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SHIFTED PLASTIC HINGE COLUMN CONNECTIONS USING GROUTED SLEEVES FOR ACCELERATED BRIDGE CONSTRUCTION

by

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A dissertation submitted in partial fulfilment of the requirements for the degree of Doctor of Philosophy
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in the College of Engineering and Computer Science
at the University of Central Florida
Orlando, Florida

Fall Term
2017

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Accelerated bridge construction (ABC) is being increasingly used in new bridge construction and repair. ABC typically requires prefabricated elements joined with mechanical couplers. Grouted sleeves (GSs) offer good construction tolerances and load transfer between precast elements. However, previous research identified some performance issues with precast columns employing GS connections for seismic regions. Therefore, there is a need to develop improved connection details. This research consists of three components: testing of six large-scale precast reinforced concrete column models, a series of individual component tests on GS bar splices, and analytical studies. Large-scale, precast column models were designed and experimentally tested using a shifted plastic hinge (SPH) concept to minimize the damage in the capacity-protected elements and retain the column ductility. The column testing matrix considered aspect ratio, moment gradient, and splicing details. Column models were tested in an upright cantilever configuration under quasi-static cyclic load. Results showed that SPH can be used for both flexural and flexural-shear columns. Two types of component tests were performed: tensile tests to quantify the tensile behavior of the splices, and strain penetration tests to quantify the slip at the sleeve ends. The tests were used to obtain constitutive models for the bond-slip behavior of the GS splices. Results showed that GS splices developed the full ultimate stress of the spliced bars and that the slip at sleeve ends can considerably influence the global behavior of the precast columns. The analytical models were developed in OpenSees using fiber-based beams models and they incorporated the calibrated bond-slip models of GS splices. The large-scale column tests were simulated and compared with respective experimental results. Analytical results showed that the developed models were able to mimic the column behavior and can be used for analysis of GS precast columns.
To my dear and precious family, who have always pushed me to achieve my goals and supported me in every single step I set forth for myself. My words cannot express my appreciation for you,
ACKNOWLEDGMENTS

The completion of this study was made possible with the guidance, continuous support and endless patience of Professor Kevin R. Mackie. I have felt privileged to be his student and enjoyed working with him and meeting with him on a regular basis in the last four years. His contributions to my education will be retained throughout my professional life.

I would like to express my sincere gratitude to Dr. Zachary B. Haber for the expert advice and the support he always freely provided throughout the course of the study.

I want to thank those that helped me in the laboratory work of this research: Juan Cruz, Titchenda Chan, Yasir Al-Lebban, Munaf Al-Ramahee, Jacob Solomon, Blake Lozinski.

I would like to thank all my friends and colleagues for their friendship and unconditional support. Also, thanks to the many people that I have shared an office with over the past four years.

The research presented in this document was funded by Splice Sleeve North America (SSNA). However, conclusions and recommendations are made by the authors and do not necessarily represent the views of the sponsor.
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CHAPTER 1: INTRODUCTION

1.1 Background

The United States (US) highway bridge inventory is crucial for transportation of goods to support both local and national commerce. Furthermore, highway bridges play an important role in the daily commute of the traveling public. As the bridge inventory continues to age, the number of users has been constantly increasing from 1945 to present day at a rate of 2.5 million vehicles per year.

The greater majority of bridges in the US were built soon after World War II, and were designed and constructed for a design life of 50 years. Today, many of these bridges have reached or exceed their intended design life, and now many of these structures require major rehabilitation or replacement. As of 2015, approximately 10% of the nearly 610,000 bridges in the US have been rated “structurally deficient” according to the metrics used by the Federal Highway Administration (FHWA) [26]. This rating does not necessarily indicate nor imply that these bridge structures are unsafe, but that they require significant and frequent maintenance, and may require posted weight restrictions to reduce service loads. Furthermore, another 14% are rated “functionally obsolete”, which means that they do not meet current traffic and safety requirements. Repair and replacement of these bridges is rather challenging, especially in areas with traffic congestion issues such as densely populated cities. The replacement of off-grid bridges can also be challenging due to availability of construction materials. A similar set of challenges are associated with the construction of new bridges.

Traditionally, cast-in-place (CIP) concrete has been most commonly used in the construc-
tion of highway bridges. This led departments and agencies to take the initiative to develop programs to reduce the impact of conventional bridge construction on the public safety, constructibility, economy, and environment. These programs are called Accelerated Bridge Construction (ABC) techniques. One of the ABC examples is a program called Every Day Counts (EDC), which was developed by FHWA in cooperation with the American Association of State Highway and Transportation Officials (AASHTO) in 2009. This program aims to employ innovations to speed up the construction process. Highways for Life (HFL) is another ABC technique which was launched by FHWA to construct longer-lasting bridges in faster ways using innovations. Various state departments of transportation have started to develop their own ABC systems and procedures.

An essential part of ABC is the use of precast (prefabricated) bridge elements which provide many benefits compared with CIP construction. Some of the benefits include:

1. Reduced Traffic Disruption: CIP bridge construction contributes to traffic congestion since it requires significant on-site construction work. ABC significantly eliminates most of the on-site work because it utilizes precast components and thus less effect on traveling public.

2. Minimized Work Zone Risk: Bridge construction often involves working near high-speed traffic, at high elevations, and/or above water. These situations subject workers to high risks during CIP construction. ABC allows workers to have a significant working time reduction when precast elements are fabricated in a safe and controlled environment off-site.

3. Improved Constructibility: Working at high elevations and in congested areas are common practice in bridge construction. CIP construction have many tasks that need to be performed at these conditions, while ABC construction techniques eliminate
significant number of on-site tasks thus alleviating constructibility pressures.

4. **Reduced Environmental Impact:** Bridges that are over water in or environmentally sensitive areas that have restrictions and regulations are good candidates for ABC construction. Precast elements use significantly minimizes area contamination which might be caused by spilled concrete or parts of formwork. These contaminations are most likely to occur during CIP construction.

5. **Increased Quality:** Precast elements for ABC are produced in plants where high quality, well cured products are monitored in repetitive and systematic operations. On the other hand, CIP products are less controlled than precast products.

6. **Reduced Cost:** Although initial cost of precast bridge elements is quite higher than their CIP counterparts, the life-cycle cost of precast bridge is less. Initial cost of precast elements is related to the innovative connections of precast elements. Lack of specialized contractors and standardization are factors which raise the cost [69]. However, savings from reduced construction delays decrease the life-cycle cost of the project; making the precast construction competitive to CIP construction.

Precast elements have been employed in bridges for a long time. Precast bridge superstructures started in the 1950s. Years after that precast bridge substructures were utilized in bridge construction. Precast elements have been used in regions with low seismic activities. However, they have seen limited use in regions with high seismicity, especially for substructure elements, due to concerns related to connection seismic performance. Connection regions for precast substructure elements typically coincide with plastic hinge zones. Thus, under earthquakes these connections are subject to high deformation demands. Thirty-six out of fifty US states are considered to be moderate and high seismic zones [48]. Thus, ABC construction is facing a national challenge.
1.2 Previous Research on ABC Substructure Connections for Seismic Zones

1.2.1 Socket Connection

Insertion of a precast member into an adjacent member makes the socket connection. The socket connection is completed by either filling the socket with grout or concrete or casting the adjacent member around the precast member in place. Lateral load capacity for this connection depends on the embedment length of the precast member, while the vertical load capacity depends on the friction between the precast member and the socket. Therefore, the surface of the socket or the precast member is often roughened to provide better bonding interface.

Socket connections have been studied for use in substructure connections. Haraldsson et al. [32] performed experimental investigation on three column-footing subassemblies that utilized socket connection. The three specimens had a diameter of 20 in and aspect ratio of 3.0. The embedded length of the columns was roughened in a saw-tooth pattern. Two specimens had a column embedment length to column diameter ratio of 1.1 while the ratio was 0.5 for the third specimen. Lateral cyclic tests showed that the connection behavior comparable to that of a CIP system. Also, the study demonstrated how that connection type provides constructibility advantages. The connection allows for large placement tolerances and it does not need grouting if the adjacent member is CIP.

Mashal and Palermo [49] conducted experimental study on half-scale precast two-column bent utilizing member socket connection between the column and the footing. Each column had a diameter of 19.7 in and a height of 115 in. Horizontal quasi-static cyclic loading was applied to the model. Both columns and footings were precast so the socket connection was completed by inserting the column inside the footing and then grouting the socket. The
Socket was one column diameter deep and it was roughened for better bond. The model showed a very stable hysteresis behavior and formed plastic hinges similar to what is expected in CIP systems. Also, the achieved ductility capacity was good.

1.2.2 Pocket Connection

It is one of the ABC connections that involve creating an opening in a footing or a bent-cap. Then a column with protruding reinforcement is inserted into the pocket. The pocket then is filled with concrete or grout. The main consideration for this connection type is the development length of the reinforcing inside the pocket since the bond of reinforcing bars is the mechanism to transfer force in this connection. Pocket connections provide significant construction tolerances. Restrepo et al. [61] investigated the seismic performance of two 0.42 scale bridge column model connected to bent-cap using pocket connection. The models response was compared with CIP model as a control specimen. The first model had more reinforcement in the joint (designated CPFD) and the bent-cap than the second model (designated CPLD). Testing protocols was based on displacement ductility levels.

The major finding of this investigation was that both specimens showed stable hysteresis loops with insignificant capacity degradation compared with CIP model. Furthermore, both specimens achieved a displacement ductility of 8.0 compared with 10.0 for the CIP model. CPFD specimen developed plastic hinging very similar to CIP model, which was in the column near the joint as expected. Also, the joint shear stiffness was comparable to that of CIP model although a slightly different crack pattern developed in the CPFD joint. CPLD specimen exhibited plastic hinging in the column near the joint and joint shear deformations. The specimen exhibited a lower joint shear stiffness which led to wider joint cracks and larger bar slip compared with CPFD model. Overall, both specimens showed similar behavior to
the CIP model.

1.2.3 Hybrid Connections

Precast elements that are connected using hybrid connections exhibit two mechanisms: self-centering mechanism which is done by employing a prestressing tendon through the joint and energy dissipation mechanism by using bonded mild steel bars that are spliced or anchored in the plastic hinge region, where they yield to dissipate energy. If the precast consists of several segments, mild steel bars may be used to connect the segments and increase the energy dissipation. The self-centering tendon is designed to be elastic under lateral loading.

Several experimental studies on precast columns incorporating hybrid connections have been conducted in the past. Billington and Yoon [16] investigated seven 1/6 scale precast segmental columns connected using hybrid connection under drift-based cyclic loading. Two sets of columns were fabricated: four short columns and three long columns. For each set, one column consisted of regular concrete segments and the other columns included ductile fiber-reinforced concrete (DFRCC) segment in the plastic hinge region besides the regular concrete segments. Column cross section was (8 x 8) in and had four unbonded 7-wire strands with a diameter of 0.375 in for post-tensioning. Results showed that columns with DFRCC in their hinge regions dissipated more energy than the columns with traditional concrete segments up to drift levels between 3%- 6%. Also, it was found that columns with DFRCC segment which was embedded deeper into the fixity region dissipated more energy than the columns with their DFRCC segments less embedded.

Yamashita and Sanders [74] conducted shake table testing on single 1/4 scale precast segmental column consisted of three reinforced concrete (RC) segments connected using hybrid connection. The column had aspect ratio of 4.0 in the testing direction in investigate its flex-
ural characteristics. The column had no conventional reinforcement to cross the segments. The joints between the segments had shear keys to prevent slippage. Segmental bridge epoxy adhesive was used to join the footing, segments, and the loading head. Twelve unbonded 7-wire strands with a diameter of 0.6 in were used for post-tensioning. The specimen performed very well and showed good ductility with essentially no residual displacement and limited spalling at the column base. Also, the joints between the segments remained closed except that at the column base.

1.2.4 Grouted Duct Connections

In grouted duct connections, protruding reinforcing bars from a precast member can be fully developed in the adjacent member. Ducts are placed in the adjoining member with a sufficient length to develop the reinforcing bars. The ducts are then filled with grout. Grouted ducts connections for seismic regions have been investigated by a number of researchers. Restrepo et al. [61] investigated the seismic performance of 0.42 scale column-cap beam subassembly that employed grouted duct connection (denoted GD) and it was compared to CIP model. The column had a diameter of 20 in and a height of 45 in. The column was reinforced longitudinally with 16 #5 and transversely with #3 hoops spaced at 1.5 in. Corrugated ducts (1.75 in diameter) were used in the bent-cap to house the reinforcing bars. High-strength, non-shrink, cementitious grout was used to fill the ducts and anchor the column longitudinal reinforcing bars. Cyclic loading results showed that the GD specimen exhibited stable hysteretic behavior without significant capacity degradation. Full plastic hinging was characterized in the column near the joint, similar to CIP model. GD model achieved displacement ductility of 8.0 that was 20% less than that of CIP model.
Tazarv [72] conducted a study on a half-scale precast bridge column model that employed a grouted duct connection to a CIP footing (denoted PNC). The column had an aspect ratio of 4.5 and a diameter of 24-in. The column was reinforced longitudinally with 11 #8 reinforcing bars and a # 3 spiral with a 2-in pitch transversely. Protruding reinforcing bars from the column base were anchored 28-in in 3-in corrugated steel ducts located in the footing using ultra-high performance concrete (UHPC). The longitudinal reinforcing bars were debonded 8-in in the column-footing interface to improve the ductility of the column. Slow cyclic loading was used with a drift-based loading protocol. The column was compared to a conventional CIP column having the same details. Results showed that both PNC and CIP models had similar hysteretic behavior. Also, the displacement ductility capacity of CIP and PNC models was 7.36 and 6.30, respectively. The observed damage was similar between both models for most drift levels. The grouted duct connection was inspected upon the completion of the test and there was no damage. Overall, the connection was concluded to be emulative of a conventional CIP construction.

1.2.5 Bar Coupler Connections

Bar coupler connections are used to splice the reinforcing bars in order to transfer the force between the adjacent members. Bar couplers have the advantage of allowing the reinforcement details in the connection to be similar to these in CIP construction as long as the coupler has enough space for placement. A number of mechanical bar splices are commercially available in the market nowadays. A comprehensive literature review regarding these connections and their seismic performance was conducted are presented in Chapter 2.

In the United States, mechanical reinforcing bar splices have been used in low and moderate seismic zones for bridge substructure connections (Edison Bridge over the Caloosahatchee
River in Fort Meyers, Florida, Route 9N over Sucker Creek in Hague, New York and the Riverdale Rd. Bridge over I-84 outside Salt Lake City, Utah) but not in high seismic zones. One of the most common mechanical splices are grouted splice couplers. They have gathered a great deal of attention from designers and engineers due to their good field tolerates and ease of assembly. However, bridge columns with grouted splice connections have only been subject to a limited number of investigations in the US. Nevertheless, research thus far has indicated some performance issues related to this type of connection detail for seismic applications.

1.3 Behavior of connections with grouted splices

1.3.1 Background

Grouted sleeve couplers was originally developed and used in the construction of the Ala Moana hotel in Honolulu, Hawaii [76]. Grouted sleeve devices typically consist of a metallic sleeve (ductile cast iron or mild steel) and a cementitious grout filler material. A schematic of one of these devices is shown in Figure 1.1a. Discontinuous bars are inserted into opposite ends of the sleeve, which is subsequently filled with grout. After curing, tensile and compressive forces are transferred by the ribs on the reinforcing bars into the grout filler material and then to the metallic sleeve. In some cases, the inner surface of the sleeve is lined with a series of lugs or ribs to aid force transfer and shorten the development length of the embedded bars. Although non-proprietary grouted sleeve systems have been developed and tested [24, 41], proprietary systems are most commonly deployed.

At the precasting plant, bars are inserted into the tapered-end of the sleeve (factory end) during construction of a reinforcement cage. Figure 1.1b shows an example of a column rebar
cage with grouted sleeves installed at the base. During casting, specialized form devices are used to prevent concrete from entering the base of the sleeve, which are typically provided by the manufacturer of the sleeve. Figure 1.1c shows the base of the finished precast column with a grouted splice connection. Grout ports can be observed which are used to connect the precast column with an adjacent member. Once delivered to the construction site the column can be lowered on the reinforcing bar dowels that protrude from an adjacent member and the assembly can be grouted. Grout is pumped through plastic tubes that protrude from the column at grout inlet and outlet locations; grout tubes are placed prior to casting concrete at the precasting yard. After grout curing, the connection would be considered completed and would able to withstand applied loads.

Figure 1.1: Grouted sleeve details
1.3.2 Uniaxial Tests

The uniaxial performance of GS systems has been previously investigated by numerous researchers. To date, studies have focused on monotonic [24, 51, 41, 31], cyclic [31], fatigue [57], and high strain-rate [62] loading regimes. Generally speaking, GS splice systems exhibit three primary failure modes: (1) bar fracture; (2) bar pull-out from the grouted sleeve; and (3) fracture of the metallic sleeve. Spliced bar assemblies can develop adequate ductility prior to failure by any one of these three primary failure modes. However, for seismic design purposes, bar fracture would be considered to the most desirable of the three modes.

1.3.3 Column Tests

Research on the element-level behavior of bridge columns with grouted splice connections is more limited. There have been three studies conducted in the US that focused on the behavior of precast columns with grouted splice connections. Figure 1.2 presents an illustration of the five different connection details that have been investigated.

Haber et al. [30] reported on the cyclic performance of three large-scale column models, two of which were precast columns that had grouted splice connections, and the third one was a baseline CIP column model. Columns model had 24-in diameter cross-sections, an aspect ratio (AR) of 4.5, and where reinforced longitudinally with 11 # 8 Gr. 60 bars and transversely with a Gr. 60 # 3 spiral with a 2-in pitch. The CIP column was designed for a target displacement ductility of 7.0. The first precast column had a moment connection at the column-footing joint that was similar to Detail 1 shown in Figure 1.2; grouted splices were cast into the base of the column and connected directly to the footing. The second precast column employed a precast pedestal one-half column diameter in height separating
the footing and the precast column similar to Detail 2 shown in Figure 1.2. Footing bar dowels passed through grout-filled corrugated steel ducts within the pedestal and connected with grouted splices at the base of the precast column. Columns where tested using slow cyclic loading in a single cantilever configuration. All specimens showed similar performance in terms of ultimate load capacity and energy dissipation up to 6% drift. However, the achieved average displacement ductility capacity of the precast columns was 4.5 compared with 7.4 for the CIP column. Reduced ductility capacity (40%) in precast columns was a result of disrupted plastic hinge formation caused by the added stiffness of grouted splices and pedestal ducts. Strain concentrations formed at the column-footing joints and premature bar rupture occurred in the footings.

Tazarv [72] investigated the seismic performance of a connection detail similar to Detail 3 shown in Figure 1.2. A single precast column model was tested that had the same geometry...
and reinforcement ratios as those tested by Haber et al. [30]; the same CIP baseline model was used for comparison. Similar to Detail 2, the precast column was connected atop a pedestal, which was CIP instead of precast. The longitudinal reinforcing bars that passed through the pedestal were debonded from concrete using duct tape to improve plastic rotation and displacement ductility capacity, but were fully bonded within the grouted splice. The column was tested using slow cyclic loading in a single cantilever configuration. In comparison to the precast columns tested by Haber et al. [30], Tazarv and Saiidi’s precast column exhibited a higher displacement ductility capacity (6.32) and well-distributed plasticity within the pedestal region as indicated from measured strain profiles. Furthermore, the observed damage was concentrated within the pedestal as opposed to the previous precast columns where damage occurred a few inches below footing surface.

