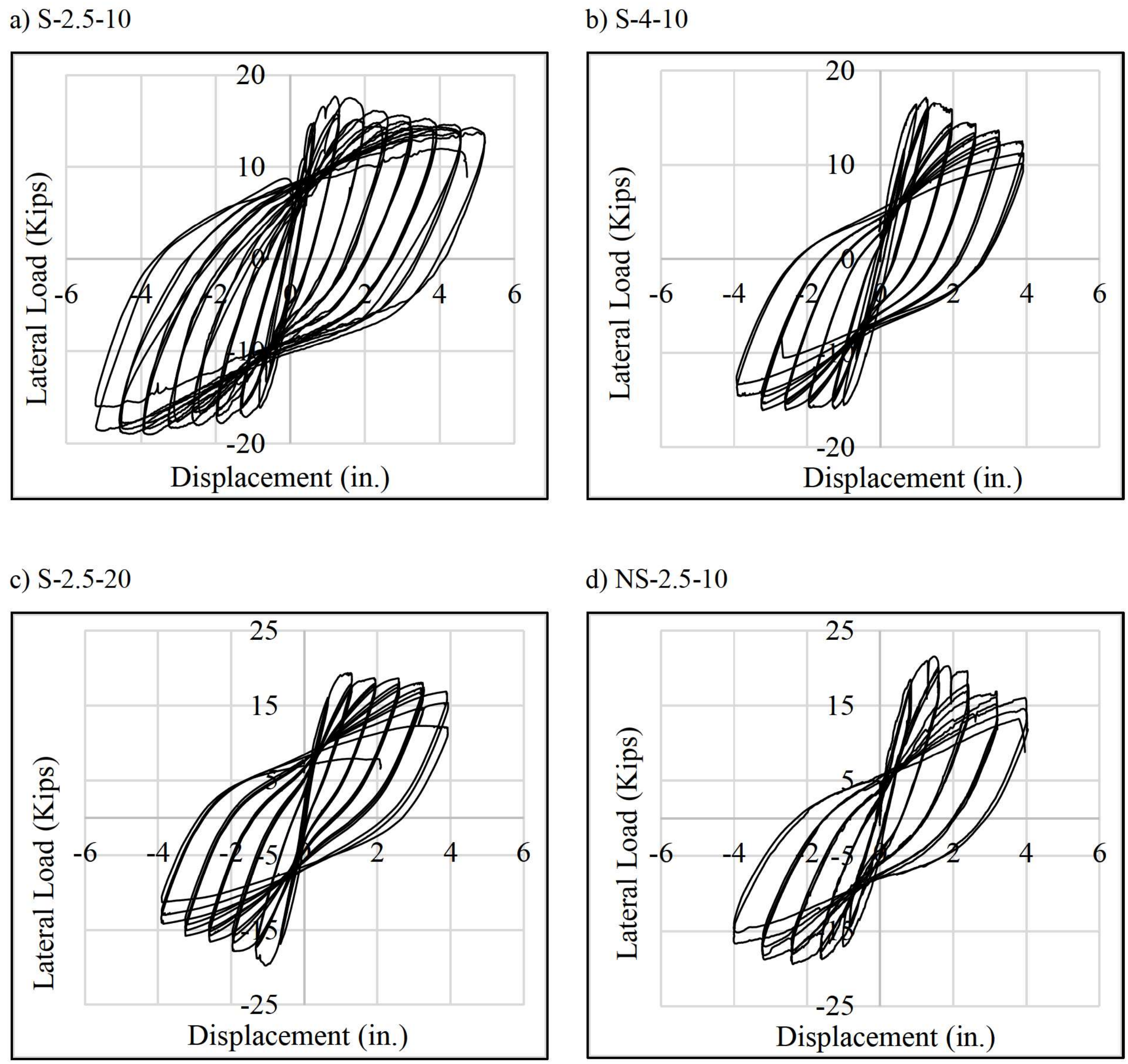

FIGURE 8 Hysteric load deflection curve of the specimens.