Another study was conducted by Pantelides et al. [53] where the performance of eight half-scale bridge columns was investigated. Two sets of four columns were tested. Each set consisted of one CIP column and three precast columns with grouted couplers. The first set was a column-footing connection and employed NMB Splice Sleeve grouted couplers in the precast columns. The second set was a column-cap beam connection and employed Lenton Interlock couplers in the precast columns. The difference between the couplers is that the first one is longer and connects the rebars by grouting at both ends, while the second coupler type is shorter and the connection is completed by threading the factory dowel to the top end and grouting the field dowel to the bottom end. For each set, one precast specimen had a typical grouted coupler connection (Detail 1 in Figure 1.2). Another precast specimen that employed the grouted splices in the footing or cap-beam (Detail 4 in Figure 1.2). The third precast specimen employed the grouted couplers at the base of the column shaft in addition to a debonded region of $8d_b$ in the bar dowels of footing or cap-beam (Detail 5 in Figure 1.2). All columns were tested using slow cyclic loading. The displacement ductility capacities for
the first set of precast columns were reduced by 39%, 31%, and 24%, respectively, compared with their CIP column. For the second set of precast columns, the displacement ductility was reduced by 51%, 42%, and 69%, respectively, compared with their CIP model. The researchers noted that the behavior and failure of the first column (Detail 1) in each set was similar to that observed by Haber et al. [30]. The second precast column (Detail 4) in each set showed improvement compared to the columns with Detail 1. Well-distributed hinging was achieved by moving the splices into the capacity-protected elements. However, it should be noted that this detail could create constructability issues related to congestion of reinforcement in the footing or cap-beam. The third column in the first set (Detail 5) exhibited the least reduction in displacement ductility due to the provided debonded length which alleviated the strain concentration at the column-footing interface. The reduction in ductility of the third column in the second set could not be judged due to error to test termination.

In summary, previous research identified some performance issues with precast columns employing grouted splice connections. Of specific concern were columns that employed grouted splices within the column directly adjacent to the capacity protected element (similar to that of Detail 1 shown in Figure 1.2). This design detail resulted in premature failure and reduced displacement ductility capacity compared with corresponding CIP columns. Although alternative design details have been investigated that show improved displacement ductility capacity and seismic performance, they are not the most practical in terms of a constructibility standpoint. For instance, the use of pedestals, which can improve performance in some cases (Detail 3), require more on-site construction time, placement of couplers within the footing (Detail 4) may result in constructibility issues related to reinforcement congestion. Also, adequate debonded length (like Detail 5) may not be provided by bridge contractors. Given the demand for ABC and popularity of grouted splice connections, there is still a need
to develop improved details, design methods, and analysis techniques.

1.4 Design Concept

In modern seismic design, bridge columns are designed and detailed to undergo damage during an earthquake event. In an extreme case, it would be expected that concrete spalls and reinforcing bars undergo yielding. Figure 1.3-a depicts the expected plastic hinge region at the column-footing interface for conventional CIP bridge columns. As long as this region is properly detailed, well-distributed plasticity will occur at the base of the column producing adequate plastic rotation and displacement ductility capacities. Figure 1.3-b shows a typical GS connection detail, similar to those discussed in the previous section, where the couplers are located within the column shaft, and the associated locations where plasticity has been shown to occur. In this case, previous research has shown that majority of plasticity is forced into the capacity-protected footing which reduces the plastic rotation and ductility capacities.

![Figure 1.3: Plastic hinge locations for conventional and precast columns](image)

The proposed method is based on shifting the plastic hinge location away from the column-
adjacent member interface. The concept of shifted plastic hinging (SPH) has been used in the past for both new construction and repair of existing structures. Scribner and Wight [67] employed plastic hinge relocation in RC beams by using intermediate reinforcement layers in the beam-column joint extending in the beam to approximately twice the beam effective depth from the joint. It was observed that the use of intermediate reinforcing layers inhibited crack opening near the joint and distributed the cracks over a longer length from the joint. It also increased the dissipated energy compared with a reference assembly.

Similar study was conducted by Abdel-Fattah and Wight [2]. They used shifted plastic hinge mechanism in RC building beams to prevent damage in the beam-column joints. In their study, the plastic hinge mechanism was relocated away from the beam-column joint by providing additional steel reinforcement layers that extended a distance 1.5 times the effective beam depth from the joint into the beam. This detail stiffened the beam section directly adjacent to the beam-column joint minimizing deformation and damage in the panel zone. Another example of this design practice in building construction is the use reduced beam sections (RBS) in structural steel design.

In bridge applications, plastic hinge shifting has been primarily used for repair and retrofit of earthquake-damaged columns. Lehman et al. [39] investigated different post-earthquake repair techniques on a series of scaled bridge column models. After initial testing, columns had numerous buckled bars and the confined concrete core was damaged. One repair method employed additional reinforcement and an added concrete jacket around the damaged hinge to shift plasticity upward upon additional load cycles.

Rutledge et al. [64] in their study used the plastic hinge shifting concept to repair bridge columns. Two large scale cantilever columns, which were previously damaged, were repaired using CFRP sheets at the column base region. The first column was wrapped with CFRP
sheets up to a distance of 23.5 in from the footing surface and the other column was wrapped up to 47 in. Both repaired columns showed greater load capacity than the previous behavior before repair. One column developed plastic hinge above the CFRP sheet (at approximately 27.5 in high above the footing), while the other column shifted the plastic hinge to a location below the footing surface due to excessive CFRP confinement that was used for that column.

Parks et al. [55] used a similar technique to repair a scaled precast bridge column model employing a grouted splice connection at the column-footing joint. In this study, a carbon fiber reinforced polymer (CFRP) jacket was used as a stay-in-place form for repair concrete that was cast over additional reinforcement anchored in the footing. Upon reloading, a new plastic hinge formed directly above the repaired section which had been effectively stiffened by additional reinforcement and added confinement from CFRP.

Figure 1.3-c shows the proposed design concept for precast columns with GS connections where the majority of plasticity is designed to occur above the coupler region. To shift the plastic hinge, the plastic moment capacity of the section at the column-footing (or column-cap) interface was increased relative to the section located above the GSs by using currently available GS splice systems that allow for transition splicing. Transition splicing refers to using a GS device to join bars of different size and/or different grades of steel. The transition index $i_T$ is defined as the incremental difference between the largest and smallest bar inserted into the splice. In the proposed design concept, a normal-strength (NS) steel bar is inserted at the factory end of the coupler and an up-sized, high-strength (HS) steel bar is inserted at the field end. Although the moment demand is higher at the base of the column, the larger, HS bars significantly increase the yield strength of the section shifting the critical section above the GSs.
1.5 Plastic Hinge Length

The concept of plastic hinge length has been under study since the early 1950’s. The equivalent plastic hinge length ($L_p$) should be distinguished from the actual plastic hinge length ($L_{pr}$) since the latter is the physical region in which plastic deformations spread along the RC member length whereas, the former is a fictitious term that is used in lumped plasticity approaches to combine all inelastic deformations to determine the column post-yield displacement [34, 52]. Plastic hinges occur at the maximum regions of RC members where large inelastic curvatures form at that region. The plastic curvature in the plastic hinge ($L_p$) region is typically assumed to be constant. For RC columns, if the plastic hinge length is known, the column tip displacement can be easily determined by integrating the curvature profile along the column and vice versa. Therefore, accurate assessment of the plastic hinge length ($L_p$) is crucial for the relationship between section-level response and member-level response of concrete columns.

1.5.1 Previous Research on Plastic Hinge Length

1.5.1.1 Baker (1956)

Ninety four beam/column tests were performed by six laboratories under the auspices of the European Concrete Committee in the 1950s in order to investigate moment-curvature relationship of beams and columns. The main test variables consisted of yield strength, amount of tension reinforcement, amount of compression reinforcement, concrete strength, single or double concentrated loads, and axial load. The concrete strength varied from 2.5 ksi to 5.8 ksi and the yield strength of reinforcement ranged from 40 ksi to 85 ksi. The amount of tension reinforcement ranged from 0.25% to 4%. The ratio of “binding steel”
varied between 0.05% and 1.51%. According to Baker [12], the binding steel ratio is defined as the volumetric ratio of binding steel (one stirrup plus compression steel between stirrups) to confined concrete (stirrup spacing \( X \) area enclosed by stirrup). Test specimens were subjected to axial loads of \( 0.15f'_cA_g \) to \( 1.0f'_cA_g \). Based on the experimental results, the following equation was proposed to calculate the plastic hinge length \( (L_p) \):

\[
L_p = k_1 k_2 k_3 \left( \frac{z}{d} \right)^{0.25} d
\]  

(1.1)

where:

\( k_1 = 0.7 \) for mild steel

\( = 0.9 \) for cold worked steel

\( k_2 = 1 + 0.5 \frac{P}{P_0} \)

\( k_3 = 0.9 - \frac{0.3}{23.5} (f'_c - 11.7) \quad (f'_c \text{ in MPa}) \)

\( z = \) distance from critical section to point of contraflexure

\( d = \) beam effective depth

Baker [12] reported that the plastic hinge lengths ranged from \( 0.4d \) to \( 2.4d \) for practical values of \( z/d \). The \( z/d \) ratio represents the moment gradient effect, which is similar to the shear span-to-depth ratio \( (L/h) \). Baker and Amarakone [13] simplified Equation 1.1 to the following:

\[
L_p = 0.8 k_1 k_3 \left( \frac{z}{d} \right) c
\]  

(1.2)
where \( c \) is the depth of neutral axis at collapse.

### 1.5.1.2 Mattock (1964)

Mattock [50] performed thirty seven beam tests to investigate the effect of several parameters on the behavior of RC beams. The parameters included concrete strength (4 to 6 ksi), effective depth of beam (10 and 20 in), moment gradient \((z/d = 2.75 \text{ to } 11)\), and tension reinforcement amount (1% to 3%) and bar yield strength (47 to 60 ksi). Results suggested that the spread of plasticity along a beam length increased as the \( z/d \) ratio increased, and as the net tension reinforcement \((q - q')/q_b\) decreased. The following empirical relationship was proposed to calculate the plastic hinge length \((L_p)\).

\[
L_p = \frac{d}{2} \left[ 1 + \left( 1.14 \sqrt{\frac{z}{d} - 1} \right) \left( 1 - \frac{q - q'}{q_b} \sqrt{\frac{d}{16.2}} \right) \right]
\]

(1.3)

where

\( d = \) effective depth of a beam (in)

\( z = \) distance of critical section to point of contraflexure (in)

\( q = \) tension reinforcement index \[= \left( \frac{A_s}{b d} \right) \left( \frac{f_y}{f_c'} \right) \]

\( q' = \) compressive reinforcement index \[= \left( \frac{A_s'}{b d} \right) \left( \frac{f_y}{f_c'} \right) \]

\( q_b = \) balanced tension reinforcement index \[= \left( \frac{A_b}{b d} \right) \left( \frac{f_y}{f_c'} \right) \]
1.5.1.3 Corley (1966)

Corley [22] expanded Mattock’s [50] work by testing forty simply supported concrete beams subjected to single point loads. The main test variables were confinement and size effects. In addition, the effects of moment gradient and amount of tension reinforcement were investigated. The major variables in the study ranged as follows: (1) width of test beams: 3, 9, and 12 in; (2) effective depth of test beams: 5, 10, 24, 30 in; (3) span of test beams: 36, 72, 144, 165, 240, and 330 in; (4) amount of tension reinforcement: between 1% and 3%; (5) ratio of “binding steel”, similar to Baker’s [12] definition, between 0.3% and 9%. Test results suggested that the spread of the plastic hinge region was primarily a function of the beam geometry and that the beam size did not have a significant influence on the rotational capacity. It was also concluded that the effect of \((q - q')/q_b\) could be ignored. A simple expression was proposed to calculate the plastic hinge length \((L_p)\).

\[
L_p = \frac{d}{2} + 0.2\frac{z}{\sqrt{d}} \quad (1.4)
\]


Park et al. [54] tested four full-scale square concrete columns with sections of 22X22 in\(^2\) and aspect ratio \((L/h)\) of 2.0. The axial loads applied to the column specimens ranged from \(0.2f'_cA_g\) to \(0.6f'_cA_g\). They calculated the plastic hinge length \((L_p)\) experimentally using Equation 1.5 using the moment area theorem with the lumped plasticity approach.

\[
\Delta = \Delta_y + \Delta_p = \frac{\phi_yL^2}{3} + (\phi - \phi_y)L_p(L - 0.5L_p) \quad (1.5)
\]

21
They found that the obtained plastic hinge lengths were insensitive to axial load level and had an average value of 0.42\(h\), where \(h\) is the overall column depth. They suggested using a simple plastic hinge length of 0.4\(h\) for concrete columns. Using a similar approach, Priestley and Park [58] proposed the following equation for the calculation of plastic hinge length in RC columns.

\[
L_p = 0.08L + 6d_b
\]  

(1.6)

where

\(L =\) distance from the critical section to the point of contraflexure (in)

\(d_b =\) diameter of longitudinal reinforcement (in)

The above Equation has two components. The first term mainly accounts for column bending, and the second term accounts for bar slip due to the due to bar tensile strain penetration into the footing.

Equation 1.6 was revised by Paulay and Priestley [56] to account for different grades of flexural steel reinforcement. The revised expression is given in Equation 1.7.

\[
L_p = 0.08L + 0.15d_b f_y
\]  

(1.7)

Equation 1.7 is the most commonly used expression for the plastic hinge. It is adopted by the design codes Caltran SDC [18] and AASHTO [1].
Berry et al. [15] conducted a statistical analysis on thirty seven columns from the PEER Structural Performance Database [14]. The selected columns had to meet the requirements of modern bridge column which are in the following:

1. Axial load ratio \( (P/f'_cA_g) \leq 0.3 \).

2. Spiral spacing (pitch) of \( 6d_b \) or less, where \( d_b \) is the longitudinal reinforcement diameter.

3. Effective confinement ratio \( (\rho_{eff}) \geq 0.05 \) where

\[
\rho_{eff} = \frac{\rho_s f_y}{f'_c} \quad (1.8)
\]

where \( \rho_s \) is the spiral reinforcement ratio, \( f_y \) is the spiral yield strength.

4. Concrete cover to transverse reinforcement \( \leq 0.1D \), where \( D \) is the column diameter.

5. Longitudinal reinforcement ratio \( (\rho_l) \leq 4\% \).

Using the statistical analysis, the researchers proposed a plastic hinge expression as given in Equation 1.9. Like the plastic hinge proposed by Priestley and Park [58], the proposed expression by Berry et al. [15] contains both a moment gradient and strain penetration component.

\[
L_p = 0.0375L + 0.015f_y \frac{d_b}{\sqrt{f'_c}} \quad (psi) \quad (1.9)
\]
Around the same time Berry et al. [15] introduced their plastic hinge equation, Bae and Bayrak [11] proposed a plastic hinge length expression that was based on experimental work supplemented with analytical work. The researchers examined previous expressions of plastic hinge length and the discrepancies between them, specifically in their sensitivity to axial load, $P$. They found that aspect ratio, axial load and reinforcement ratio were key factors in defining the plastic hinge region. They developed Equation 1.10 to determine the ultimate tip displacement of a column under a wide range of axial loads.

The expression implies that the plastic hinge region starts at a distance $0.25h$ above the column/footing interface, and therefore adds this distance to the plastic hinge length. The researchers noted that the reason behind that could be the additional stiffness that was added from employing large footings in their experimental work.

The researchers stated that the plastic hinge length does not include a strain penetration component ($L_{sp}$), instead they recommended adding that component separately to the flexural component. Therefore for reasons of comparison with other plastic hinge models, a strain penetration component was included, as given in Equation 1.10.

$$L_p = L \left(0.3 \frac{P}{P_o} + 3 \frac{A_s}{A_g} - 0.1\right) + 0.25h + L_{sp} \geq 0.25h \quad (1.10)$$

where

$$P_o = 0.85 f_c' (A_g - A_s) + f_y A_s \quad (1.11)$$

where
\[ A_s = \text{steel area} \]

\[ A_g = \text{cross-sectional gross area} \]

\[ h = \text{section depth in the strong direction} \]

### 1.6 Objectives and Scope

The main objective of this study was to improve the seismic performance of GS precast column-footing connection details using the shifted plastic hinging technique. The improved detailing also aims at reducing the damage in adjacent elements. The connection was tested and evaluated for use in ABC in moderate to high seismic regions. Based on the experimental results, design expressions and procedures were developed.

The study consisted of three major parts. The first part included two rounds of experimental testing of large scale circular bridge column models. In the first round, four 0.42-scale columns were designed, fabricated and tested. They included two sets of columns, one with aspect ratio of 4.0 and the other with aspect ratio of 2.5. In each set, one column was CIP column as a reference and the other column was a precast column utilizing GS connection. In the second round, Two precast columns with GS connections were tested: one had a 0.42-scale and the other had a 0.33-scale. Both had aspect ratio of 4.0.

The second part of the study was focused on uni-axial tests of the grouted coupler components. The tests included monotonic tension test and strain penetration test. Along with the uni-axial testing program, one-dimensional analytical modeling of the grouted splices was proposed and bond slip constitutive laws of the bar-grout interface were obtained by reverse analysis.
The third part of the study was involved with analytical investigations on the column models using OpenSees. A three dimensional fiber-based model was proposed for both CIP and GS precast columns. The model featured more physical representations than the commonly used two-dimensional fiber-based models. The model made use of the proposed bond slip laws for GS components. The analytical results were compared to the experimental results to validate the modeling procedures.

1.7 Document Outline

The study is presented in seven chapters. Chapter 1 presents the background on accelerated bridge acceleration and literature review regarding various column connections. Also the Chapter presents the concept of plastic hinge and shifted plastic hinge formation and their literature review. Following that, Chapter 2 presents literature review about the mechanical couplers for column connections and their uni-axial behavior. The Chapter also presents background on the bond between steel bar and any cementitious material as a tool to study the bar slip. The Chapter ends with presenting the experimental program which was designed to study the tensile behavior of GSs splicing steel bars and the slip behavior of the steel bars into the grout.

Chapter 3 presents the experimental program which was conducted. The chapter present the design methodology of the columns and construction procedures. The Chapter also presents details about column instrumentation plans and column setup.

Chapter 4 presents results from column tests. Test results included the hysteresis behavior, energy dissipation, damage progression, plastic hinge strain, and deformation components. Results of the GS component tests are presented in Chapter 5. Monotonic tensile test
results are presented first. Then the strain penetration test results along with the analytical modeling are presented.

Chapter 6 presents the analytical modeling of the column component for all the columns which are tested in the study. Material and elements models that are adopted are presented first. After that the modeling procedures for CIP and precast columns are presented. Finally, analytical results are compared with experimental results to validate the modeling procedures. Finally, observations, conclusions and recommendation for future research are presented in Chapter 7.
CHAPTER 2: REINFORCING BAR COUPLERS

2.1 Introduction

As discussed in Chapter 1, mechanical bar splices, known as bar couplers, are used to connect the reinforcing bars to join the adjacent members. The connection creates a continuous load path to transfer tension forces. The mechanism of bar couplers is similar to that of welded but joint but the bar couplers provide quicker installation time than welding. Several types of mechanical couplers have been developed and commercially available. Five selected mechanical couplers are discussed in this Chapter:

1. Shear Screw Couplers (SSC)
2. Headed Bar Couplers (HC)
3. Gouted Sleeve Couplers (GS)
4. Threaded Couplers (TC):
5. Swaged Couplers (SC)

The selection was based on: Caltran’s pre-qualified list of proprietary mechanical couplers with accepted performance and availability of experimental data for the specific couplers. Figure 2.1 depicts the selected mechanical couplers. Of these couplers, the grouted sleeve (GS) couplers were the focus of the study.
In this Chapter, requirements and limitations of US codes on application of mechanical couplers are presented. Then a comprehensive literature is conducted on available experimental tests data regarding the performance of mechanical coupler (specifically the five listed couplers). After that the experimental program to test the behavior of GS coupler that connect the reinforcing bars is explained.

2.2 Current US Code Requirements for Mechanical Splices

The mechanical bar couplers have been widely utilized in the connections of reinforced concrete members. However, most code requirements prohibit or allow their use with limitations regarding the type of the coupler and the location where it can be used. Table 2.1 presents the current code requirements and limitations for the mechanical splices. The codes evaluate the couplers based on: stress, strain and slip limits depending on the specified code. GS
coupler is classified as “Type 2”, “Full Mechanical Connection”, and “Service” bar splice according to ACI 318, AASHTO, and Caltrans, respectively.

Table 2.1: US Codes restrictions on mechanical bar splices

<table>
<thead>
<tr>
<th>Code</th>
<th>Splice Type</th>
<th>Stress Limit</th>
<th>Strain Limit</th>
<th>Slip Limit</th>
<th>Location Restriction</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 318</td>
<td>Type 1</td>
<td>1.25 $f_y$</td>
<td>None</td>
<td>None</td>
<td>Shall not be placed within a distance equal to twice the member depth from critical sections where yielding is likely to occur</td>
</tr>
<tr>
<td></td>
<td>Type 2</td>
<td>1.0$f_u$</td>
<td>None</td>
<td>None</td>
<td>Shall be permitted at any location but shall not be located less than half the member depth from the joint face</td>
</tr>
<tr>
<td>AASHTO</td>
<td>Full Mechanical Connection</td>
<td>1.25 $f_y$</td>
<td>None</td>
<td>#3-14: 0.01&quot;</td>
<td>Shall not be placed in plastic hinge of columns in SDC C and D</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>#18:0.014&quot;</td>
<td></td>
</tr>
<tr>
<td>Caltrans</td>
<td>Service</td>
<td>None</td>
<td>2%</td>
<td>$\epsilon_y$</td>
<td>Service splices are permitted in capacity protected members.</td>
</tr>
<tr>
<td></td>
<td>Ultimate</td>
<td>9% for bar $\leq$ #10 , 6% for bar $\geq$ #11</td>
<td>$&lt;2%$</td>
<td></td>
<td>Ultimate splices are permitted outside the plastic hinge zone for ductile members.</td>
</tr>
</tbody>
</table>

Mechanical bar couplers have been widely used in capacity protected structural members. Their use in critical regions like plastic hinge regions is banned. In the last few years, there
has been an increasing interest to utilize bar couplers in precast bridge column connections. Noting that columns are allowed to undergo substantial nonlinearity while bridge collapse is prevented, such application for bar couplers needs sufficient test data and special detailing. To be able to incorporate these couplers in bridges and especially bridge columns, seismic performance of couplers, seismic performance of columns with these couplers, and new guidelines and specifications are necessary.

### 2.3 Past Studies on Coupler Performance

#### 2.3.1 Shear Screw Coupler (SCC)

The mechanical performance of SCCs was investigated by Lloyd [42]. The SSC used in the study was Bar-Lock L-Series. Two ASTM A615 Grade 60 bars were used: # 6 and # 8. Monotonic and cyclic tensile tests were conducted; 40 samples per bar size for each test. Monotonic test was performed following ASTM A370 and ASTM E8. Cyclic test protocol consisted of applying 100 cycles with a stress range of 5-90% of the specified yield strength (60 ksi). The monotonic test results showed that SSC specimens for both bar sizes (# 6 and # 8) achieved ultimate tensile strength comparable to the control specimens. Several failure modes were noticed: bar rupture outside the coupler region but inside the gauge length, bar rupture outside coupler region and gauge length, bar rupture at the first screw inside the coupler, bar fracture inside the coupler and bar pullout from the sleeve.

Under cyclic loading, all specimens passed the test without failure. Moreover, some of these specimens were selected randomly to be tested under additional 100 cycles. None of these specimens failed or showed signs of degradation. Also, eight random specimens that experienced 100 cycles of loading were tested monotonically to failure to examine the
residual strength. They all exhibited ultimate strengths similar to the control specimens. Slip measurements from cyclic tests showed that the accumulated slip was less than 0.0015 in. The results showed that the Bar-Lock coupler is a good alternative mechanical splice from a strength point of view.

Huaco Cárdenas [35] investigated two types of SCCs, six-screw coupler (short splice) and eight-screw coupler (long splice) which were used to connect Grade 60 ASTM A706 and ASTM A615 # 8 steel bars. The short splice had pointed screws, while the long splice had the edge screws rounded and the rest of the screws pointed. Two cyclic tests were performed. The first was axial cyclic tension test on four specimens: two long splices and two short splices connecting ASTM A706 bars. Progressive load control cycles was used until failure. Two cycles was applied for each load value. The second test was compression-tension cyclic test on ASTM A615 spliced bars. Two long splices and four short splices were used. A displacement control protocol was applied based on the yield displacement of the bar. Cyclic tests showed that the all specimens except one developed tensile strength that satisfy ACI 318 and AASHTO strength requirements for mechanical splices listed in Table 2.1. The single specimen that did not meet the codes requirement was a short splice specimen that failed due to shear failure of the bolts under compression-tension cyclic test. Both cyclic tests showed that the long SCC splice specimens failed due to bar rupture outside the coupler region, while the short splice specimens failed due to bar rupture at the splice edge next to the first bolt.

The performance of SSC was tested by Yang et al. [75]. The steel bars that were spliced by the couplers were # 8 ASTM A615 Grade 60 steel bars. Three specimens were investigated using static tensile test. Results showed that the average ultimate load was comparable to the reference control bars. Two specimens failed by bar fracture inside the coupler at the exterior bolt, while one sample failed by bar fracture away from the coupler.
2.3.2 Grouted Sleeve Couplers (GS)

Noureddine [51] investigated the strain capacity of GS specimens connecting steel bars. Four specimens were tested utilizing #18 bars: two ASTM A615 and two ASTM A706 Grade 60 steel bars. Monotonic tensile testing was performed on the specimens to failure. Results showed that the ultimate strength of the GS coupled bars were comparable to that of the control bars for both steel types. Also, the coupled bars achieved an approximate average strain of 7% and 12% for ASTM A615 and ASTM A706 bars, respectively, which met the minimum limits of ASTM requirement for the corresponding steel types. The grouted couplers were specified as “Class I” splice for the achieved strain capacity and they were recommended for use in plastic hinge zones in moderate to high ductility demand regions.

Michigan Department of Transportation (MDOT) [36] studied the performance of epoxy coated bars spliced with two types of grouted couplers: threaded-grouted coupler (TGS) and splice sleeve grouted coupler (GS). Three #6 and three #11 specimens were fabricated for each splice type and tested following ASTM A1034. Test protocol was slip, fatigue, post-fatigue slip and ultimate load tests for each specimen. The specimens were subjected to slip test according to ASTM A1034. Then fatigue testing was applied in tension with a stress range of 18 ksi and frequency of 9 Hz for 1 million cycles. If a specimen survived the fatigue loading, another slip test was performed. Finally the specimen was subjected to static monotonic tensile test to failure.

Results showed that TGS specimens achieved average slip of 0.004 in and 0.005 in for #6 and #11, respectively. GS specimens achieved average slip of 0.007 in and 0.009 in for #6 and #11, respectively. Both splices achieved less than the slip limit in AASHTO LRFD standard for No. 6 and No. 11 steel bars, which is 0.01 in. All specimens for both mechanical splices were able to withstand the fatigue loading without failure and pass the second slip test. The
average ultimate load for TGS specimens was 169% and 148% of the yield strength ($f_y$) of the bars for #6 and #11, respectively. GS specimens achieved ultimate load of 166% and 175% of $f_y$ for #6 and #11, respectively. Both splices achieved ultimate load more than the specified limit in AASHTO LRFD requirement. Different failure modes were observed. For TGS specimens, the common failure location was at the threaded connection either by bar fracture at the reduced threaded section or by thread shear failure. For GS specimens, the failure modes were either pull out of one the reinforcing bars in the specimen from the sleeve, bar fracture outside the sleeve region, or sleeve fracture in the middle. However, the failure mode did not have noticeable effect on the ultimate strength of the samples. Both splices performed well under slip, fatigue and ultimate strength; therefore, they were recommended for MDOT use.

Haber [28] conducted tensile tests on GS splices connecting #8 ASTM A615 Grade 60 bars. Static, dynamic, cyclic and slip tests were performed. Three specimens were tested for each protocol except the cyclic loading which had one specimen. Static test followed ASTM A370 with two displacement rates: pre-yield and post-yield rates. Dynamic testing was performed with a strain rate of (70000 $\mu$ε/sec) that resemble an earthquake experience. The cyclic loading was tension-compression cyclic loading with a constant compression stress of 3 ksi and a tension stresses which were increments of the bar yield strength. Slip test followed California Test (CT) 670 standard.

Static test showed that GS specimens developed average ultimate strength comparable to the control bars. Also, the coupler region showed an average ultimate strain of 5.3% which was lower than the ultimate strain of the bar by 66.7%. Dynamic loading results showed that the average ultimate strength was slightly higher than that in static test due to loading rate effect. The strains in all regions were comparable to those in static test. Slip test showed that all three specimens exhibited slip less than 0.014 in, which is Caltrans’ slip criterion.
Result of the single specimen tested cyclically showed that the load and ultimate strain reduced by 11% and 52%, respectively, compared with the static test. Also, it was noticed that the bar rupture at failure did not experience necking in the fracture zone which meant non-ductile failure as opposed to the bar rupture that occurred in the static test (ductile failure). All samples failed due to bar rupture away from the coupler region. GS coupler exhibited consistent results for static and dynamic tests.

### 2.3.3 Threaded Couplers (STC) and (TTC)

Noureddine [51] conducted tensile tests on #18 ASTM A615 and A705 Gr. 60 reinforcing steel bars connected with tapered threaded couplers (TTC); two specimens per bar type. The specimens were tested under monotonic tensile loading to failure. The average ultimate load for TTC samples was $1.46f_y$, where $f_y$ is the specified yield strength of the spliced bars, which was 15% smaller than those observed for the control bars. TTC samples achieved strain less than 4% in the spliced bars prior to failure, which was low. The samples failed by stripping of the threads.

Rowell et al. [62] tested eighteen mechanical splices samples which incorporated nine tapered threaded couplers (TTC) specimens and nine straight threaded couplers (STC) specimens to connect #10 ASTM A615 Gr. 60 steel bars. Both types of splicing couplers were subjected to three strain rates, slow-rate (3000-4000 $\mu$ε/sec), intermediate-rate (62000-65000 $\mu$ε/sec), and high-rate tests (3.2-3.8x10⁶ $\mu$ε/sec).

At slow strain rate, the average ultimate strength for both splice types were comparable to those of the control reference bars. For TTC specimens, the average developed strain was 11% compared with 10% measured for the control bars. For STC specimens, the average developed strain was 7%, which was lower than the control bar ultimate strain. At
intermediate-strain rate, the average ultimate strength and strain for TTC specimens were 8% and 50% lower than the control bars, respectively. For STC specimens, ultimate strength was comparable to the control bars, while ultimate strain had a reduction of 10% compared with the control bar. At rapid-strain rate, the average ultimate strength and strain for TTC specimens had a reduction of 24% and 86%, respectively, compared with the control bars. For STC specimens, the ultimate load was comparable to the control bars. The maximum measured strain had a reduction of 8% compared to the control specimens.

The failure mode for STC specimens was due to bar fracture outside the splice for all strain rates. TTC specimens had several failure modes. At slow strain-rate, all samples failed due to bar rupture outside the splice. Under intermediate strain rate, two samples failed due to bar fracture away from the splice and one sample failed due to a bar fracture at the last few threads just outside the coupler. At rapid-strain rate, all specimens failed by bar rupture outside the coupler at the last few threads.

2.3.4 Headed Couplers (HC)

Rowell et al. [62] investigated the performance of headed bar coupler (HC) splicing # 10 ASTM A615 Grade 60 steel bars. Three levels of strain rates were used similar to those presented earlier for threaded couplers. Three specimens were tested for each strain rate and compared to control samples under the same strain rates.

At slow rate tests, the average ultimate tensile strength for HC specimens were comparable to the control bars. Also, HC specimens developed a strain capacity of 110% compared with control bars. At intermediate-strain rate tests, HC specimens achieved ultimate strength comparable to the their control bars. An average reduction in ductility of 40% was observed compared to the control specimens. At rapid-strain rates, HC specimens achieved ten-
sile strengths close to the controls. The specimens had reduced ductility of approximately 50%. HC specimens failed by bar rupture outside the heat-affected zone of the coupler at slow-strain rates. Bar rupture was either outside or inside the heat-affected zone under intermediate and high-strain rates.

Haber [28] conducted monotonic, cyclic, and dynamic tests on # 8 ASTM A706 Gr. 60 steel bars connected using headed bar couplers (HC). Four specimens were tested under static and dynamic loadings while only two specimens were tested under cyclic loading. The mode of failure was bar fracture outside the coupler zone in all specimens. It was observed that the couplers allowed steel bars to achieve their ultimate strains. The measured average strain over the coupler region was 7.7% in the static tests. Similar behavior and failure mode were observed in the cyclic tests.

2.3.5 Swaged Couplers (SC)

Noureddine [51] tested samples of swaged couplers (SC) splicing # 18 reinforcing steel bars. Four samples were tested: two ASTM A615 and two ASTM A706 Grade 60 steel bars. Static tensile test to failure was performed to investigate load and strain capacity of SCs. Results showed that the average ultimate strength for the coupler was comparable to the control bars. The average ultimate strain was 8 and 9% for A615 and A706 specimens, respectively. The swaged coupler was classified as “Class I” coupler, which indicates that the coupled bars can develop a minimum of 7% and 10% strain for ASTM A615 and A706, respectively. Two different failure modes were noticed. Couplers splicing A615 bars failed by bar pullout from the coupler, while couplers splicing A706 bars failed due to bar fracture away from coupler.

Yang et al. [75] investigated the performance of SCs through tensile testing of four specimens: three fully swaged and one specimen was 1 in unswaged from one side only. The connected
steel bars were #8 ASTM A615 Grade 60 steel bars. Results showed that the average ultimate load was comparable to the reference control bars. All specimens failed due to bar rupture away from the coupler region and the rupture was preceded by necking. The unswaged sample showed similar behavior and failure mode to the swaged samples.

2.4 Bond-slip Behavior

Bond between reinforcing bar and concrete is crucial for the performance of RC structures. Load is transferred from the deformed bar to the surrounding concrete by three mechanisms: (1) chemical adhesion between the bar and concrete, (2) frictional force developed from the roughness of the interface, and (3) bearing of the ribs against the concrete surface [4].

When the bar initially slips relative to surrounding concrete, surface adhesion is lost and bearing forces on the bar ribs and friction forces on the ribs and barrel of the bar are mobilized. When slip increases, friction on bar barrel reduced, making the forces at the contact faces between the ribs and the surrounding concrete as the principal mechanism of force transfer.

Bond force and thus bond stress distribution along member is dependent on the rate of change of tensile force in the bar. For a flexural member, bar tensile force varies significantly due to presence of cracks thus influencing bond stress distribution. Even for a member with a pure tensile force, the bond stress, \( u \), is nonlinear because of the local concrete cracks adjacent to the bar ribs. ACI 318 [3] limits the maximum bond stress using the formula given in Equation 2.1. For design purposes, it is convenient to assume a uniform bond stress along the member. Therefore, researchers and engineers tend to use ACI 318 formula for
that assumption.

\[ u = 9.5 \frac{f_c'}{d_b} \]  

(2.1)

Where \( f_c' \) is the concrete compressive strength and \( d_b \) is the reinforcing bar diameter. However, the bond behavior between the reinforcing bar and any cementitious material is best described by a local shear stress-slip behavior, also called bond slip model. One of the earliest studies to describe the bond behavior of concrete-steel bar interface was done by Eligehausen et al. [25]. The bond slip model is described by Equation 2.2

\[
\begin{align*}
    u &= \begin{cases} 
        u_1 \left( \frac{s}{s_1} \right)^\alpha & 0 < s \leq s_1 \\
        u_{max} & s_1 < s \leq s_2 \\
        \frac{u_3 - u_1}{s_3 - s_2} (s - s_2) + u_1 & s_2 < s \leq s_3 \\
        u_3 & s > s_3 
    \end{cases}
\end{align*}
\]

(2.2)

Where \( s_1 = 0.04 \text{ in}, \ s_2 = 0.12 \text{ in}, \ s_3 = 0.43 \text{ in}, \ u_1 = 1.96 \text{ ksi}, \ u_3 = 0.725 \text{ ksi}, \) and \( \alpha = 0.4. \)

The researchers used steel bars #8 Gr. 60 and concrete with \( f_c' = 4.35 \text{ ksi}. \)

Since the innovation of new column-footing or column-beam connections that incorporate advanced cementitious materials such as grouted duct and grouted sleeve connections, new bond slip models were needed to understand and quantify the steel bar slip of such connections.

Thirty-two pullout tests were carried out by Raynor et al. [60] to study the bond behavior of steel bars embedded in grouted ducts. Three different bar sizes were included in the test.
matrix: #6, #8, and #10 ASTM A706 Gr. 60 steel bars. A short embedment length of 2 in was used to ensure elastic bar response and approximately uniform bond stress. Commercial steel post-tensioning ducts with a thickness of 0.02 in and internal diameter of 3 in were used. The grout was reinforced with long fibers (0.75 in long) and had average compressive strength of 7 ksi at the testing day. Using the test results and the aid of one-dimensional finite element modeling, Raynor et al. [60] developed a local bond slip model for the grouted duct assembly.

Steuck et al. [71] did a similar study to the work of Raynor et al. [60]. The main objective was to examine if larger bar diameters would satisfy the described behavior by the latter researchers. The study involved experimental testing as well as one-dimensional numerical modeling. Fourteen pullout tests of large diameter bars embedded in the grout-filled ducts were performed. Bar sizes of #10, #14, and #18 ASTM A706 Gr. 60 were used. Other testing parameters involved: the effects of embedment length (varied from $2d_b$ up to to $14d_b$), and presence of fibers. A corrugated steel pipe was used with a wall thickness of 0.064 in and a nominal diameter of 8 in. The testing day compressive strength of grout ranged from 7500 psi to 10000 psi. Similar to the model developed by Raynor et al. [60], a bond slip model was developed using the same approach. A distinct feature in Steuck’s model is that the local slip is related to the bar diameter. Figure 2.2 depicts the three aforementioned bond slip models. The plots show the normalized shear stress with respect to concrete or grout strength against the normalized slip with respect to the bar diameter.
GS couplers are being increasingly used in column-footing connections for their good construction tolerance and a excellent load-transfer mechanism between the connecting members. Therefore, understanding bond slip behavior of these couplers is important to quantify and understand the lateral displacement of the column. Previous research have not addressed this topic yet. In the present study, experimental and numerical programs are set to cover this topic in the subsequent sections.

2.5 Experimental Program

Behavior of GS couplers was investigated by means of axial loading. The experimental program was divided in two phases; each phase coincided with the corresponding column testing phase. The first phase consisted of monotonic static tensile testing of GS couplers
and it was conducted in Splice Sleeve Japan (SSJ) facility in Japan. The goal was to characterize the tensile behavior of the spliced bars and determine their failure mode. The second phase consisted of monotonic tensile static testing and strain penetration (slip) testing of GS couplers. It was conducted in the Structures Laboratory at the University of Central Florida using Instron Universal Testing Machine (UTM) with a capacity of 224.8 kip. The goal of phase two was to expand the knowledge of the GS coupler behavior under tensile loading and also quantify the strain penetration of the spliced bars into the grout. The tensile test and the strain penetration test were performed in displacement control mode.

2.5.1 Phase One - Uniaxial Tensile Test

2.5.1.1 Specimen Details

Three sets of tensile test specimens (refer to Table 2.2) were prepared and tested each having three samples. The first identifier in the specimen ID refers to bar that represents the column longitudinal reinforcement (the factory end). The second identifier in the specimen ID denotes the bar that represents the footing dowel (the field end). The number indicates the bar size and the letter refers to the bar Grade. Letters “N” and “H” indicate steel Grade as 60 and 100, respectively. The first set, denoted 6N-7H, employed the same bars sizes and splice configuration as G-40-1 column model. That is, an ASTM A615 Grade 60 #6 bar was inserted in the factory end, an ASTM A1035 Grade 100 #7 bar inserted into the field end, and the bars where spliced with a GS sleeve design for #7 bars. The second set, denoted 5N-6H, utilized the same bars sizes and splice configuration as G-25-1 column model. That is, an ASTM A615 Grade 60 #5 bar was inserted in the factory end, an ASTM A1035 Grade 100 #6 bar inserted into the field end, and a GS sleeve #7 was used to spliced the bars. The third set, denoted 11N-14H, had a Grade 60 #11 bar spliced with a Grade 100 #14 using a
splice designed for #14 bars. This sample set had $i_T = 3.0$ and was prepared to investigate up-scaling behavior of transition splicing. Furthermore, this configuration is representative of what might be deployed in actual precast bridge columns.