Measured Curvatures

The specimens were instrumented with 8 potentiometers to measure the curvature of the column. The potentiometers were mounted on steel bars that passed through the column at four different distances (4 in. space) from the cap beam face. Some potentiometers stopped working at high displacement levels and further measurement of curvature was not possible. The maximum average curvature profile at each displacement level versus the column height is shown in Figure 9. Results show that in the seismic details, the plastic hinge was placed at the desired location (between two layers of UHPC). For the non-seismic detail (NS-2.5-10) the plastic hinge formed above the splice region and the results show that the use of UHPC in splice region prevented development of plastic hinge to the cap beam as well. This shows the capability of this specimen for seismic area as well.

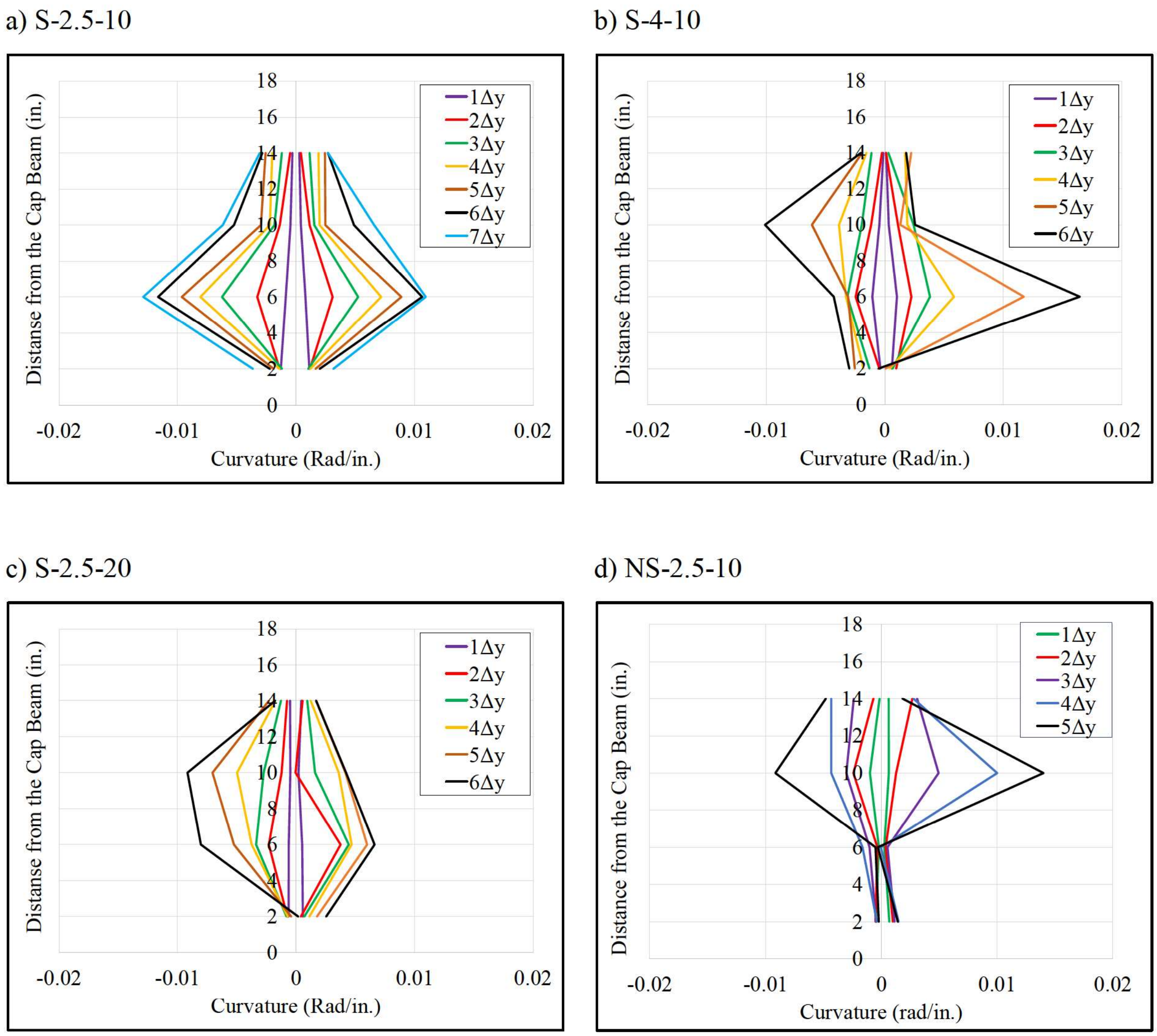

FIGURE 9 Plastic hinge curvature profiles for each displacement.