Table 2.2: Phase one tensile test matrix for spliced bar assemblies

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Sleeve Size</th>
<th>Top Bar</th>
<th>Bottom Bar</th>
<th>Transition Index ($i_T$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Size</td>
<td>Grade (ksi)</td>
<td>Size</td>
</tr>
<tr>
<td>6N-7H</td>
<td>#7</td>
<td>#6</td>
<td>60</td>
<td>#7</td>
</tr>
<tr>
<td>5N-6H</td>
<td>#6</td>
<td>#5</td>
<td>60</td>
<td>#6</td>
</tr>
<tr>
<td>11N-14H</td>
<td>#14</td>
<td>#11</td>
<td>60</td>
<td>#14</td>
</tr>
</tbody>
</table>

2.5.1.2 *Specimen Construction and Materials*

A special horizontal steel frame was used to fabricate the spliced specimens (see Figure 2.3). The frame is designed in such a way that maintain the alignment of the coupler and the reinforcing bars which are inserted in both ends of the coupler. The horizontal alignment is controlled using adjustable holders on different locations on the main frame. The coupler is placed such that the inlet and outlet ports are facing upward. After the coupler assembly was aligned and fixed against any movement, the coupler end ports were covered with a duct tape. Then a high-strength, non-metallic, cementitious grout was mixed according to manufacturers’ specifications for at least 2.5 min. The grout was poured from the inlet port (the port close to the field dowel) until it exit from the outlet port (the port close to the factory dowel), which indicated that the coupler was filled with the grout.
2.5.1.3 Instrumentation and Testing

The uniaxial tensile test set-up for the spliced bars was developed according to ASTM A1034 [7] and California Test CT 670 [23]. Specimens were instrumented with foil-backed strain gages such that longitudinal strains were captured from NS and HS bars, and at the mid-height of the GS sleeve (Figure 2.4). The overall deformation of the spliced bar assembly was captured using a digital extensometer. The extensometer gage length, which is also referred to as the “coupler region”, or $L_{cr}$, included the entire GS length ($L_{sp}$), and approximately 0.8 in of bar length on either end of the coupler. The clear unsupported length of the specimen ($L_{clear}$) was determined based on laboratory testing machine capabilities. Spliced bar assemblies were loaded monotonically until failure. Instrumentation plan and test set-up are shown in Figure 2.4 and Figure 2.5, respectively. The test was controlled using displacement mode and the displacement rates are shown in Table 2.3.
Figure 2.4: Instrumentation plan for the tensile test of the GS spliced bars
Table 2.3: Displacement rates for static monotonic tension test for phase one

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Displacement Rate (in/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5N-6H</td>
<td>0.03218</td>
</tr>
<tr>
<td>6N-7H</td>
<td>0.0338</td>
</tr>
<tr>
<td>11N-14H</td>
<td>0.046</td>
</tr>
</tbody>
</table>

2.5.2 Phase Two - Uniaxial Tensile and Strain Penetration Tests

2.5.2.1 Specimen Details

Sixteen spliced bar assemblies were fabricated in the Large-scale Structures Laboratory at the University of Central Florida in order to perform uniaxial tensile and strain penetration
tests. The test matrix is shown in Table 2.4. Eight configurations were considered each had two specimens. The first identifier in the specimen ID refers to bar that represents the column longitudinal reinforcement (the narrow end of the sleeve). The second identifier in the specimen ID denotes the bar that represents the footing dowel (the wide end of the sleeve). The number indicates the bar size and the letter refers to the bar Grade. Letters “N” and “H” indicate steel Grade as 60 and 100, respectively. The spliced configurations included splicing same bar Grades with $i_T = 0$, splicing different bar Grades with $i_T = 1$ and $i_T = 3$.

Table 2.4: Phase two tensile test matrix for spliced bar assemblies

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Sleeve Size</th>
<th>Bottom Bar</th>
<th>Top Bar</th>
<th>Transition Index ($i_T$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Size</td>
<td>Grade (ksi)</td>
<td>Size</td>
</tr>
<tr>
<td>5N-6H</td>
<td>#6</td>
<td>#5</td>
<td>60</td>
<td>#6</td>
</tr>
<tr>
<td>6N-7H</td>
<td>#7</td>
<td>#6</td>
<td>60</td>
<td>#7</td>
</tr>
<tr>
<td>6N-9H</td>
<td>#9</td>
<td>#6</td>
<td>60</td>
<td>#9</td>
</tr>
<tr>
<td>11N-14H</td>
<td>#14</td>
<td>#11</td>
<td>60</td>
<td>#14</td>
</tr>
<tr>
<td>5N-5N</td>
<td>#5</td>
<td>#5</td>
<td>60</td>
<td>#5</td>
</tr>
<tr>
<td>6N-6N</td>
<td>#6</td>
<td>#6</td>
<td>60</td>
<td>#6</td>
</tr>
<tr>
<td>7N-7N</td>
<td>#7</td>
<td>#7</td>
<td>60</td>
<td>#7</td>
</tr>
<tr>
<td>11N-11N</td>
<td>#11</td>
<td>#11</td>
<td>60</td>
<td>#11</td>
</tr>
</tbody>
</table>

2.5.2.2 Construction and Materials

The specimens were fabricated using a special vertical wooden frame. The frame was shored to maintain its plumb position. After that, the bottom bar and coupler were tied to the frame (Figure 2.6). Small wooden shims were used behind the coupler to ensure the alignment.
Then the top bar was inserted in the coupler and tied to the frame as well (Figure 2.7).

Coupler specimens were filled with a high-strength, non-metallic, cementitious grout provided by the manufacturer. The grout was mixed according to manufacturers’ specifications. After the grout was properly mixed for 2.5 min and poured into the couplers, the excess grout was cleaned from the coupler top end. The specimens were allowed to cure for five days before removal from the frame. During preparation of GS specimens, six grout cubes were cast according to ASTM C109 [6] to measure the compressive strength of the grout ($f'_g$). The average ($f'_g$) at the day of testing was around 13000 psi.

Figure 2.6: Grouted coupler test specimens during installation
2.5.2.3 Instrumentation and Testing

The test set-up for the spliced bar assemblies was developed following ASTM A1034 [7] and California Test CT 670 [23]. Specimens were instrumented with strain gages to measure the longitudinal strains of NS and HS bars. The strain gauges were 1 in away from the sleeve ends. The overall deformation of the spliced bar assembly was captured using an extensometer having a gage length, which is also referred to as the “coupler region”, or $L_{cr}$, equal to the entire GS length ($L_{sp}$) plus approximately 2 in of bar length on each end of the coupler. $L_{clear}$ is the unsupported length between the machine grips for the specimen and it was determined
using Equation 2.3 which was adopted from CT 670 [23]. At each end of the GS, the bar strain penetration in the grout was measured with a set of LVDTs mounted within a length of 2 in from the sleeve ends. The net strain penetration was determined by subtraction the strain gauge deformation from the readings of strain penetration LVDTs. Spliced bar assemblies were loaded monotonically until failure. The instrumentation schematic and test set-up are shown in Figure 2.8 and Figure 2.9, respectively. Loading rates used in this test were determined according to ASTM E8 [10].

\[ L_{\text{clear}} = 8d_b + L_{sp} + 16 \]  \hspace{1cm} (2.3)

Equation 2.3 uses inch as a unit and \( d_b \) is the larger bar diameter that was used in the splice.

Figure 2.8: Instrumentation plan for the tensile and SP tests of the GS spliced bars
Figure 2.9: GS spliced bars test setup
CHAPTER 3: COLUMN DESIGN METHODOLOGY

3.1 Introduction

The proposed design concept was evaluated by experimental testing of six large-scale bridge column models, and a series of uniaxial tensile and strain penetrations tests on spliced bars assemblies. All six columns models were designed to be representative of highway bridge columns used in short- to medium-span concrete bridges found in California, and were of varying scale. The experimental program was executed in two phases. Phase one examined proof of concept, and phase two provided supplemental testing for development of design expression and calibration of analytical models. Both phases contained column testing and uniaxial tensile testing.

3.2 Phase One Experiments

3.2.1 Large-Scale Column Tests

The primary goal for phase one column testing was to demonstrate proof of concept, and investigate the effect of column aspect ratio. A total of four columns were designed, constructed, and tested. Each column model was approximately 0.42-scale assuming a full-scale bridge column has a diameter of 48 in. A test matrix is shown in Table 3.1. The first letter of the column ID, “C” or “G”, denotes if the column was CIP or precast with GS connections, respectively. The second identifier indicates that column aspect ratio. Herein the aspect ratio (AR) is defined as the ratio between cantilever level (distance between the footing surface and loading point) and the column diameter. The two ARs tested were 2.5 denoted by
“25” and 4.0 denoted by “40”. The last identifier, “1”, indicates the columns were tested in phase one. Table 3.1 also provides some basic information regarding reinforcement details, which will be discussed in detail in the subsequent section.

<table>
<thead>
<tr>
<th>Column ID</th>
<th>Construction Method</th>
<th>Aspect Ratio</th>
<th>Diameter (in)</th>
<th>Reinforcing Ratios</th>
<th>Longitudinal Bar Sizes</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-25-1</td>
<td>CIP</td>
<td>2.5</td>
<td>20</td>
<td>1.43%</td>
<td>0.50%</td>
</tr>
<tr>
<td>G-25-1</td>
<td>Precast</td>
<td>2.5</td>
<td>20</td>
<td>1.0%&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.50%</td>
</tr>
<tr>
<td>C-40-1</td>
<td>CIP</td>
<td>4.0</td>
<td>20</td>
<td>1.95%</td>
<td>0.74%</td>
</tr>
<tr>
<td>G-40-1</td>
<td>Precast</td>
<td>4.0</td>
<td>20</td>
<td>1.43%&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.74%</td>
</tr>
</tbody>
</table>

<sup>a</sup> Denotes reinforcement ratio within the column shaft
<sup>b</sup> High-strength steel - ASTM A1035 Grade 100
<sup>c</sup> Normal-strength steel - ASTM A615 Grade 60

3.2.1.1 Design and Detailing

Column models were designed according to the Caltrans’ SDC [18]. Although the SDC currently prohibits the placement of mechanical reinforcing bar splices (i.e., the grouted splices) within plastic hinge zones, previous studies on GS connections used the SDC as the primary design criteria [30, 72]. Thus, the SDC was used to increase the body of knowledge on these connections, and to maintain some continuity with previous work.

Pertinent details of the four column models are shown in Figure 3.1 through Figure 3.4. The baseline CIP columns, denoted C-40-1 and C-25-1, were designed to have a target displacement ductility capacity of $\mu_c \approx 7.0$, determined using the elasto-plastic analysis procedure outlined in the SDC. Moment-curvature analysis was conducted using OpenSees using expected material properties defined in the SDC and presented in Table 3.2 and Table 3.3. The concrete core was modeled using a confined concrete model represented by Mander’s model [46] and the concrete cover was modeled using Kent-Scott-Park concrete model [66]. The
material models in OpenSees are “Concrete04” and “Concrete01” for confined and unconfined concrete, respectively. The Gr. 60 steel bars were modeled using “ReinforcingSteel” material which is based on Chang and Mander model [20]. The design axial load was 126 kip, which corresponds to an axial load index (ALI) of 0.08. The ALI is the ratio of the axial load to the product of the column section area and the compressive strength of concrete.

**Loading Direction**

![Diagram of loading direction](image)

**Figure 3.1: C-40-1 column design details**
Figure 3.2: G-40-1 column design details
Figure 3.3: C-25-1 column design details
Figure 3.4: G-25-1 column design details
Table 3.2: Concrete material properties for moment-curvature analysis

<table>
<thead>
<tr>
<th>Material</th>
<th>Unconfined Concrete</th>
<th>Confined Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column</td>
<td>$f'_c$ (ksi)</td>
<td>$f'_{cu}$ (ksi)</td>
</tr>
<tr>
<td>C-40-1</td>
<td>5</td>
<td>0</td>
</tr>
<tr>
<td>G-40-1</td>
<td>5</td>
<td>0</td>
</tr>
<tr>
<td>C-25-1</td>
<td>5</td>
<td>0</td>
</tr>
<tr>
<td>G-25-1</td>
<td>5</td>
<td>0</td>
</tr>
</tbody>
</table>

Note: All values are used as negative input in OpenSees

Table 3.3: Steel material properties for moment-curvature analysis

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>$f_{ye}$ (ksi)</th>
<th>$\epsilon_{ye}$</th>
<th>$E_s$ (ksi)</th>
<th>$E_{sh}$ (ksi)</th>
<th>$f_{ue}$ (ksi)</th>
<th>$\epsilon_{su}$</th>
<th>$\epsilon_{sh}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel (Gr. 60)</td>
<td></td>
<td>68</td>
<td>0.0023</td>
<td>29000</td>
<td>1305</td>
<td>95</td>
<td>0.09</td>
<td>0.009</td>
</tr>
</tbody>
</table>

Displacement ductility capacity is the ratio of the calculated ultimate column displacement, $\Delta_u$, to the calculated effective yield displacement, $\Delta_y$ and is expressed in Equation 3.1.

$$\mu_c = \frac{\Delta_u}{\Delta_y}$$

Moment-curvature analysis was used along with a lumped plasticity approach to determine effective yield and ultimate displacements for ductility calculations. The plastic hinge length, $L_p$, which is required for lumped plasticity analysis, was that proposed by Paulay and Priestley [56] and is defined in Equation 3.2:

$$L_p = 0.08L + 0.15f_{ye}d_{bl} \geq 0.3f_{ye}d_{bl}$$

where $L$ is the column cantilever length measure from the footing surface to the point of
load application, $f_{yc}$ and $d_{bl}$ are the expected yield strength and diameter of the column longitudinal bars, respectively.

The design procedure starts by selecting initial longitudinal and transverse reinforcements that approximately achieve the target displacement ductility of 7.0. For each trial of selection, moment-curvature analysis was performed on the column section to determine the effective yield and ultimate curvatures. Then the yield and ultimate displacements were obtained using Equation 3.3 and Equation 3.4:

$$
\Delta_y = \phi_y \frac{L^2}{3} \quad (3.3)
$$

$$
\Delta_u = \Delta_y + (L - 0.5L_p)L_p(\phi_u - \phi_y) \quad (3.4)
$$

where $\phi_y$ is the effective yield curvature of the column, $\phi_u$ is the ultimate curvature of the column, and $L_p$ is the plastic hinge length.

After the determination of $\Delta_y$ and $\Delta_u$, the displacement ductility was calculated. If it was far from 7.0, then the longitudinal and transverse reinforcement ratios were varied and the above procedure was repeated.

C-40-1 was reinforced longitudinally with ten ASTM A615 Grade 60 #7 bars, which correspond to a longitudinal reinforcement ratio of 1.95%, and transversely with ASTM 1064 W4.5 plain wire spiral with 1.5 in spacing, which corresponds to transverse reinforcement ratio of 0.74%. The moment-curvature response of the critical section (section A-A) of C-40-1 column is shown in Figure 3.5a.

The calculated displacement ductility was $\mu_c = 7.05$ and the plastic lateral load capacity, $V_p$,
was 42.5 kips. The plastic lateral is defined as $V_p = M_p/L$. Section failure was characterized by crushing of core concrete.

C-25-1 was reinforced longitudinally with ten ASTM A615 Grade 60 #6 bars which correspond to a longitudinal reinforcement ratio of 1.43%, and transversely with ASTM 1064 W4.5 plain wire spiral with 2.25 in spacing, which corresponds to transverse reinforcement ratio of 0.5%. The calculated moment-curvature of C-25-1 column for the critical section (section A-A) is shown in Figure 3.5b.

The calculated displacement ductility was $\mu_c = 8.0$ and the plastic lateral load capacity, $V_p$, was 54.9 kips. The plastic lateral is defined as $V_p = M_p/L$. Crushing of core concrete determined the section failure criteria.

Both columns had 1.75 in of clear cover between the spiral and the exterior of the column shaft. At the prototype level, this corresponds to 4.16 in of clear cover in the shaft, which is slightly larger than what would normally be used for nonaggressive environments. The enlarged cover was used such that the precast columns would have a minimum of 2 in cover at the prototype level within the coupler regions.

The precast columns, denoted G-40-1 and G-25-1, were designed to achieve the same approximate plastic lateral load, $V_p$, capacity as their CIP counterparts assuming the plastic hinge forms above the coupler region. The displacement ductility and plastic lateral load capacity for the precast columns were estimated using moment-curvature analysis for section A-A (critical section) in Figure 3.2 and Figure 3.4 which is above the coupler region.

The existing plastic hinge length expression, Equation 3.2, was used with a few additional assumptions. It is noted that currently available expressions for plastic hinge length are not entirely valid for the proposed design concept. That is, available expressions do not account
for the presence of couplers and their local effects on plastic rotation capacity. Thus, the region containing the GSs was assumed to be rigid, and the columns were assumed to behave emulative to CIP columns from the top of the GSs upward.

Figure 3.5: Moment-curvature response of CIP columns

To achieve the aforementioned objective, the sections above the coupler region (section A-A in Figure 3.2 and Figure 3.4) were reinforced with ten ASTM A615 Grade 60 #6 bars in the case of G-40-1, and ten ASTM A615 Grade 60 #5 bars in the case of G-25-1. For G-40-1 and G-25-1 these reinforcement configurations correspond to longitudinal reinforcement ratios of 1.43% and 1.0%, respectively. The transverse reinforcement ratios for the precast columns were similar to their corresponding CIP columns. The longitudinal steel ratio above the coupler region was reduced since the distance between the critical section (section A-A) and point of applied lateral load (mid-height of the leading head) was shorter for the precast columns compared to the CIP columns. Herein, this length is referred to as the effective moment arm ($L'$), which is defined in Equation 3.5. The calculated moment-curvature of the critical section (section A-A) of the precast columns is shown in Figure 3.6. Like the
case in CIP columns, core concrete crushing determined the section failure criteria.

\[ L' = L - L_{sp} \]  \hspace{1cm} (3.5)

where \( L_{sp} \) is the length of the GS splice.

![Graph showing moment-curvature response of precast columns](image)

**Figure 3.6: Moment-curvature response of precast columns**

The calculated plastic lateral load was 41.9 and 58 kips for G-40-1 and G-25-1 columns, respectively. The plastic load was defined as \( V_p = M_p / L' \). The calculated plastic loads were approximately similar to the calculated values of the corresponding CIP columns (42.5 and 54.9 kips for C-40-1 and C-25-1, respectively).

The sections at the column-footing interface (section C-C in Figure 3.2 and Figure 3.4) were reinforced with ten ASTM A1035 Grade 100 #7 bars in the case of G-40-1, and ten ASTM A1035 Grade 100 #6 bars in the case of G-25-1. The reinforcement ratios at these sections correspond to the same longitudinal steel ratios as C-40-1 (1.95%) and C-25-1 (1.43%). The longitudinal reinforcement configurations used for the precast columns both had a transition
index, which is the incremental difference in bar diameter between the spliced bars, $i_T = 1.0$.

A design check was required to ensure the maximum tensile stress in the precast columns footing dowels (longitudinal reinforcement at section C-C) did not exceed the capacity of the GS splices. The footing dowels and longitudinal bars within the columns shaft were connected using a commercially-available grouted coupler designed for ASTM A706 and A615 Grade 60 bars. The GS splices met AASHTO full mechanical connection (FMC) and ACI Type 2 performance requirements; these performance requirements were discussed in Chapter 2. Thus, it was critical that maximum stress in the footing dowels be less than the expected tensile strength $f_{ue}$ of a corresponding ASTM A706 and A615 Grade 60 bar. The Caltrans SDC and AASHTO Seismic Design Specification both indicate that $f_{ue} = 95$ ksi for both bar types. A moment-curvature analysis was conducted on section C-C. Concrete materials were the same as those used in section A-A but HS steel material was defined using data from Shahrooz [70]. When the plastic moment at section A-A is reached, the maximum tensile stress in the footing dowels (section C-C) can be determined in two steps. First, the moment at section C-C is determined from the column moment diagram. Second, the corresponding steel tensile stress is determined from the moment-curvature of section C-C. It was determined that the maximum expected stress in the footing dowels was 91.6 ksi and 74 ksi for G-40-1 and G-25-1 models, respectively.

The footing and the loading head had the same design and details for all column models in this phase. Both were designed to be elastic during the experiment. The footing was 60 in long, 60 in wide, and 30 in deep. The main reinforcement consisted of 11 #6 reinforcing bars (Grade 60) in both directions on top and bottom sides of the footing. The footing contained a block-out concrete region at the bottom side of the footing in the loading direction. The block-out region was intended to provide a space for axial load application. Rectangular
hoops were added to the footing sides next to the block-out region to add to the footing shear strength.

The loading head dimensions were 24 in long, 20 in wide, and 14 in high. The reinforcement consisted of 6 #4 reinforcing bars (Grade 60) evenly spaced in the loading direction, 4 #4 bars evenly spaced in the other direction (perpendicular to loading direction) and 3 #3 bars evenly spaced along the head height.

3.2.1.2 Column Construction and Materials

Construction of all four columns began by building the footing formwork and laying out the bottom mat of footing reinforcing steel. Once the bottom mat was placed, the reinforcing cage for the column shafts (or dowel cages) were built and placed onto the bottom mat of the footing reinforcement cage. The remainder of the footing reinforcement was placed thereafter. The footing concrete was cast first on January 28, 2015 and allowed to cure prior to completing the shaft formwork. The concrete used had a specified 28-day compressive strength of 4 ksi, and a maximum 0.375 in coarse aggregate; all four columns used the same concrete mix from the same ready mix supplier. Prior to casting, the slump of concrete was measured and it was 6 in.

Also, 4 in x 8 in cylindrical samples were fabricated to measure concrete strength. Footing concrete was allowed to cure for seven days prior to beginning construction on the column forms and falsework for the loading head. After the formwork and reinforcement were placed, the concrete was cast for the columns and loading heads on April 27, 2015; the slump was measured and it was 6 in. Also, concrete compression cylinders samples were fabricated prior to casting. All form-work was removed after seven days. Figure 3.7 through Figure 3.10 shows the completed footings and columns. Table 3.4 presents the measured concrete
properties for the footings and column shafts following ASTM C39 [9]. Also, tensile properties of the steel bars used in phase 1 are shown in Table 3.5. The results for each bar size are the average of three samples and tested following ASTM E8 [10] and ASTM A370 [8].

Figure 3.7: Completed footings
Figure 3.8: Completed CIP column
Figure 3.9: Completed precast shaft
Figure 3.10: Precast column base

Table 3.4: Measured concrete compressive strength - phase one

<table>
<thead>
<tr>
<th>Column ID</th>
<th>Pour Date</th>
<th>28 Day Test Day</th>
<th>28 Day Test Day</th>
<th>Testing Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-40-1</td>
<td>01/28/15</td>
<td>4980</td>
<td>9490</td>
<td>09/21/15</td>
</tr>
<tr>
<td>G-40-1</td>
<td>02/28/15</td>
<td>9460</td>
<td>04/27/15</td>
<td>10/09/15</td>
</tr>
<tr>
<td>C-25-1</td>
<td></td>
<td>9430</td>
<td>5120</td>
<td>12/04/15</td>
</tr>
<tr>
<td>G-25-1</td>
<td></td>
<td>9370</td>
<td></td>
<td>11/20/15</td>
</tr>
</tbody>
</table>

Note: All results are reported in psi, and reflect the average of three samples.
Table 3.5: Measured tensile properties of the steel bars - phase one

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Grade (ksi)</th>
<th>( f_y ) (ksi)</th>
<th>( \epsilon_y )</th>
<th>( E ) (ksi)</th>
<th>( f_u ) (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#5</td>
<td>60</td>
<td>64.8</td>
<td>0.002471</td>
<td>26980</td>
<td>106.37</td>
</tr>
<tr>
<td>#6</td>
<td>60</td>
<td>65.27</td>
<td>0.002472</td>
<td>27355</td>
<td>107.27</td>
</tr>
<tr>
<td>#7</td>
<td>60</td>
<td>68.56</td>
<td>0.002576</td>
<td>27286</td>
<td>103.93</td>
</tr>
<tr>
<td>#6H</td>
<td>100</td>
<td>126.1*</td>
<td>0.006387*</td>
<td>28740</td>
<td>161.9</td>
</tr>
<tr>
<td>#7H</td>
<td>100</td>
<td>130*</td>
<td>0.006731*</td>
<td>27535</td>
<td>177.73</td>
</tr>
<tr>
<td>W4.5</td>
<td>-</td>
<td>98.46*</td>
<td>0.004467*</td>
<td>31828</td>
<td>105.87</td>
</tr>
</tbody>
</table>

* denotes that the yield properties are determined using 0.2% offset method

“H” denotes high-strength bars

The installation of precast column shafts took place within the Large-scale Structures Laboratory at the University of Central Florida. Prior to the installation, the footing dowels for G-40-1 and G-25-1 were cut to size; the footing dowels were purposefully constructed longer than required to avoid any situation that may lead to an embedded length less than the minimum required by the manufacturer for the bar dowels in the sleeve.

Dowels were sized according to documentation provided by the GS manufacturer. Once the footing dowels were cut to size, the surface of the footing was cleaned to remove loose concrete and dust. Furthermore, any loose concrete found on the footing dowels was also removed. The footing surfaces were then pre-wetted to achieved a saturated surface dry (SSD) condition (Figure 3.11). Prior to setting the precast column shafts in place, shims were placed to allow for a bedded grout layer 0.25 to 0.5 in. Shims also aided in plumbing the column shafts. Once the shims were placed, a thick layer of bedding grout was placed on each footing, and grout washers were installed on each footing dowel (Figure 3.12). The bedding grout was commercially available Rapid Set cementitious grout. The grout washers
were provided by the manufacturer. These thin aluminum washers have a foam ring on the inside, which allows the washer to grip onto reinforcing bar. These washers prevent bedding grout from entering the grout sleeve coupler.

Once the bedding grout and grout washers were placed, the precast column shaft was lowered onto the footing dowels (Figure 3.13). The column shafts were subsequently shored prior to completing the remainder of the installation process. Prior to grouting the GSs, excess bedding grout was removed (Figure 3.14). The grout used in the GS splices was SS mortar, a proprietary grout system developed by the manufacturer specifically designed for the NMB splice sleeve system. The grout was mixed in a 5-gallon bucket according to manufacturer’s specifications using a 800 RPM electrical hand drill with a cementitious grout mixing paddle attachment. Grout was injected into the couplers using Tsumarl N-18 hand pump, which was provided by the manufacturer. The couplers were filled from the bottom PVC tube, and pumping was not stopped until grout began to flow from the upper PVC tube. Once this occurred, a stopper was placed in the top tube and the grout pump tip was removed from the bottom port and replaced with a stopper (Figure 3.15). During grouting, a series of 2-in diameter grout cylinders samples cast for later testing. The testing day compressive strength of the filler grout was 17.8 ksi and 18.7 ksi for G-40-1 and G-25-1 columns, respectively, and it was obtained according to ASTM C39 [9]. Figure 3.16 shows a photo of the completed connection.
Figure 3.11: Pre-wetting column-footing interface

Figure 3.12: Placement of bedding grout and sleeve washers
Figure 3.13: Precast column placement

Figure 3.14: Completed column placement
Figure 3.15: Pumping grout into the coupler sleeve
Figure 3.16: Completed precast connection
3.2.1.3 Instrumentation and Testing

Column models were instrumented with linear variable displacement transducers (LVDTs) to capture plastic hinge curvatures, shear deformations, and bond-slip rotations (Figures 3.17a and 3.17b). Columns were also instrumented with several layers of strain gauges installed on the longitudinal and transverse reinforcing bars. Column tip deformations were recorded using a series of three string potentiometers mounted on the column loading heads, which could capture displacement and rotation of the head. A generalized instrumentation schematic is shown in Figure 3.18. During testing, data was collected using digital data acquisition system, and was collected at a frequency of 10 Hz.

Column models were tested at the Large-scale Structures Laboratory at the University of Central Florida. Tests were conducted using a single cantilever test set up. Test set-up details are shown in Figure 3.19. Lateral load was applied with a 110 kip servo-controlled hydraulic actuator. During testing, lateral load was measured from a load cell mounted on the servo-controlled actuator, and actuator displacement was recorded via an on-board LVDT. A nominally constant axial load of 126 kip was applied to each column model using a two-way acting hollow-core hydraulic jack and high-strength post-tensioning rod that passed through the center of each column. Fluctuation in axial load was controlled using a pressure relief value, and loads were monitored and recorded using an in-line pressure transducer.
Column models were subjected to slow cyclic loading using a drift-based displacement-control loading protocol (refer to Figure 3.20). Two full push and pull cycles were completed at drift levels of 0.25, 0.5, 0.75, 1, 2, 3, 4, 5, 6, 8 and 10% until failure. Two loading rates were used. A loading rate of 1 in/min was used up to 4% drift, following which, the loading rate was increased to 4 in/min up to failure.
3.3 Phase Two Experiments

This phase was developed after completion of phase one experiments. The goal of phase two testing was to investigate different parameters that affect plastic hinge formation and investigate the strain penetration effects within the grouted coupler sleeve.
Figure 3.19: Column test setup
3.3.1 Large-Scale Column Tests

Two columns were designed, constructed, and tested in phase two. The test matrix for phase 2 is shown in Table 3.6; the precast column tested in phase 1 is also shown for comparison. Both columns tested in phase 2 were precast and had an aspect ratio of AR = 4.0. Thus, these columns were denoted G-40-2 and G-40-3. Column G-40-2 was 0.42-scale and was designed to investigate the effect of $i_T \geq 1.0$. Higher degree of $i_T$ leads to a bigger difference in moment capacity between the section below and above the coupler region; the section below the coupler region would have higher moment capacity. Consequently, the curvature above the coupler region would be higher than the curvature below the coupler region at any drift level. That causes the section below the coupler region to have a more limited curvature than if low degree of $i_T$ would be used. Consequently, curvature over the couple region and strain penetration into the footing would be reduced thus reducing the column lateral displacement. As a result, the column displacement ductility would be reduced.
Table 3.6: Phase two column model test matrix

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>G-40-1</td>
<td>Precast</td>
<td>4.0</td>
<td>20</td>
<td>1.43&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.74</td>
</tr>
<tr>
<td>G-40-2</td>
<td>Precast</td>
<td>4.0</td>
<td>20</td>
<td>1.43&lt;sup&gt;a&lt;/sup&gt;</td>
<td>1.11</td>
</tr>
<tr>
<td>G-40-3</td>
<td>Precast</td>
<td>4.0</td>
<td>16</td>
<td>1.59&lt;sup&gt;a&lt;/sup&gt;</td>
<td>1.13</td>
</tr>
</tbody>
</table>

<sup>a</sup>Denotes reinforcement ratio within the column shaft
<sup>b</sup>High-strength steel - ASTM A1035 Grade 100
<sup>c</sup>Normal-strength steel - ASTM A615 Grade 60

Column (G-40-3) was a 0.33-scale and was designed to investigate the effect of moment gradient on the plastic hinge formation. Moment gradient refers to distribution of the bending moment along the length of the column as a result of applied lateral load. It has been shown in previous studies on plastic hinge formation that the analytical plastic hinge length depends, in part, on the member length [12, 50, 22, 58]. Thus, the length G-40-3 was reduced compared with G-40-2. However, both columns maintained the same aspect ratio of 4.0. To achieve this, the diameter of G-40-3 was reduced from 20 in to 16 in.

### 3.3.1.1 Design and Detailing

Column models were designed according to a simplified displacement based design (DBD) method developed using results from phase 1 testing. A detailed discussion of this method is provided in the next subsection. Similar to the methods presented in the Caltrans’ SDC [18] and AASHTO [1], the simplified method presented here utilized \( M - \phi \) analysis and a lumped plasticity-based approach. The method accounts for the location of the SPH and the deformation within the coupler region.

The pertinent design details of G-40-2 and G-40-3 are shown in Figure 3.21 and Figure 3.22, respectively. Both columns were designed for a target displacement ductility capacity of
$\mu_c \approx 7$. Moment-curvature analysis was conducted on section A-A in Figure 3.21 and Figure 3.22 using the expected material properties defined in Caltrans’ SDC and presented earlier in this Chapter. The compressive strength of concrete was assumed to 6.5 ksi instead of 5 ksi previously. The plastic hinge length, $L_p$, expression used for analysis is explained in the simplified DBD method in the next subsection.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{g402_column_design_details}
\caption{G-40-2 column design details}
\end{figure}
Moment-curvature response of both columns at section A-A is shown in Figure 3.23. The calculated plastic lateral load was 47.2 and 30.4 kips for G-40-2 and G-40-3 columns, respectively. The plastic load was defined as $V_p = \frac{M_p}{L'}$. The calculated displacement ductility was $\mu_c = 6.8$ and 6.95 for G-40-2 and G-40-3, respectively, using the simplified DBD procedure.
To achieve the target design ductility, G-40-2 was reinforced with ten ASTM A615 Grade 60 #6 bars at the section above the coupler region (section A-A in Figure 3.21), which corresponds to longitudinal reinforcement ratio of 1.43%. The section at the column-footing interface (section C-C in Figure 3.21) was reinforced with ten ASTM A1035 Grade 100 #9 bars. The longitudinal reinforcement configuration used for the G-40-2 column had transition index, $i_T = 3.0$. For the transverse reinforcement, G-40-2 was reinforced with ASTM A1064 W4.5 plain wire spiral with spacing of 1 in which corresponds to transverse reinforcement ratio of 1.11%. The design axial load for G-40-2 was 126 kip which corresponds to an $ALI = 0.08$.

G-40-3 had ten ASTM A615 Grade 60 #5 bars at the section above the coupler region, which corresponds to longitudinal reinforcement ratio of 1.59%. The section at the column-footing interface had ten ASTM A1035 Grade 100 #6 bars. The longitudinal reinforcement configuration used for the G-40-3 column had transition index, $i_T = 1.0$. G-40-3 was reinforced transversely with ASTM A1064 W4.5 plain wire spiral with spacing of 1.25 in which corre-
sponds to transverse reinforcement ratio of 1.13%. The design axial load for G-40-3 was 78 kip which corresponds to an \( ALI = 0.08 \).

Similar to the methodology described for phase one tests, the maximum tensile stress in the longitudinal reinforcement (footing dowels) at the column-footing interface (section C-C in Figure 3.21 and Figure 3.22) was checked and made sure that it did not exceed the tensile capacity of the GS couplers. The stress check was done by conducting moment-curvature analyses on section A-A and C-C.

When the plastic moment at section A-A is reached, the moment at section C-C is determined from the column moment diagram. Then the corresponding steel tensile stress at section C-C is determined from the moment-curvature of that section. The maximum expected tensile stress in the footing dowels was 92.8 ksi and 86.9 ksi for G-40-2 and G-40-3 models, respectively.

Design details of the footing and the loading head were similar to the ones in phase one except for one single detail in G-40-3 column. The footing depth had to be decreased from 30 in to 22 in in order to set up the column properly.

### 3.3.1.2 Column Design Procedure

Columns G-40-2 and G-40-3 were designed according to the simplified DBD procedure discussed by Haber et al. [29]. Key aspects of the design procedure are depicted in Figure 3.24. The ductility, \( \mu \), is calculated using Equation 3.6. Each of the displacement components (\( \Delta_y \) and \( \Delta_p \)) in the ductility includes a contribution from the sleeve region (denoted by a subscript “s”) and a contribution from the effective column shaft length (denoted by a prime), which are defined in Equation 3.7 and Equation 3.8. These displacement components can
be determined using the moment-area method, which is commonly employed for DBD of conventional bridge columns. The contribution from the sleeve region \((L_{sl})\) in both effective yield and plastic states can be calculated from Equation 3.9 and Equation 3.10. These equations assume curvature distributions are idealized using elastic and inelastic GS strain ratio parameters denoted by \(SR_E\) and \(SR_I\), respectively. These parameters represent the ratio of strain developed over the splice to the strain developed in the NS steel bars, and can be determined using tensile testing. Chapter 5 describes how these parameters can be determined experimentally. The curvatures \(\phi_{y,A}\) and \(\phi_{p,A}\) are the effective yield and plastic curvatures at section A as determined from sectional analysis, respectively. The contribution from the effective column length \((L')\) can be obtained from Equation 3.11 and Equation 3.12 for the effective yield and plastic states, respectively. The analytical plastic hinge length \((L_p)\) is taken as 0.08\(L'\), and the parameters \(SR_E\) and \(SR_I\) are determined by tensile testing of GS spliced bars. Further design details and assumptions are contained in [29].

Figure 3.24: Displacement ductility of GS precast column with SPH
\[ \mu = \frac{\Delta y + \Delta p}{\Delta y} \] 
\hspace{10cm} (3.6)

\[ \Delta y = \Delta'_y + \Delta_{y,sl} \] 
\hspace{10cm} (3.7)

\[ \Delta p = \Delta'_p + \Delta_{p,sl} \] 
\hspace{10cm} (3.8)

\[ \Delta_{y,sl} = S R_{E} \phi_{y,A} L_{sl} \left( L - 0.5L_{sl} \right) \] 
\hspace{10cm} (3.9)

\[ \Delta_{p,sl} = S R_{I} \phi_{p,A} L_{sl} \left( L - 0.5L_{sl} \right) \] 
\hspace{10cm} (3.10)

\[ \Delta'_y = \frac{1}{3} L'^2 \phi_{y,A} \] 
\hspace{10cm} (3.11)

\[ \Delta'_p = \phi_{p,A} L_p \left( L' - 0.5L_p \right) \] 
\hspace{10cm} (3.12)

### 3.3.1.3 Column Construction and Materials

Construction of phase two columns was similar to that for the precast columns in phase one testing. Construction of both columns started by building the footing and column formwork separately. The footing reinforcing steel was laid out in the forms. Then the
dowels cages were built and placed onto the footing reinforcement cage. The reinforcing cages for the column shafts were built on a spacial wooden base which was fabricated for that purpose. Both column shafts and footings were cast at the same time on July 24, 2016. The concrete had a specified 28-day compressive strength of 4 ksi, and a maximum 0.375 in course aggregate. Concrete cylindrical samples were fabricated and concrete slump was measured. The measured slump was 6 in. Concrete properties for phase two columns are listed in Table 3.7. Three concrete cylinders were tested for the 28-day compressive strength. Also, three concrete cylinders were tested for the test day of each column. The cylinders were tested under compression load following ASTM C39 [9]. Figure 3.25 shows the completed footings and column shafts. Furthermore, Table 3.8 lists the tensile properties of the steel bars used in phase 2. The results for each bar size are the average of three samples and tested following ASTM E8 [10] and ASTM A370 [8].

Connection installation of the precast column shafts and the footing was performed at the Large-scale Structures Laboratory at the University of Central Florida. The procedure was exactly as was done for the columns in phase 1. Figure 3.26 and Figure 3.27 show some of the installation steps of the precast columns. The testing day compressive strength of the filler grout was 14.5 ksi and 13.5 ksi for G-40-2 and G-40-3 columns, respectively, and it was obtained according to ASTM C39 [9] and ASTM C109 [6].