By comparing the curves plotted in Figure 9, the effects of axial load in the plastic hinge can be investigated. In the specimen S-2.5-20 with higher axial load ratio the maximum curvature was less than other specimens. It is worthwhile to notice that the UHPC splice region in non-seismic detail prevented forming the plastic hinge in the cap beam.

Measured Strains

The longitudinal bars of the specimens were instrumented with 8 strain gauges at different levels. The peak tensile strain profiles of the longitudinal rebars were measured at different displacement levels. By losing some strain gauges at 3Δy, measuring strains of the bars was not feasible (Figure 10). Results for the specimens show that the rebars yielded at desired area and strains were limited to elastic range in UHPC and cap beam area.

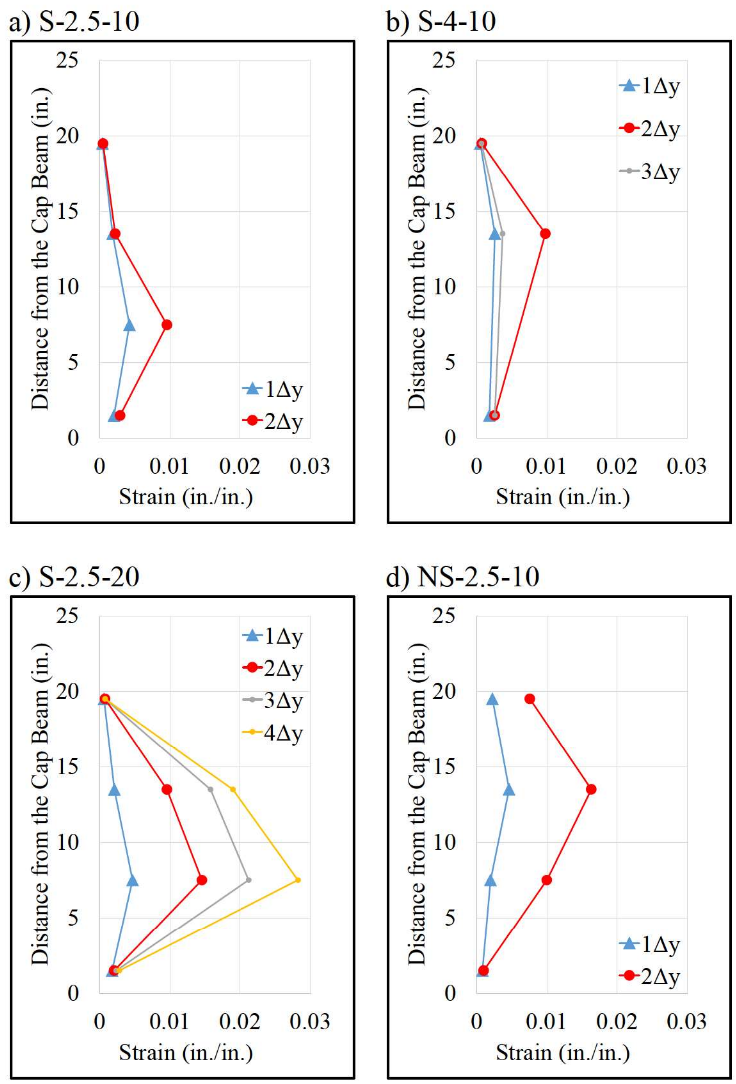

FIGURE 10 Peak tensile strain profiles of bars measured at different levels.

MOMENT CURVATURE ANALYSIS

The moment capacity of the specimens was calculated using a Moment-Curvature section analysis software. For this analysis, the curves were obtained with related axial load and the idealized curves were derived according to Caltrans [14]. Considering the equivalent analytical plastic hinge length and the length of the column, the local displacement capacity of a member can be calculated based on its moment –curvature analysis which includes idealized yield and plastic displacement due to rotation of the plastic hinge.