<table>
<thead>
<tr>
<th>Column ID</th>
<th>Concrete Strength (psi)</th>
<th>Test Day</th>
</tr>
</thead>
<tbody>
<tr>
<td>G-40-2</td>
<td>4270</td>
<td>4600</td>
</tr>
<tr>
<td>G-40-3</td>
<td>4890</td>
<td>4890</td>
</tr>
</tbody>
</table>

Note: All results are reported in psi, and reflect the average of three samples.
Table 3.8: Measured tensile properties of the steel bars in phase 2

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Grade (ksi)</th>
<th>$f_y$ (ksi)</th>
<th>$\epsilon_y$</th>
<th>$E_s$ (ksi)</th>
<th>$f_u$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#5</td>
<td>60</td>
<td>65.6</td>
<td>0.00234</td>
<td>27450</td>
<td>105.9</td>
</tr>
<tr>
<td>#6</td>
<td>60</td>
<td>64.8</td>
<td>0.002435</td>
<td>28830</td>
<td>106.7</td>
</tr>
<tr>
<td>#6H</td>
<td>100</td>
<td>126.1*</td>
<td>0.006387*</td>
<td>28340</td>
<td>161.9</td>
</tr>
<tr>
<td>#9H</td>
<td>100</td>
<td>128.3*</td>
<td>0.00625*</td>
<td>28430</td>
<td>165.3</td>
</tr>
<tr>
<td>W4.5</td>
<td>-</td>
<td>98.46*</td>
<td>0.00432*</td>
<td>30320</td>
<td>104.8</td>
</tr>
</tbody>
</table>

* denotes that the yield properties are determined using 0.2% offset method

“H” denotes high-strength steel
Figure 3.26: Lowering the column onto the footing
3.3.1.4 Instrumentation and Testing

Instrumentation of column models in phase two was similar to that in phase one expect for little details with regards to how many sensors and their locations (Figure 3.29). A general instrumentation schematic and is shown in Figure 3.28. Test setup and data acquisition was similar to that in phase one.
Figure 3.28: Instrumentation schematics
Figure 3.29: Column instrumentation
CHAPTER 4: COLUMN EXPERIMENTAL RESULTS

4.1 Introduction

This chapter presents the test results from large-scale column model tests. Results from phase one testing are presented first followed by results from phase two tests. Some of the results discussed included: measured lateral force-displacement hysteresis curves, the average backbone and idealized elasto-plastic pushover curves, damage progression, and plastic hinge deformation and strain profiles. At the end of the chapter, results discussion is presented to summarize the key findings.

Test results presentation starts with the measured lateral force-displacement hysteresis curves, the average backbone and idealized elasto-plastic pushover curves, damage progression, and plastic hinge deformation and strain profiles.

4.2 Phase One Column Tests

Four Columns were tested in phase one. Two columns had AR = 4.0: one CIP and one precast. The other two columns had AR = 2.5: one CIP and one precast. For each AR, CIP and precast column are presented together to compare them at every aspect in their behavior.
4.2.1 Columns with AR = 4.0

4.2.1.1 Force-Displacement Behavior

The hysteresis loops for columns with AR = 4.0 are shown in Figure 4.1. Important events are also plotted in the Figure along with the elasto-plastic curves in the push and pull directions. Both columns exhibited stable hysteresis behavior. The hysteresis behavior for G-40-1 showed less lateral load capacity compared to C-40-1, which was due to a slight construction error. G-40-1 was designed to have 1.75 in concrete cover above the sleeve location, but was constructed with 2.25 in of concrete cover. The change in cover led to less confined concrete area, shorter distance between the neutral axis and the tension steel, and slightly lower section capacity.

The average pushover curves for columns with AR = 4.0 are shown in Figure 4.2. The pushover curve was obtained by averaging the peak values from the hysteresis loops for both push and pull cycles. The elasto-plastic (EP) curve was calculated such that the area under the EP and pushover curves had equivalent areas. The elastic portion of EP curve was determined by passing it through the “first yield point” on the pushover curve. First yield point was determined from strain gauge measurements from the steel bars located at the extreme faces of the push and pull directions of the column. The first yield point is determined when strain readings reached the yield strain of the bar; it was determined for push and pull cycles and then both readings were averaged. The ultimate point of the pushover curve was the drift point where the lateral load dropped by 20% in the 1st cycle. If no load drop occurred in the 1st cycle, then the ultimate point was taken as the maximum achieved drift in that direction. This procedure was applied in pull and push directions.
The EP curve was used to determine displacement ductility. The target displacement ductility was 7.0. Both columns achieved approximately the target ductility. G-40-1 column
showed reduction of 8% in average displacement ductility, $\mu_{avg}$, compared with C-40-1 column. The measured and calculated plastic load, $V_p$, for C-40-1 column was 45 kip and 44.7 kip, respectively. For G-40-1 column, the measured and calculated plastic load, $V_p$, was 37 kip and 41.75 kip, respectively.

4.2.1.2 Energy Dissipation Capacity

To make sure that this difference in lateral capacities is not related to performance behavior of the precast column, the energy dissipation of both CIP and precast columns was computed. The energy dissipation for the columns presented in terms of the equivalent viscous damping ratio $\zeta_{eq}$. The damping ratio normalizes the energy dissipation with respect to the effective stiffness, allowing an effective comparison between specimens without the effect of having different lateral capacities. It can be calculated using Equation 4.1.

$$\zeta_{eq} = \frac{E_D}{2\pi K_{eff} d_{max}^2}$$  \hspace{1cm} (4.1)

$$K_{eff} = \frac{F_{max} - F_{min}}{d_{max} - d_{min}}$$  \hspace{1cm} (4.2)

where $\zeta_{eq}$ is the equivalent viscous damping ratio, $E_D$ is the energy dissipation per cycle, $K_{eff}$ is the effective stiffness, $d_{max}$ and $d_{min}$ are the maximum positive and negative displacements of the hysteresis loop, respectively, and $F_{max}$ and $F_{min}$ are the associated force at the maximum positive and negative displacements, respectively.

Figure 4.3 shows the equivalent viscous damping ratio for columns C-40-1 and G-40-1 for the 1st and 2nd cycles up to 6% drift level where reliable damping ratio exhibited. It is
obvious that the damping ratio for both columns is very low up to 1% drift level. After that it begins to increase due to yielding of reinforcing bars. It can be seen that after yielding, G-40-1 column dissipated more energy than C-40-1 for in both 1st and 2nd cycles which indicates that higher strain was developed in the longitudinal bars of that column.

![Figure 4.3: Equivalent viscous damping ratio for columns with AR = 4.0](image)

### 4.2.1.3 Damage Progression

Figure 4.4 through Figure 4.10 depicts and compares the damage progression for columns with AR = 4.0 at 1, 2, 3, 4, 5, 6, and 8% drift levels. Before 1% drift, hairline flexural cracks developed for both columns. At 1% drift (Figure 4.4), more hairline flexural cracks appeared in both columns and some previous cracks became wider. Also, shear cracks began to appear and extend at the end 2% drift (Figure 4.5).

It was clear that shear and flexural cracks were mostly above the coupler region for G-40-1
column. By the end of 3% drift (Figure 4.6), minor spalling occurred in both columns at
the column-footing interface.

At 4% drift (Figure 4.7), flexural cracks localized for both columns at different locations
along the column shaft. For C-40-1, wide flexural cracks localized near the column-footing
interface. For G-40-1 the localization of flexural cracks occurred above the coupler region.
The difference in crack localization locations is to be expected, and can be attributed to
several factors. For C-40-1, cracks localization was caused by longitudinal bar strain pen-
etration into the footing and high tensile strain in the column longitudinal bars near the
column base. For G-40-1, cracks localization was due to the fact that the column was de-
tailed such that the critical section exists above the sleeve. Also, strain penetration of the
column bars into the grouted couplers contributes to the wide flexural crack at the top sleeve.
Furthermore, during construction of the precast columns a rubber cap is used on the top
end of the sleeve preventing column concrete from entering the sleeve during casting. This
cap debonds the longitudinal bars over a very short length (approximately 0.5 in) resulting
in larger concentrated rotations for a given applied load, which increases the crack width.

By 5% (Figure 4.8), concrete spalling occurred within 8 in from the footing surface for C-40-1
for both push and pull sides. For G-40-1, minor spalling occurred within the the coupler
region. At 6% drift (Figure 4.9), the columns exhibited more spalling. Both columns had
spalled concrete within the first 12 in from the footing surface. In addition, G-40-1 column
had spalled concrete within the first 4 in above the coupler region.

By 8% drift (Figure 4.10), significant difference in spalling height was observed between the
two columns. C-40-1 did not have a noticeable change in spalled concrete compared with
previous drifts, while G-40-1 had spalled concrete over the first 26 in above footing surface.
This difference is not unexpected given that G-40-1 was designed to develop damage above
the sleeve location. Although concrete spalled within the coupler region, the damage did not penetrate into the core concrete. Another noticeable difference between the two columns was the amount of cracking in the footing. C-40-1 developed cracks in the footing that became significant at higher drift levels as a result of strain penetration, while G-40-1 did not exhibit footing cracks until failure.

At failure, both columns exhibited bar buckling and fracture of transverse reinforcement as result of bar buckling. Unlike C-40-1, G-40-1 had longitudinal bar fracture occur during the first pull to -8%. As expected, the damage that occurred in C-40-1 was concentrated near the column-footing interface, whereas damage in G-40-1 was concentrated above the coupler region.

![Damage at 1% drift - 2nd cycle (North side)](image)

Figure 4.4: Damage at 1% drift - 2nd cycle (North side)
Figure 4.5: Damage at 2% drift - 2nd cycle (North side)

(a) C-40-1 Column
(b) G-40-1 Column

Figure 4.6: Damage at 3% drift- 2nd cycle (North side)

(a) C-40-1 Column
(b) G-40-1 Column
Figure 4.7: Damage at 4% drift- 1st cycle (North side)

Figure 4.8: Damage at 5% drift- 2nd cycle (North side)
Figure 4.9: Damage at 6% drift- 2nd cycle (North side)

Figure 4.10: Damage at 8% drift- 1st cycle (North side)
4.2.1.4 Plastic Hinge Deformations

Total displacement of the column tip is the sum of: flexural, shear and strain penetration deformations. These deformation components were determined using the instrumentation configuration depicted in Figure 4.11. Figure 4.12 depicts the individual components of the column tip displacement for columns with AR = 4.0 for the 1st push cycles only which is considered to be representative of other cycles. The results are shown up to 4% drift where reliable data are considered. The maximum instrumentation error was approximately 12%, which occurred in G-40-1.

For C-40-1 model, strain penetration in the footing contributes the most (40-50%) to the overall displacement observation of column models. this level of displacement due to bond slip is not uncommon in conventional reinforced concrete columns Lehman [40]. This was followed by the flexural (35-45%) and shear components (5-10%) of deformation. Flexural deformations were spread relatively uniform among gauge locations L1-L4. Thus, the hinge behavior was typically of a well-detailed CIP column.

Figure 4.11: Schematics of curvature, bond slip and shear instrumentations - Phase One
On the other hand, plastic hinging behavior of G-40-1 model was different. Strain penetration in the footing still had a significant contribution (24-30%) to the overall column displacement but less than that observed in C-40-1. This was because the high-strength bars did not exhibit nonlinear behavior, as expected. Another significant contribution can be seen from the flexural component at level L2 which contained the critical section of the column at the top of the sleeves. That was due the reasons mentioned in the previous section: existence of the critical section above the sleeve, strain penetration of column steel bars into the grouted sleeves and the increased deformation due to longitudinal bar debonding by the rubber cap.
at the top of the sleeve. This also explains why flexural deformation in level L2 for G-40-1 model is bigger than that in C-40-1 model although the level location is the same for both models. Flexural deformation contributions decrease above level L2. Flexural deformation at the mid-height of the sleeve (L1) exists but small (5-9%). Flexural behavior above the sleeve shows a well-distributed flexural behavior that indicates a successful plastic hinge shifting. Shear deformation was approximately the same for both columns, implying that the effective aspect ratio had negligible effect on shear intensity for flexural-dominant columns.

4.2.1.5 Plastic Hinge Strains

Tensile strains measured from longitudinal reinforcing bars can provide insight into the spread of plasticity along a column shaft. Strain profiles for C-40-1 and G-40-1 models are shown in Figure 4.13 for the 1st push cycles up to a reliable drift level (6% drift). The strain distribution was measured at six and seven levels throughout the footing and column for C-40-1 and G-40-1 models, respectively. The plots identifies the footing surface location, the yield strain of reinforcing bars, and the sleeve region. As expected, the C-40-1 column displayed well-distributed plasticity along the bottom length of the column shafts and spread of plasticity into the footings. For G-40-1 column, yielding of longitudinal bars developed in the target critical section above sleeve region, propagating up the column shaft with increasing drift levels. This indicates that SPH was adequately achieved in the precast column. Also, the longitudinal strains in HS footing dowels were below yield even at high drift levels, consistent with the design objective, while the strain near the column-footing interface for C-40-1 model exceeded the yield strain at 1% drift and became a lot higher at increasing drift levels which was a sign of significant strain penetration (bar slip) into the footing.
The plots also shows the length over which plasticity occurred within the column shaft, $l_{plastic}$. It is defined as the distance between the top of the sleeve region (or footing surface in the case of CIP columns) and the intersection of the strain profile (or its extension) with the yielding line. The plasticity length was a little smaller for G-40-1 compared with C-40-1 column.

Figure 4.13: Measured longitudinal reinforcing bar strain profiles of columns with AR = 4.0

4.2.2 Columns with AR = 2.5

4.2.2.1 Force-Displacement Behavior

The hysteresis loops for columns with AR = 2.5 are shown in Figure 4.14. Important events such as onset of yielding, concrete spalling, spiral fracture, and longitudinal bar fracture are also plotted in the Figure along with the elasto-plastic curves in the push and pull directions.
Both models achieved approximately similar lateral load capacity. The hysteresis behavior for C-25-1 exhibited more lateral load capacity in the pull direction (south direction) due to extra concrete cover in the compression zone. During construction, the reinforcing steel cage was unintentionally shifted 1 in towards the North side of the column. The precast column model, G-25-1, exhibited less ductile behavior than the control model (C-25-1). This can be attributed to the precast model undergoing more shear deformation than the control model. The stiffened coupler region in G-25-1 caused the column to have a shorter effective aspect ratio that resulted in higher shear intensity and deformation compared with C-25-1. It can be observed from the hysteresis loops of G-25-1 that there is noticeable degradation at high drift levels compared to C-25-1 model. This was caused by shear degradation of concrete from widening of the inclined shear cracks and reduced aggregate interlock along the cracks.

The average pushover curves for columns with AR = 2.5 are shown below in Figure 4.15 and they were determined using the same procedure followed in the previous columns with...
AR = 4.0. G-25-1 column showed a reduction of 19% in average displacement ductility, \( \mu_{avg} \), compared with C-25-1 column. The measured and calculated plastic load, \( V_p \), for C-25-1 column was 60.6 kip and 58.8 kip, respectively. For G-25-1 column, the measured and calculated plastic load, \( V_p \), was 58.4 kip and 58.2 kip, respectively.

![Average force-displacement backbone curve for columns with AR = 2.5](image)

**Figure 4.15:** Average force-displacement backbone curve for columns with AR = 2.5

### 4.2.2.2 Energy Dissipation Capacity

Figure 4.16 shows the equivalent viscous damping ratio for columns C-25-1 and G-25-1 for the 1st and 2nd cycles up to 5% drift level. It can be noticed that the damping ratio for both columns is very low up to 1% drift level (prior to steel yielding). After that damping ratio started to increase due to yielding of reinforcing bars. It was observed that before yielding, G-25-1 column dissipated more energy than C-25-1 in both 1st and 2nd cycles, while it dissipated less dissipated energy than C-25-1 in both 1st and 2nd cycles. The reason behind that behavior was not known. Generally, both columns had comparable energy dissipation.
4.2.2.3 Damage Progression

Figure 4.17 through Figure 4.22 shows the damage progression for columns with AR = 2.5 at 1, 2, 3, 4, 5, 6% drift. Before 1% drift, few hairline flexural cracks initiated. At 1% drift, both columns had wider flexural cracks (Figure 4.17). By the end of 2% (Figure 4.18), both columns developed a lot of shear cracks. Also, flexural cracks became wider at this drift level. At 3% drift (Figure 4.19), both shear and flexural cracks increased in width for both columns and minor concrete spalling occurred near the column-footing interface. At 4% drift (Figure 4.20), significant spalling can be observed for both columns. The spalled concrete height for C-25-1 was 9 in above the footing surface and 14 in for G-25-1. The increased height of spalling in G-25-1 is not unexpected. Shear cracks were wider in G-25-1 than in C-25-1 (refer to Figure 4.21) due to the presence of the coupler region that stiffened the base.
of the column and shortened the effective cantilever length.

Figure 4.17: Damage at 1% drift- 2nd cycle (North side)

By the end of 5% drift (Figure 4.21), C-25-1 did not exhibit significant changes in apparent damage, while G-25-1 showed substantial changes in damage progression. Damage penetrated the concrete core above the coupler zone. Bar buckling occurred on the north side of the column causing the couplers to push away from the core concrete. Spalling of cover concrete can be observed in the south side of G-25-1 model up to 24 in above the footing surface. At 6% drift (Figure 4.22), shear cracks of C-25-1 column became wider, and G-25-1 exhibited more degradation where bar buckling and core damage are more severe at this drift level. At failure, both columns exhibited bar buckling and fracture of transverse reinforcement as result of bar buckling. Unlike C-25-1, G-25-1 had two longitudinal bars fractured during the first push to 6% and one longitudinal bar fracture during the first pull to 6%.

Strain penetration developed in the footing of C-25-1, which caused delamination of concrete at the surface of the footing. This was not noticeable in G-25-1 due to presence of high-
strength bars with enlarged sizes that led to elastic behavior at the section below the sleeves.

Figure 4.18: Damage at 2% drift- 2nd cycle (North side)

Figure 4.19: Damage at 3% drift- 2nd cycle (North side)
Figure 4.20: Damage at 4% drift- 2nd cycle (North side)

(a) C-25-1 Column

(b) G-25-1 Column

Figure 4.21: Damage at 5% drift- 2nd cycle (East side)

(a) C-25-1 Column

(b) G-25-1 Column
4.2.2.4 Plastic Hinge Deformations

Figure 4.23 shows components of deformation for the 1st push cycles of columns with AR=2.5 up to 4% drift level. These components were determined in a similar method as in the columns with AR=4.0 using the instrumentation configuration depicted in Figure 4.11. For C-25-1 model, similar to the C-40-1 previously presented, the strain penetration in the footing contributes the most to the overall displacement of the column model (40-53%). This was followed by a total flexural (30-40%) and shear (9-13%) contributions to the overall column deformation. Flexural deformations were concentrated within gage lengths L1 and L2.
Contributions of displacement components for the corresponding precast column, G-25-1, were approximately similar to those in G-40-1 model, but with larger contributions from shear deformation. Strain penetration in the footing was significant but less than that in C-25-1 due to the elastic behavior of HS footing dowels. The flexural component at level L2 (top of the sleeves) was another significant contribution to the total displacement for the same reasons explained in G-40-1 model. Like G-40-1 column, flexural deformation at the mid-height of the sleeve (L1) was small. However, the shear contribution appeared to be more than all other models, as was expected since the effective aspect ratio \( (L_{eff}/D) \) was
reduced from 2.5 to 1.94. The inclined shear cracks for this model were wider and caused gradual reduction in aggregate interlock with increasing drift ratios. The lower effective AR combined with low transverse reinforcing ratio led to more shear deformation. That caused the shear strength to drop at high drifts and thus affecting the lateral load capacity of the column (refer to the hysteresis loops of G-25-1 model). The plastic hinging for G-25-1 started to develop above the sleeve but shear degradation disrupted a well-defined plastic hinge.

4.2.2.5 Plastic Hinge Strains

Strain profiles for C-25-1 and G-25-1 models are shown in Figure 4.24 up to 5% drift level for the 1st push cycles. The plots show the footing surface location, the yield strain of reinforcing bars, and the sleeve region. The strain distribution was measured at six levels throughout the footing and column length. As expected, the C-40-1 column displayed well-distributed plasticity along the bottom length of the column shafts. Also, plasticity spread in the footing was excessive which indicated strain penetration effect in the footing. On the other hand, G-25-1 model had limited strain propagation in the footing as expected due to the presence of HS dowels. Yielding of longitudinal bars of G-25-1 was concentrated above the couplers and propagated more into the column height with increasing drift levels. There was 27% reduction in spread of plasticity for G-25-1 column compared with the corresponding C-25-1 column as shown from the $l_{plastic}$ measurement. That was due to the higher shear degradation which caused damage within and above sleeve region at advanced drift levels which decreased the yielding propagation.
Figure 4.24: Measured longitudinal reinforcing bar strain profiles of columns with AR=2.5

4.3 Phase Two Column Tests

4.3.1 Column G-40-2

4.3.1.1 Force-Displacement Behavior

The hysteresis loops for G-40-2 column is shown in Figure 4.25 along with important events and elasto-plastic curves in the push/pull directions. The column exhibited stable hysteresis behavior. The column exhibited strength degradation after the first fracture of transverse reinforcement, which occurred during the push to 8% drift. The progression of failure for the column included fracture of transverse reinforcement, longitudinal bar buckling, and subsequent longitudinal bar fracture.
The average pushover curves for G-40-2 are shown in Figure 4.26. The measured displacement ductility is depicted in the plot, which was determined to be 6.3. The target design displacement ductility was 6.8. Thus, there is relatively good agreement between the expected and achieved ductility. Furthermore, the plastic lateral loads, $V_p$, determined from the experiment correlated well with calculated values; the plastic lateral load is the flat region of the elastic-plastic curve. The measured and calculated plastic lateral loads were 44.1 kip and 44.4 kip, respectively.
4.3.1.2 Energy Dissipation Capacity

Figure 4.27 shows the equivalent viscous damping ratio for column G-40-2 for the 1st and 2nd cycles up to 6% drift level. It can be noticed that the damping ratio for the column is very low up to 1% drift level (before steel yielding). After steel yielding, damping ratio began to increase due to longitudinal bar yielding. The trend and values of damping ratio are comparable to those observed in previous columns.
4.3.1.3 Damage Progression

Figures 4.28 through 4.31 shows the damage progression for G-40-2 column at 1, 2, 3, 4, 5, 6, and 8% drift ratios, respectively. All photos presented represent the north side of the column, which is the compression face under push cycles, and the tension face under pull cycles. Prior to 1%, few thin flexural cracks started to appear. At 1% drift (Figure 4.28a), more thin flexural cracks started to appear. At 1% drift (Figure 4.28a), more thin flexural cracks occurred.

At 2% drift (Figure 4.28b), flexural cracks became wider, and thin shear cracks began to occur. By the end of 3% (Figure 4.29a), significant spalling was observed within the top portion of the coupler region. At 4% drift (Figure 4.29b), spalling increased. The spalled concrete height extended from the top third portion of the sleeve region to 5 in above the sleeve region for. At 5% (Figure 4.30a), concrete spalling increased significantly and extended...
to 12 in above the coupler region. By the end of 6% drift (Figure 4.30b), spalling increased, and more transverse and longitudinal reinforcing bars were exposed. At 8% (Figure 4.31), damage penetrated the concrete core more. At failure, the column exhibited bar buckling and fracture of transverse reinforcement as result of bar buckling. Subsequently, the column had four longitudinal bars fracture during 8% drift. No sign of strain penetration into the footing was observed until failure.

![Damage progression of G-40-2 at 1% and 2% drift](image)

Figure 4.28: Damage progression of G-40-2 at 1% and 2% drift
Figure 4.29: Damage progression of G-40-2 at 3% and 4% drift

Figure 4.30: Damage progression of G-40-2 at 5% and 6% drift
4.3.1.4 Plastic Hinge Deformations

Figure 4.32 depicts the displacement contributions for column G-40-2 for the 1st push cycles up to 6% drift level. The deformation components were determined using the instrumentation configuration depicted in Figure 4.33. It was observed that bond slip contribution to the
total column displacement was comparable to the previous precast columns. Flexural deformation from the coupler region (L1) contributed about (10-15%) to the total deformation. Flexural components from gage (L2) contributed the most to the column tip displacement, which was expected since this region contained the critical section. Regions above L2 shows a well-defined shifted hinge above the sleeve. Shear deformation contributed about (10-13%) to the total deformation which was comparable to previous columns with AR = 4.0.

Figure 4.32: Deformation components for G-40-2
4.3.1.5 Plastic Hinge Strains

Strain profiles for G-40-2 model are shown in Figure 4.34 for the 1st pull cycles up to 6% drift level. The strain distribution was measured at seven levels throughout the footing and the column shaft. Footing surface, measured yield strain, sleeve region, and measured spread of plasticity are identified in the plot. It can be noticed that the strain on the footing of G-40-2 model was significantly below the yielding strain due to the presence of HS footing dowels with $i_T = 3.0$ as which satisfied the design objectives. Also, it can be seen that yielding of longitudinal bars developed directly above the coupler region (the target critical section) and propagated up the column shaft with increasing drift levels. The measured spread of plasticity, $l_{plastic}$, was 20.7 in which is comparable to the previous columns with AR= 4.0.
Figure 4.34: Measured longitudinal reinforcing bar strain profiles of column G-40-2

4.3.2 Column G-40-3

4.3.2.1 Force-Displacement Behavior

Figure 4.35 shows the hysteresis loops and the pushover curves in push/pull directions for G-40-3 column. Also, several important events are identified in the plot. The column exhibited stable hysteresis behavior. The column did not exhibit strength degradation until the first fracture of transverse reinforcement, which occurred during the push to 10% drift. After that failure progressed to include longitudinal bar buckling and subsequent longitudinal bar fracture.
Average pushover curves for G-40-3 are shown in Figure 4.36. Also the plot shows the average measured displacement ductility, $\mu_{avg}$, which was determined to be 6.6. The target design displacement ductility was 6.9. There is a good agreement between the expected and achieved ductility. Furthermore, the achieved plastic lateral loads, $V_p$, correlated well with calculated value. The measured and calculated plastic lateral loads were 28.1 kip and 28.9 kip, respectively, which shows excellent agreement.
4.3.2.2 Energy Dissipation Capacity

Figure 4.37 shows the equivalent viscous damping ratio for G-40-3 column for the 1st and 2nd cycles up to 8% drift level. Prior to yielding, the damping ratio is very low up to 1% drift level. After longitudinal steel yielding, damping ratio began to increase due to yielding. Behavior of G-40-3 column in dissipating energy is similar to the previously presented columns.
4.3.2.3 Damage Progression

Figure 4.38 through Figure 4.41 shows the damage progression for the G-40-3 column at 1, 2, 3, 4, 5, 6, 8, and 10% drift ratios, respectively. All photos presented represent the north side of the column. Prior to 1% drift, the column developed very few thin flexural cracks above the coupler region. At 1% (Figure 4.38a), the column developed more flexural cracks.

At 2% drift (Figure 4.38b), thin shear cracks were observed. At 3% (Figure 4.39a), concrete spalling was observed within the coupler region. At 4% drift (Figure 4.39b), significant spalling can be observed. The spalled concrete height extended from the top half portion of the sleeve region to 6 in above the coupler region. At 5% drift (Figure 4.40a), spalling increased and more transverse reinforcements were exposed.
By the end of 6% drift (Figure 4.40b), spalling increased, and more transverse and longitudinal reinforcing bars were exposed. At 8% (Figure 4.41a), no noticeable change was observed in column damage. By 10% (Figure 4.41b), very significant spalling can be observed (15 in above the coupler region) and more longitudinal bars were exposed. At failure, the column exhibited bar buckling and fracture of transverse reinforcement as result of bar buckling. The column had three longitudinal bars fracture at 10% drift.

(a) 1% Drift- 2nd Pull Cycle
(b) 2% Drift- 2nd Pull Cycle

Figure 4.38: Damage progression of G-40-3 at 1% and 2% drift
Figure 4.39: Damage progression of G-40-3 at 3% and 4% drift

Figure 4.40: Damage progression of G-40-3 at 5% and 6% drift
4.3.2.4 Plastic Hinge Deformations

Figure 4.42 depicts the displacement contributions for column G-40-3 for the first push cycles up to 8% drift. The deformation components were determined using the instrumentation configuration depicted in Figure 4.43. It can be observed that the bond slip contribution to the total column deformation was comparable to previous precast columns. Coupler region contribution (L1) was small as observed in previous precast columns. Flexural contributions in the target critical section (L2) and above indicated a well-shifted hinging above the sleeve. Shear deformation was around 10% during the test up to failure.
Figure 4.42: Deformation components for G-40-3
Figure 4.43: Schematics of curvature, bond slip and shear instrumentations for G-40-3 column

4.3.2.5 Plastic Hinge Strains

Measured strain profiles of longitudinal reinforcing bars for G-40-3 model are shown in Figure 4.44 for the 1st pull cycles up to 6% drift level. The strain distribution was measured at seven levels throughout the footing and the column shaft. Locations of the footing, sleeve, and the measured yield strain of the bar are plotted as well. As observed in previous precast models, the strain in the HS dowel bars of G-40-3 model was below the yielding strain which was expected. Also, steel bars above sleeve region developed yielding at the critical section and then yielding propagated up the column with increasing drift levels which indicated that SPH was developed as expected. The plot also shows that spread of plasticity, \( l_{\text{plastic}} \), is comparable to other precast columns with AR = 4.0.
4.4 Comparison with Previous Tests

Six large scale column models were constructed and tested and evaluated. Of them, four columns were precast columns utilizing GS connections and were designed using SPH methodology. Test results showed clearly that SPH worked as expected for the columns in the study. Use of SPH with grouted sleeve connections improved the displacement ductility significantly compared with previous studies [72, 28, 53]. It also minimized the damage in the capacity protected element (footing). Test results suggest that the energy dissipation for such connections is comparable to CIP connections. Strain distributions along the column height showed that shifted hinging occurred above the coupler region as expected. Visual damage progression supported that conclusion. Results also suggest that the deformation within the
coupler region should be included in the design procedure, otherwise the ductility will be underestimated. Furthermore, test observations suggest that the debonded length within the rubber cap region needs to be accounted for, because it increases the rotational capacity of the column.

Results from previous studies on the use of GS in seismic regions are summarized in Table 4.1. All previous precast column models were designed to emulate their CIP corresponding models (emulative design). However, in the current study, the precast column models were designed to achieve the same lateral load capacity of the reference CIP models. The precast models were not completely emulative to the CIP models. The first column in Table 4.1 shows the studies which were conducted on GS connections including the current study. The second column in the Table depicts the column identification that was used in the corresponding study. The Table also shows the details about the column models related to aspect ratio, reinforcement ratios and model scale in columns 3 though 6, respectively. Type of each column model and, location of grouted coupler in the precast column and connection type, either C-F (column-footing connection) or C-CB (column-bent cap connection) are presented in column 7 and 8 in the Table. Transition index, $i_T$, is also shown in column 9 in the Table. The index is zero for the previous studies because the precast columns were emulative to their CIP reference columns. Two transition indices were used in the current study: 1 and 3. Displacement ductility is shown in column 10 in the Table. The last column summarizes the reduction in ductility compared with the CIP equivalent model.
Table 4.1: Comparison of seismic performance of column models with grouted sleeve couplers

<table>
<thead>
<tr>
<th>Reference</th>
<th>Column ID</th>
<th>AR</th>
<th>Long. Trans.</th>
<th>Scale</th>
<th>Column Description</th>
<th>Conn. Type</th>
<th>$\tau$</th>
<th>Ductility</th>
<th>Reduct.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Haber et al. (2013)</td>
<td>CIP</td>
<td>4.5</td>
<td>1.92% 1.05%</td>
<td>0.5</td>
<td>reference model</td>
<td>C-F</td>
<td>-</td>
<td>7.36</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>GCNP</td>
<td>4.5</td>
<td>1.92% 1.05%</td>
<td>0.5</td>
<td>precast model</td>
<td>C-F</td>
<td>0</td>
<td>4.52</td>
<td>39%</td>
</tr>
<tr>
<td></td>
<td>GCNP</td>
<td>4.5</td>
<td>1.92% 1.05%</td>
<td>0.5</td>
<td>precast model, pedestal on top of footing</td>
<td>C-F</td>
<td>0</td>
<td>4.53</td>
<td>39%</td>
</tr>
<tr>
<td>Tazarv et al. (2014)</td>
<td>CIP</td>
<td>4.5</td>
<td>1.92% 1.05%</td>
<td>0.5</td>
<td>reference model</td>
<td>C-F</td>
<td>-</td>
<td>7.36</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>GCNP</td>
<td>4.5</td>
<td>1.92% 1.05%</td>
<td>0.5</td>
<td>precast model, pedestal on top of footing, bars debonded through pedestal</td>
<td>C-F</td>
<td>0</td>
<td>7.07</td>
<td>4%</td>
</tr>
<tr>
<td>Pantelides et al. (2014)</td>
<td>GGSS-CIP</td>
<td>4.6</td>
<td>1.3% 1.9%</td>
<td>0.5</td>
<td>reference model to GGSS group models</td>
<td>C-F</td>
<td>-</td>
<td>8.9</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>GGSS-1</td>
<td>4.6</td>
<td>1.3% 1.9%</td>
<td>0.5</td>
<td>precast model, coupler at the column end</td>
<td>C-F</td>
<td>0</td>
<td>5.4</td>
<td>39%</td>
</tr>
<tr>
<td></td>
<td>GGSS-2</td>
<td>4.6</td>
<td>1.3% 1.9%</td>
<td>0.5</td>
<td>precast model, coupler at the footing</td>
<td>C-F</td>
<td>0</td>
<td>6.1</td>
<td>32%</td>
</tr>
<tr>
<td></td>
<td>GGSS-3</td>
<td>4.6</td>
<td>1.3% 1.9%</td>
<td>0.5</td>
<td>precast model, coupler at the column end, bars debonded in the footing</td>
<td>C-F</td>
<td>0</td>
<td>6.8</td>
<td>24%</td>
</tr>
<tr>
<td></td>
<td>FGSS-CIP</td>
<td>4.6</td>
<td>1.3% 1.9%</td>
<td>0.5</td>
<td>reference model to FGSS group models</td>
<td>C-CB</td>
<td>-</td>
<td>9.9</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>FGSS-1</td>
<td>4.6</td>
<td>1.3% 1.9%</td>
<td>0.5</td>
<td>precast model, coupler at the column end</td>
<td>C-CB</td>
<td>0</td>
<td>4.9</td>
<td>50%</td>
</tr>
<tr>
<td></td>
<td>FGSS-2</td>
<td>4.6</td>
<td>1.3% 1.9%</td>
<td>0.5</td>
<td>precast model, coupler at the cap beam</td>
<td>C-CB</td>
<td>0</td>
<td>5.8</td>
<td>42%</td>
</tr>
<tr>
<td></td>
<td>FGSS-3</td>
<td>4.6</td>
<td>1.3% 1.9%</td>
<td>0.5</td>
<td>precast model, coupler at the column end, bars debonded in the cap beam</td>
<td>C-CB</td>
<td>0</td>
<td>3.1 (e)</td>
<td>69%</td>
</tr>
<tr>
<td>Current Study</td>
<td>C-40-1</td>
<td>4.0</td>
<td>1.95% 0.74%</td>
<td>0.42</td>
<td>reference model to G-40-1</td>
<td>C-F</td>
<td>-</td>
<td>7.8</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>G-40-1</td>
<td>4.0</td>
<td>1.43% 0.74%</td>
<td>0.42</td>
<td>precast model, coupler at the column end</td>
<td>C-F</td>
<td>1</td>
<td>7.15</td>
<td>9%</td>
</tr>
<tr>
<td></td>
<td>C-25-1</td>
<td>2.5</td>
<td>1.43% 0.50%</td>
<td>0.42</td>
<td>reference model to G-25-1</td>
<td>C-F</td>
<td>-</td>
<td>6.91</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>C-25-1</td>
<td>2.5</td>
<td>1.0% 0.50%</td>
<td>0.42</td>
<td>precast model, coupler at the column end</td>
<td>C-F</td>
<td>1</td>
<td>5.6</td>
<td>19%</td>
</tr>
<tr>
<td></td>
<td>G-40-2</td>
<td>4.0</td>
<td>1.43% 1.11%</td>
<td>0.42</td>
<td>precast model, coupler at the column end</td>
<td>C-F</td>
<td>3</td>
<td>6.3</td>
<td>10%</td>
</tr>
<tr>
<td></td>
<td>G-40-3</td>
<td>4.0</td>
<td>1.59% 1.13%</td>
<td>0.33</td>
<td>precast model, coupler at the column end</td>
<td>C-F</td>
<td>1</td>
<td>6.6</td>
<td>6%(f)</td>
</tr>
</tbody>
</table>

*a Connection types are C-F for column-footing and C-CB for column-bent cap

*b Reduction in ductility relative to the CIP reference model

*c Not the right ductility due to error in testing

*d Denotes reinforcement ratio within the column shaft

*e Ductility reduction is based on target design ductility of 6.8

*f Ductility reduction is based on target design ductility of 6.95
CHAPTER 5: TENSILE COMPONENT RESULTS

5.1 Introduction

Along with the large scale column test which were discussed in Chapter 4, axial tension tests were performed on grouted couplers splicing steel bars with varying size and grade. The goal was to characterize the tensile behavior of the spliced bars, determine the failure mode, quantify the strain penetration (slip) of the bars into the grout, and develop simple and accurate models for the stress-strain behavior and strain penetration of the grouted couplers. Such models will be effective in modeling precast columns with grouted couplers utilized in their connections.

5.2 Monotonic Tension Test

The tensile test was performed in displacement control mode following ASTM A1034, ASTM E8, and California Test CT670. A series of tensile tests was conducted in both phases one and two. All samples achieved a well defined yield stress and, yield plateau region, and strain hardening region.

5.2.1 Phase One

Three sets of specimens were tested in phase one; each had three samples. Figures 5.1 through 5.3 depict the measured stress-strain of the grouted coupler splices for representative samples of each set on different location on the sample. All results are summarized in Table 5.1. The solid blue line is the stress-strain within the coupler region. The strain was measured from
the coupler region extensometer, while the stress was determined by dividing the applied axial load by the area of the NS (Gr. 60) bar in the spliced specimen.

The red dashed line is the average stress-strain measured for the NS (Gr. 60) bar. The strain was obtained from averaging the two strain gauges instrumented on the NS (Gr. 60) bar. The stress was the same which was used in the coupler region.

The solid black line is the average stress-strain of the HS (Gr. 100) bar. The strain was measured from the two strain gauges instrumented on the HS (Gr. 100) bar. The stress was calculated by using the applied load and divide it by the bar area of the HS (Gr. 100) bar.

Figure 5.1: Stress-strain of 5N-6H-1 on different locations on the specimen
Figure 5.2: Stress-strain of 6N-7H-1 on different locations on the specimen

Figure 5.3: Stress-strain of 11N-14H-1 on different locations on the specimen
All three lines were truncated at the point where only reliable strain data was applicable. It was noticed that Gr. 60 bars behaved as expected. Coupler region behavior was similar to Gr. 60 bar behavior in the linear elastic region but stiffer after that in the nonlinear region. The stress-strain behavior of the Gr. 100 bars was linear during the test up to specimen failure.

Parameters $SR_E$ and $SR_I$ (listed in Table 5.1) are the ratios between strain within the coupler region and strain in the NS (Gr. 60) bar in the linear elastic zone and the inelastic zone, respectively. Figure 5.4 shows the strain ratio between coupler region the NS bar, $SR$, for the first specimen of each set. The plot is shown up to strain of 0.04 in/in where reliable data existed in that range. It can be noticed that $SR_E$ is approximately equal to 1.0 and $SR_I$ is approximately close to 0.5 for all sets of specimens. It was expected to obtain $SR_I$ less than 1.0 because of the stiffening effect which comes from the presence of the sleeve that add numerous confinement to the bars through a strong medium of grout.