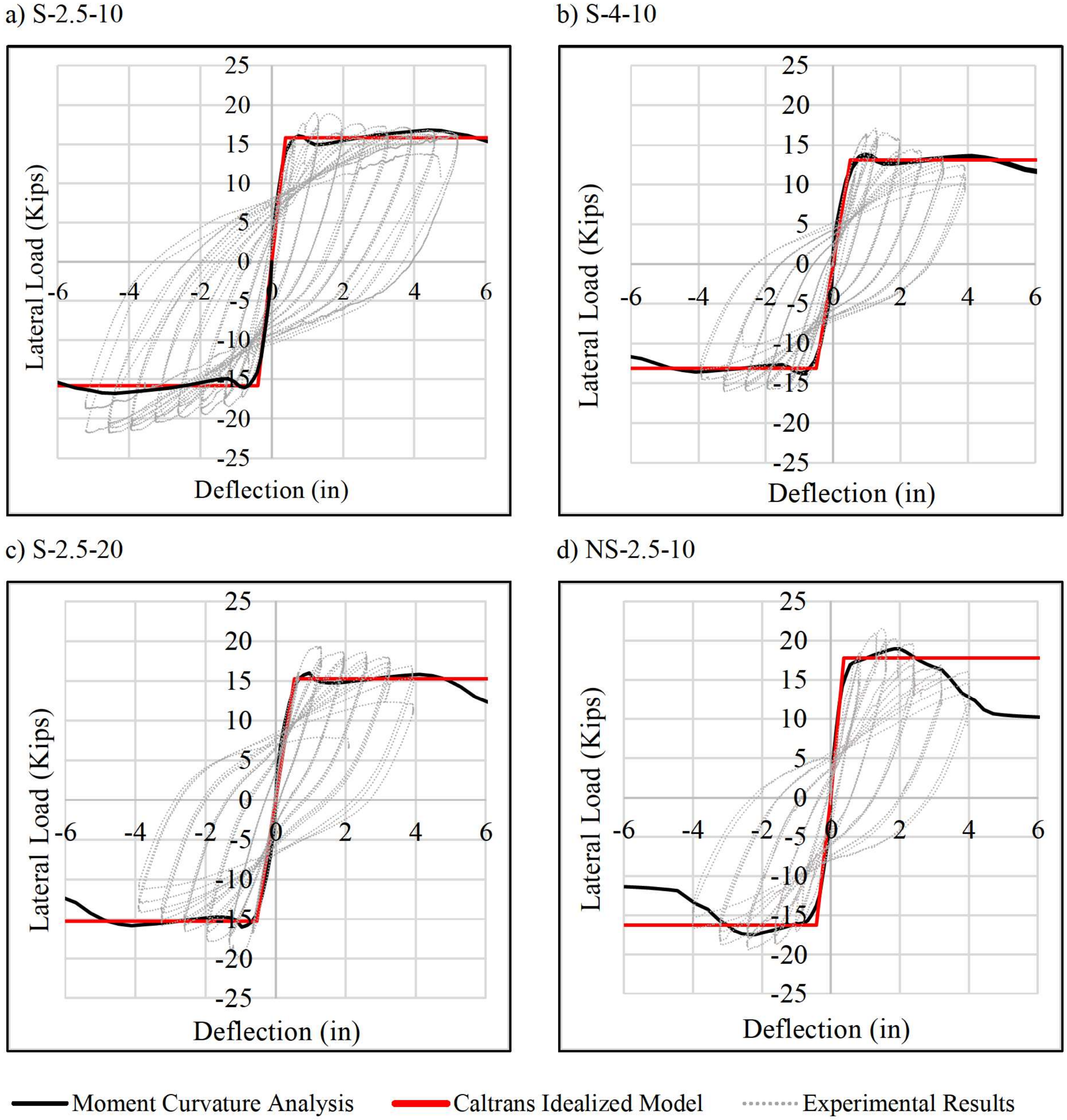

FIGURE 11 Load-Displacement curves of the specimens, based on moment curvature analysis and idealized Caltrans curve.

The effect of expected properties of the rebar, confinement of concrete and shifting the rebar due to construction imperfection were considered in the analysis. The idealized yield displacement and moment capacity of the reference specimen (S-2.5-10) calculated 0.54 in. (13 mm) and 836 kip.in. (87 kN.m) respectively. The ductility of the reference specimen (S-2.5-10), based on Caltrans, was 9. Figure 11 shows the relation between experimental and analytical load-displacement curves of the column using Caltrans and moment curvature analysis.

NUMERICAL INVESTIGATION

Many parameters can affect the load pattern on the bridges [19] and developing an experimental test for all cases are uneconomical, therefore developing a general Finite Element (FE) model can help the researchers for next steps. The numerical analyses were performed using Abaqus finite element software. Dimensionally, the simulated models were the same as the test specimens. An eight-node solid element (C3D8) was selected to model the regular concrete and UHPC, while the truss elements (T3D2) were selected for embedded rebars. Concrete Damage Plasticity (CDP) model was employed for modelling the regular concrete and UHPC materials. The material strength was based on the measured compressive strength of regular concrete and UHPC. A quasi-static analysis with small load increments were used for numerical study. The models were subjected to a monotonic displacement control loading.

The input values used for uniaxial compressive strength of concrete materials in models were the actual values taken from cylinder test at the day after the experimental test. Therefore, the uniaxial compressive strength was 6.4 ksi (44 MPa) for normal concrete and 20.1 ksi (139 MPa) for UHPC. Defining the UHPC material properties were based on previous studies [20]. The bars material considered to be elastic-perfectly plastic material in both tension and compression. Steel material input data in the model for all rebars were taken from actual tensile test. Yield and ultimate stresses were 68 ksi (486 Mpa) and 113 ksi (780 MPa) respectively. Modulus of elasticity and Poisson's ratio of steel were assumed 29000 ksi (200 GPa) and 0.2.

Numerical Simulation Results

A comparison of the results of the simulated specimens and experimental load-displacement curve are plotted in Figure 12. The difference between the FE model and experimental results can be attributed to the material properties, boundary conditions and construction imperfection. Despite the difference, a good correlation of the maximum load capacity was observed between FE model and experiments.

The lateral load capacity of all modeled specimens were similar to the analytical (Moment curvature analysis) results. The numerical results for the bars stresses are depicted in Figure 13. The model was able to simulate the column behavior with reasonable accuracy. As illustrated, the results of FE analysis could specify critical sections, and yielding development in rebars. The results of FE model show no rebar yielding or spalling in the cap beam.

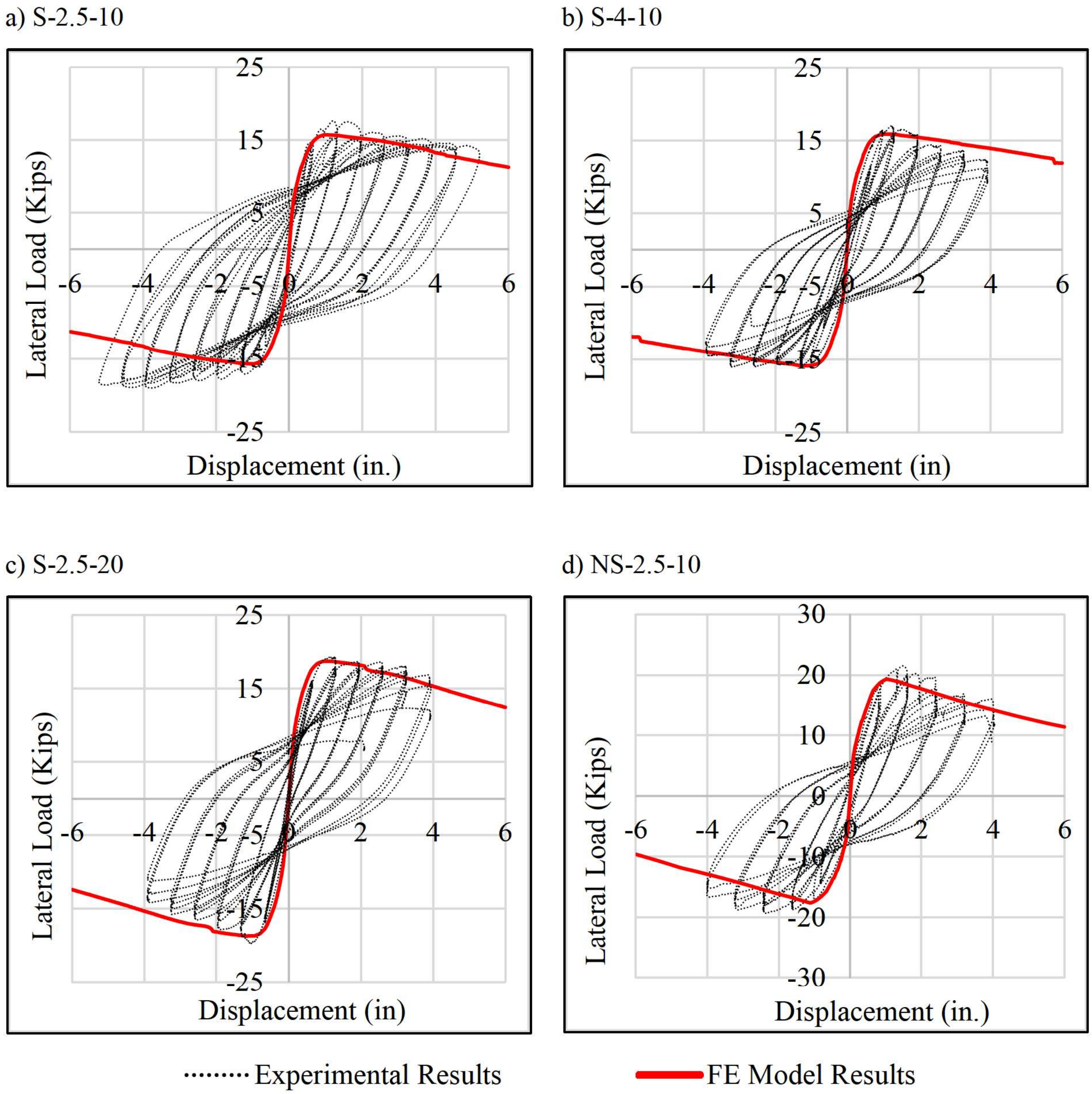

FIGURE 12 Measured hysteretic curves and numerical results of the specimens.

The numerical results are generally in good agreement with the experimental observation justifying the modelling of the proposed detail. Based on developed FE model, using parametric studies will be conducted to investigate the influences of other parameters in future studies.

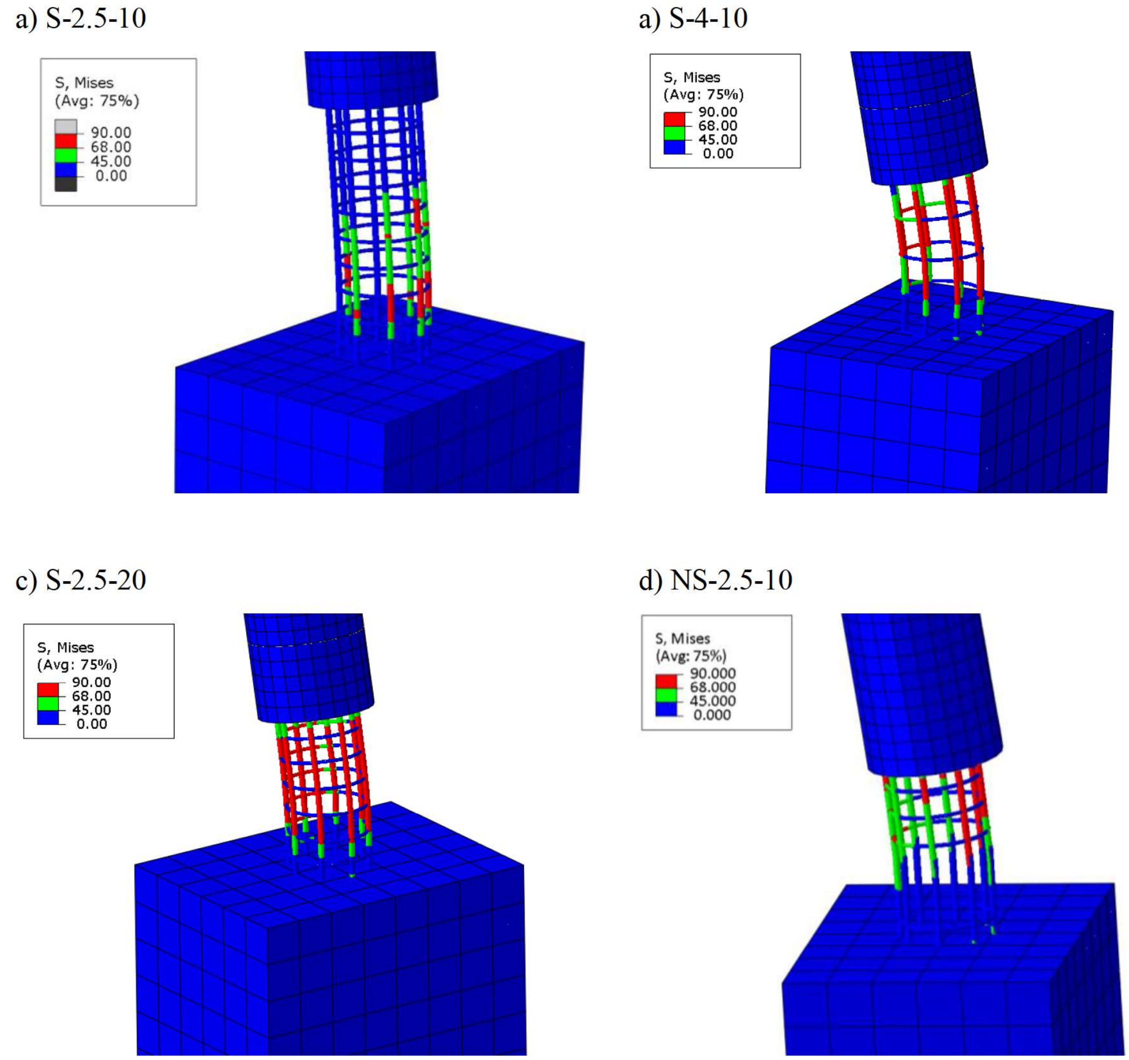

FIGURE 13 Rebar stresses in the numerical models.

SUMMARY AND CONCLUSIONS

Two types of connection for precast concrete column to cap beam for seismic and non-seismic region are proposed. A layer of Ultra-High Performance Concrete (UHPC) was used to connect precast elements. In order to force forming plastic hinge in the desired location for seismic detail, another layer of UHPC was added below the cap beam. Four specimen in cantilever configuration were tested under combined constant axial load and cyclic lateral load. Various responses of the connection were highlighted. Influence of transverse reinforcement and axial load on the behavior of the connections were investigated. The results from the experiments were used to develop a calibrated numerical model which can lead to develop a design guideline for proposed connection. Results of the study has led to the following conclusions:

- All of the specimens with seismic detail showed ductile behavior and the plastic hinge formed in the desired location. High workability of the UHPC and large tolerance of bars in the proposed detail can facilitate and accelerate the on-site construction. Designers should consider the shifting of the plastic hinge which can effect on the behavior of the structure.
- Judging from the observed ductility and failure mode of specimens, the main characteristic of the proposed connection is influenced by transverse rebar ratio. The distance between the stirrups plays a major role in preventing longitudinal bars buckling. Results show that increasing axial load, can provide more capacity but less ductility for the specimen.
- No major crack was observed in the cap beam for the proposed seismic and non-seismic details. Therefore the non-seismic detail, with a seismic design consideration can be an alternative detail even for seismic regions. This detail is preferred as it is easier to build and less expensive.
- No significant damage was found in splice region even in the absence of the transverse reinforcement in this region. Observing no failure in splice region indicates that UHPC can provide a good confinement and shear capacity. Eliminating the need for stirrups in the splice region can facilitate the construction procedure in the field. Moreover, in comparison with conventional concrete, even a short lap splice of bars in UHPC can transfer forces between spliced bars, which leads to a decrease in casting volume while saving time in the field.

ACKNOWLEDGMENT

The research study, results of which reported in this paper was partially sponsored by Accelerated Bridge Construction University Transportation Center (ABC-UTC) at Florida International University. ABC-UTC is a Tier 1 UTC funded by U.S. DOT. Authors would like to acknowledge and thank the sponsors for their support. Opinions and conclusions expressed in this paper are of the authors and do not necessarily represent those of sponsors. The authors are also thankful to Ductal® by LafargeHolcim, for providing the UHPC material.

REFERENCES

1- Culmo, Michael P. Accelerated bridge construction-experience in design, fabrication and erection of prefabricated bridge elements and systems. No. FHWA-HIF-12-013. 2011.

2- Tullini, Nerio, and Fabio Minghini. "Grouted sleeve connections used in precast reinforced concrete construction–Experimental investigation of a column-to-column joint." Engineering Structures 127 (2016): 784-803.

3- Kim, T-H., H-M. Lee, Y-J. Kim, and H. M. Shin. "Performance assessment of precast concrete segmental bridge columns with a shear resistant connecting structure." Engineering Structures 32, no. 5 (2010): 1292-1303.

4- Stephens, Max T., Dawn E. Lehman, and Charles W. Roeder. "Design of CFST column-to-foundation/cap beam connections for moderate and high seismic regions." Engineering Structures 122 (2016): 323-337.

5- Shim, Changsu, Sangyong Lee, Seongjun Park, and Chandara Koem. "Experiments on prefabricated segmental bridge piers with continuous longitudinal reinforcing bars." Engineering Structures 132 (2017): 671-683.

6- 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 (2008): 3782-3792.

7- Culmo, Michael P. Connection details for prefabricated bridge elements and systems. No. FHWA-IF-09-010. 2009.

8- Graybeal, Benjamin A. Material property characterization of ultra-high performance concrete. No. FHWA-HRT-06-103. 2006.

9- Farzad, M., Mohammadi, A., Shafieifar, M., Pham, H., & Azizinamini, A. (2017). Development of Innovative Bridge Systems Utilizing Steel-Concrete-Steel Sandwich System (No. 17-02229).

10- Baqersad, Mohamadtaqi, Ehsan Amir Sayyafi, and Hamid Mortazavi Bak. "State of the art: mechanical properties of ultra-high performance concrete." Civil Engineering Journal3.3 (2017): 190-198.

11- Farzad, Mahsa, Mohamadreza Shafieifar, and Atorod Azizinamini. Accelerated Retrofitting of Bridge Elements Subjected to Predominantly Axial Load Using UHPC Shell. No. 18-05067. 2018.

12- Shafieifar, Mohamadreza, Mahsa Farzad, and Atorod Azizinamini. Alternative ABC Connection Utilizing UHPC. No. 17-03398. 2017.

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

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

15- Aashto, L. R. F. D. "Bridge design specifications." (1998): 5-138.

16- Paultre, P., R. Eid, Y. Langlois, and Y. Lévesque. "Behavior of steel fiber-reinforced high-strength concrete columns under uniaxial compression." Journal of Structural Engineering 136, no. 10 (2010): 1225-1235.

17- Yuan, Jiqiu, and Benjamin A. Graybeal. Bond behavior of reinforcing steel in ultra-high performance concrete. No. FHWA-HRT-14-090. 2014.

18- Park, R. "Evaluation of ductility of structures and structural assemblages from laboratory testing." Bulletin of the New Zealand national society for earthquake engineering 22, no. 3 (1989): 155-166.

19- Sadeghvaziri, Eazaz, Mario B. Rojas IV, and Xia Jin. "Exploring the potential of mobile phone data in travel pattern analysis." Transportation Research Record: Journal of the Transportation Research Board 2594 (2016): 27-34.

20- Shafieifar, Mohamadreza, Mahsa Farzad, and Atorod Azizinamini. "Experimental and numerical study on mechanical properties of Ultra High Performance Concrete (UHPC)." Construction and Building Materials 156 (2017): 402-411.