The failure mode for all specimens was bar rupture away from the coupler region (refer to Figure 5.5 for 11N-14H specimens), which indicates that the coupler is able to fully develop the ultimate stress in the bars as opposed to other failure modes such as bar pullout from grout or coupler rupture which if occurred, the ultimate bar stress would not be developed. The failure mode indicated excellent performance of the grouted couplers.
Figure 5.4: Strain ratio of representative specimens from phase one
Table 5.1: Monotonic tensile test results for phase one

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Strain in NS (Gr. 60) bar</th>
<th>Strain within the Coupler Region</th>
<th>Strain Ratio</th>
<th>$f_y$ (ksi)</th>
<th>$f_u$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strain at yield Ultimate strain</td>
<td>Strain at yield Ultimate strain</td>
<td>$SR_E$</td>
<td>$SR_I$</td>
<td></td>
</tr>
<tr>
<td>5N-6H-1</td>
<td>0.002413 0.116</td>
<td>0.002432 0.0506</td>
<td>1</td>
<td>0.41</td>
<td>68</td>
</tr>
<tr>
<td>5N-6H-2</td>
<td>0.00243 0.113</td>
<td>0.00245 0.0552</td>
<td>1</td>
<td>0.48</td>
<td>74</td>
</tr>
<tr>
<td>5N-6H-3</td>
<td>0.00246 0.114</td>
<td>0.00242 0.0499</td>
<td>0.98</td>
<td>0.54</td>
<td>71.5</td>
</tr>
<tr>
<td>Average</td>
<td>0.00243 0.114</td>
<td>0.00243 0.0519</td>
<td>1</td>
<td>0.476</td>
<td>71.2</td>
</tr>
<tr>
<td>6N-7H-1</td>
<td>0.00252 0.115</td>
<td>0.00257 0.0528</td>
<td>1.02</td>
<td>0.476</td>
<td>68.9</td>
</tr>
<tr>
<td>6N-7H-2</td>
<td>0.00248 0.1185</td>
<td>0.00242 0.0527</td>
<td>0.98</td>
<td>0.473</td>
<td>66.5</td>
</tr>
<tr>
<td>6N-7H-3</td>
<td>0.00243 0.117</td>
<td>0.00237 0.0524</td>
<td>0.975</td>
<td>0.55</td>
<td>66.8</td>
</tr>
<tr>
<td>Average</td>
<td>0.00247 0.117</td>
<td>0.00245 0.0526</td>
<td>0.99</td>
<td>0.5</td>
<td>67.4</td>
</tr>
<tr>
<td>11N-14H-1</td>
<td>0.002173 0.122</td>
<td>0.0022 0.0395</td>
<td>1.01</td>
<td>0.38</td>
<td>66.2</td>
</tr>
<tr>
<td>11N-14H-2</td>
<td>0.00225 0.119</td>
<td>0.00217 0.0385</td>
<td>0.97</td>
<td>0.382</td>
<td>65.8</td>
</tr>
<tr>
<td>11N-14H-3</td>
<td>0.00218 0.12</td>
<td>0.00211 0.0406</td>
<td>0.97</td>
<td>0.367</td>
<td>66.5</td>
</tr>
<tr>
<td>Average</td>
<td>0.0022 0.12</td>
<td>0.00213 0.0395</td>
<td>0.98</td>
<td>0.376</td>
<td>66.2</td>
</tr>
</tbody>
</table>
5.2.2 Phase Two

In phase two tensile testing, eight sets of specimens were tested; each set had two specimens. The stress-strain behavior of the grouted couplers at different locations is shown in Figure 5.6 through Figure 5.8 for some specimens. All other details are listed in Table 5.2. The red dashed line represents the stress-strain behavior measured for the spliced NS (Gr. 60) bar. The average strain was obtained from the two strain gauges instrumented on the NS (Gr. 60) bar. The stress was obtained by dividing the applied axial load by the area of the NS (Gr. 60) bar in the spliced specimen. The NS bar behavior was as expected where there was linear region, yield plateau region, and strain hardening region.

The blue solid line shows the stress-strain behavior within the coupler region. The strain was measured from the coupler region extensometer, while the stress was the same stress which was obtained in NS (Gr. 60) bar. The coupler region behavior also exhibited linear, yield plateau, and strain hardening regions as in NS (Gr. 60) bar but in stiffer manner.

![Stress-strain graph](image)

Figure 5.6: Stress-strain of the bar and the coupler region for 5N-5N-1
The black solid line represents the stress-strain behavior of the spliced HS (Gr. 100) bar. It
can be seen that the behavior of HS (Gr. 100) bars was linear until specimen failure. All specimens failed by bar rupture away from the coupler region (Figure 5.9), which indicates that the grouted coupler is fully efficient in splicing steel bars.

![Image](image1.png)  
(a) 5N-5N-2  
(b) 6N-9H-1  
(c) 11N-14H-2

Figure 5.9: Failure mode of some samples in phase two

Figure 5.10 shows the the ratio between the strain within the coupler region and the strain in the NS (Gr. 60) bar for some selected specimens. The plot also shows dashed lines for $SR$ of 0.25, 0.5 and 1.0 that are used as reference. It can be observed that there is approximately linear relationship between the strain over the coupler region and the strain in the NS (Gr. 60) bar. Specimens with larger sleeve sizes and higher transition indices, $i_T$, tend to have lower $SR$ due to the stiffening effects from either the sleeve and/or the presence of HS bars. The strain ratio ($SR$) is a very important parameter that is used in the DBD method explained in Chapter 3 and detailed in the reference [29]. It is used to determine the curvature distribution in the sleeve region of the precast column. Then the column lateral displacement can be determined.
Figure 5.10: Strain ratio of representative specimens from phase two.
Table 5.2: Monotonic tensile test results for phase two

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Strain at yield</th>
<th>Ultimate Strain</th>
<th>Strain at yield</th>
<th>Ultimate Strain</th>
<th>Strain Ratio</th>
<th>$f_y$ (ksi)</th>
<th>$f_u$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5N-5N-1</td>
<td>0.00258</td>
<td>0.105</td>
<td>0.00213</td>
<td>0.0437</td>
<td>0.723</td>
<td>0.53</td>
<td>65.2</td>
</tr>
<tr>
<td>5N-5N-2</td>
<td>0.00251</td>
<td>0.102</td>
<td>0.00196</td>
<td>0.0451</td>
<td>0.743</td>
<td>0.54</td>
<td>65.6</td>
</tr>
<tr>
<td>Average</td>
<td>0.00255</td>
<td>0.1035</td>
<td>0.00205</td>
<td>0.0444</td>
<td>0.733</td>
<td>0.535</td>
<td>65.4</td>
</tr>
<tr>
<td>6N-6N-1</td>
<td>0.00252</td>
<td>0.095</td>
<td>0.00221</td>
<td>0.0435</td>
<td>0.752</td>
<td>0.545</td>
<td>65.85</td>
</tr>
<tr>
<td>6N-6N-2</td>
<td>0.0025</td>
<td>0.1</td>
<td>0.00215</td>
<td>-</td>
<td>0.778</td>
<td>-</td>
<td>66.4</td>
</tr>
<tr>
<td>Average</td>
<td>0.00251</td>
<td>0.0975</td>
<td>0.00218</td>
<td>0.0435</td>
<td>0.765</td>
<td>0.545</td>
<td>66.13</td>
</tr>
<tr>
<td>7N-7N-1</td>
<td>0.002605</td>
<td>0.0755</td>
<td>0.00248</td>
<td>-</td>
<td>0.8</td>
<td>-</td>
<td>68</td>
</tr>
<tr>
<td>7N-7N-2</td>
<td>0.00258</td>
<td>0.077</td>
<td>0.00239</td>
<td>0.0383</td>
<td>0.995</td>
<td>0.55</td>
<td>67.6</td>
</tr>
<tr>
<td>Average</td>
<td>0.002593</td>
<td>0.0763</td>
<td>0.00244</td>
<td>0.0383</td>
<td>0.89</td>
<td>0.55</td>
<td>67.8</td>
</tr>
<tr>
<td>11N-11N-1</td>
<td>0.00248</td>
<td>0.122</td>
<td>0.00223</td>
<td>0.0288</td>
<td>0.89</td>
<td>0.374</td>
<td>63.3</td>
</tr>
<tr>
<td>11N-11N-2</td>
<td>0.00255</td>
<td>0.12</td>
<td>0.00193</td>
<td>0.0235</td>
<td>0.81</td>
<td>0.365</td>
<td>63</td>
</tr>
<tr>
<td>Average</td>
<td>0.00252</td>
<td>0.121</td>
<td>0.00208</td>
<td>0.0262</td>
<td>0.85</td>
<td>0.37</td>
<td>63.15</td>
</tr>
<tr>
<td>5N-6H-1</td>
<td>0.00258</td>
<td>0.108</td>
<td>0.00158</td>
<td>0.024</td>
<td>0.54</td>
<td>0.26</td>
<td>64.75</td>
</tr>
<tr>
<td>5N-6H-2</td>
<td>0.00249</td>
<td>0.106</td>
<td>0.00157</td>
<td>0.0253</td>
<td>0.55</td>
<td>0.266</td>
<td>65.1</td>
</tr>
<tr>
<td>Average</td>
<td>0.00253</td>
<td>0.107</td>
<td>0.001575</td>
<td>0.0247</td>
<td>0.545</td>
<td>0.263</td>
<td>64.9</td>
</tr>
<tr>
<td>6N-7H-1</td>
<td>0.00255</td>
<td>0.096</td>
<td>0.00149</td>
<td>0.0258</td>
<td>0.6</td>
<td>0.31</td>
<td>65</td>
</tr>
<tr>
<td>6N-7H-2</td>
<td>0.002435</td>
<td>0.094</td>
<td>0.00157</td>
<td>0.026</td>
<td>0.586</td>
<td>0.301</td>
<td>65.6</td>
</tr>
<tr>
<td>Average</td>
<td>0.002493</td>
<td>0.095</td>
<td>0.00153</td>
<td>0.0259</td>
<td>0.593</td>
<td>0.305</td>
<td>65.3</td>
</tr>
<tr>
<td>6N-9H-1</td>
<td>0.00252</td>
<td>0.0975</td>
<td>0.00131</td>
<td>0.0156</td>
<td>0.46</td>
<td>0.21</td>
<td>69.11</td>
</tr>
<tr>
<td>6N-9H-2</td>
<td>0.00246</td>
<td>0.0965</td>
<td>0.00129</td>
<td>0.0157</td>
<td>0.49</td>
<td>0.24</td>
<td>67.4</td>
</tr>
<tr>
<td>Average</td>
<td>0.00249</td>
<td>0.097</td>
<td>0.13</td>
<td>0.01565</td>
<td>0.475</td>
<td>0.225</td>
<td>68.25</td>
</tr>
</tbody>
</table>

(-) means that no measurement could be obtained

5.3 Strain Penetration Test

This test was only conducted in Phase two. The bar slip, or strain penetration of the bar into the grouted coupler is an important segment that contributes to the overall column lateral deformation. Figure 5.11 illustrates strain penetration for one of the specimens (6N-6N-1 specimen). Therefore, during the static monotonic tensile tests, bar slip was measured for
each end of the coupler using a set of three LVDTs. The LVDTs were mounted around the steel bar in a circular pattern. The LVDTs were to be measuring deformation along a distance of approximately 2 in, measured from the sleeve edge to the point on the reinforcing bar (refer to Figure 2.8).

Two strain gauges were instrumented in the middle of the 2-in distance on opposite sides of the bar to measure bar strain. To obtain the bar slip ($\delta$), the average deformation measurement of the three LVDTs was calculated and it is denoted as ($\delta_{LVDT}$). Then the average strain gauge measurement was obtained and multiplied by the bar length within the 2-in distance and it is denoted as ($\delta_{SG}$). Finally, the bar slip was determined by deducting the deformation measured by strain gauges ($\delta_{SG}$) from the deformation measured from the LVDTs ($\delta_{LVDT}$).

Figure 5.12 and Figure 5.13 show the stress-slip behavior of some representative samples. All other details are shown in Table 5.3. The parameters $\delta_y$ and $\delta_u$ are the slip at yield and
ultimate stresses, respectively. Results showed that ultimate slip increased with bar size. Also, the bar sizes that were spliced with larger sleeve sizes had more slip than bar sizes which were spliced with the same sleeve size. For example, #5 bar spliced in #6 sleeve size had more slip than #5 bar spliced in #5 sleeve size. It is due the less confinement that the larger sleeve size provides compared with the same size sleeve.

Another observation was that the slip at the wide end is always slightly greater than that at the narrow end. This is because the wide end has a prismatic cross section as opposed to the narrow end that has a tapered cross section. The tapered cross section improves the confinement and decreases the slip. The ultimate slip of HS bars ranged between 0.0043-0.005 in except for the large size specimens (11N-14H) where the average ultimate slip was 0.0122 in. Slip in that range can be neglected for design purposes.

![Stress-slip behavior of sample 6N-6N-1](figure.png)

Figure 5.12: Stress-slip behavior of sample 6N-6N-1
5.4 Analytical Modeling of Strain Penetration

In previous studies, several bond stress-slip relationships for the interface between steel bars and concrete subjected to axial pullout were proposed. These relationships were classified in terms of bond stress distribution. One type adopts a piecewise uniform distribution; bond stress is idealized as one or two segments of uniform bond stress along the whole embedment length [5, 68, 43]. The other type idealizes the interface as a local non-uniform bond stress-slip relationship, which mimics the true mechanical behavior of the interface [25, 63, 19].

In the present study, the strain penetration, bar slip of steel bars into the grout, is quantified using the second approach mentioned above by idealizing local non-uniform bond slip relationship between the bar and the surrounding high-strength grout. The adopted relationship follows the trend adopted by Eligehausen et al. [25], Steuck et al. [71] but with different bond
stress and slip parameters. These parameters were obtained using inverse analysis by reproducing the measured stress-strain behavior of the coupler region and stress-slip at sleeve ends.

Table 5.3: Slip results at yield and ultimate stress

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Narrow End</th>
<th>Wide End</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\delta_y$</td>
<td>$\delta_u$</td>
</tr>
<tr>
<td>5N-5N-1</td>
<td>0.0053</td>
<td>0.16</td>
</tr>
<tr>
<td>5N-5N-2</td>
<td>0.0055</td>
<td>0.162</td>
</tr>
<tr>
<td>6N-6N-1</td>
<td>0.006</td>
<td>0.167</td>
</tr>
<tr>
<td>6N-6N-2</td>
<td>0.0057</td>
<td>0.165</td>
</tr>
<tr>
<td>7N-7N-1</td>
<td>0.0076</td>
<td>0.206</td>
</tr>
<tr>
<td>7N-7N-2</td>
<td>0.0073</td>
<td>0.199</td>
</tr>
<tr>
<td>11N-11N-1</td>
<td>0.011</td>
<td>0.29</td>
</tr>
<tr>
<td>11N-11N-2</td>
<td>0.01</td>
<td>0.293</td>
</tr>
<tr>
<td>5N-6H-1</td>
<td>0.0061</td>
<td>0.168</td>
</tr>
<tr>
<td>5N-6H-2</td>
<td>0.0063</td>
<td>0.172</td>
</tr>
<tr>
<td>6N-7H-1</td>
<td>0.0065</td>
<td>0.178</td>
</tr>
<tr>
<td>6N-7H-2</td>
<td>0.0068</td>
<td>0.181</td>
</tr>
<tr>
<td>6N-9H-1</td>
<td>0.0075</td>
<td>0.23</td>
</tr>
<tr>
<td>6N-9H-2</td>
<td>0.0078</td>
<td>0.25</td>
</tr>
<tr>
<td>11N-14H-1</td>
<td>0.0133</td>
<td>0.327</td>
</tr>
<tr>
<td>11N-14H-2</td>
<td>0.0135</td>
<td>0.33</td>
</tr>
</tbody>
</table>

(-) means that the bar (HS bar) was elastic

A nonlinear one-dimensional finite element (FE) model was used to investigate the bond slip of reinforcing bars embedded in grouted sleeves. The FE model was developed in OpenSees...
based on the general schematics depicted in Figure 5.14.

The FE model was composed of a series of discretized reinforcing bars (using nonlinear truss elements) connected to bond slip springs (represented by zero-length elements) at each node. The springs were connected to discretized sleeves (using nonlinear truss elements).

The material “ReinforcingSteel” available in OpenSees was used to model steel bar elements based on the measured stress-strain behavior of the bars in the study. The constitutive stress-strain model of sleeves was modeled with a bi-linear curve using the data reported by MDOT report [36]. The material “Steel02” was used to model the sleeves using the data: elastic modulus of 24500 ksi, yield stress of 76 ksi, ultimate stress of 131 ksi, and ultimate strain of 12.9%. Spring elements were modeled with a constitutive model similar to the model proposed by Eligehausen et al. [25] but with a different shear stress and slip values.

A discretization scheme of 100 bond slip springs for each bar was found to be sufficient to obtain the full stress capacity of the spliced bars. Since all GS splice samples in the study failed by rupturing the steel bar away from the sleeve, the full local shear stress-slip behavior cannot be predicted. Therefore, two segments of the curve were modeled: the ascending and the plateau portions. The objective of the FE model was to find the parameters of the constitutive bond slip model. Figure 5.15 depicts the measured and predicted stress strain behavior over the coupler region of a representative sample. The measured stress-slip at sleeve ends (narrow and wide ends) of the same sample along with the predicted curves is shown in Figure 5.16 and Figure 5.17.
Nonlinear bond slip spring

Nonlinear truss elements (bar)

Nonlinear truss elements (sleeve)

Figure 5.14: Schematics of proposed bond slip modeling

Figure 5.15: Predicted and measured stress-strain curve of the coupler region for 6N-6N-1
Figure 5.16: Predicted and measured stress-slip curve of narrow end for 6N-6N-1

Figure 5.17: Predicted and measured stress-slip curve of wide end for 6N-6N-1
Using the analytical models for all samples, three bond slip models were proposed. The first model was for the narrow end (factory end) that utilizes NS bar. The second was for the wide end (field end) that utilizes NS bar. The third model was for the wide end that utilizes HS bar. The proposed models are depicted in Figure 5.18. Also, the models by Eligehausen et al. [25] and Steuck et al. [71] are plotted for # 8 bar size. The models for NS bar are close to the model by Steuck et al. [71], which is not unexpected since their model was for bars embedded in high-strength grouted ducts. The local slip in the proposed models does not rely on the bar diameter as proposed by Steuck et al. [71]. The maximum shear stress, $\tau_g$, for the proposed models of NS bar is $46\sqrt{f'g}$. The local slip at yield, $\delta_1$, and failure, $\delta_2$, for NS bar at the narrow end is 0.02 in and 0.045 in, respectively. For the wide end of NS bar, these local slip values are 0.025 in and 0.06 in, respectively. The proposed model for HS bar is linear and it suggests that this region in the sleeve can be assumed rigid for simplicity.

To compare this approach with the constant bond stress approaches, the shear stress distribution along the interface of the sample (5N-5N-1) for the factory end (narrow end) is depicted in Figure 5.19. The shear stress is shown for yielding and ultimate states along with the approaches adopted by ACI 318 [3] and Haber [28] for regular bar-concrete and GS splice connections, respectively. It is obvious that ACI 318 underestimates the shear stress thus overestimating the resulting slip of the joint. On the other hand, Haber’s approach overestimates the shear stress thus underestimating the resulting slip. Haber’s adopted bond stress ($32.5\sqrt{f'g}$) is basically a modification of ACI 318 equation for constant bond stress. To sum up, the local shear stress-slip law provides a more accurate approach to capture the slip of bar-concrete or bar-grout joints.
Figure 5.18: Proposed bond slip models of bars embedded in GS couplers

Figure 5.19: Shear stress distribution of 5N-5N-1 for the factory end
CHAPTER 6: ANALYTICAL MODELING OF COLUMNS

6.1 Background

This chapter presents the analytical models adopted for the large-scale columns presented in this study. The OpenSees framework was used to model the columns using three-dimensional modeling. First, material constitutive models and element types are introduced followed by the developed analytical models.

6.2 Materials

6.2.1 Unconfined Concrete

Unconfined concrete was modeled using the “Concrete01” material. The constitutive law for this material follows the Kent-Scott-Park concrete model [66]. This material incorporates degrading linear loading and unloading stiffness following the work of Karsan and Jirsa [37] and does not incorporate tensile strength. The material exhibits a residual stress of $f'_{cu}$ after $\epsilon_{cu}$ is reached. The unconfined concrete material was used to represent the concrete cover. Figure 6.1 depicts the constitutive model for this material. The material requires four input parameters.

The parameters numerical values for unconfined concrete material are: 1) strain at peak stress, $\epsilon_{co} = 0.002$; 2) strain at failure, $\epsilon_{cu} = 0.005$; 3) peak stress, $f'_c$ and 4) stress at failure, $f'_{cu} = 0$. All parameters for unconfined concrete except the peak stress, $f'_c$, were kept constant for all column models and were based on the recommendations of Caltrans SDC. The peak stress, $f'_c$, was the measured average compressive strength of column concrete at the day of
testing. The stress at failure, $f'_{cu}$, was selected to be zero since the concrete cover would not have residual strength after spalling.

![Diagram of stress-strain relationship](image)

**Figure 6.1: Constitutive model for “Concrete01” material**

6.2.2 **Confined Concrete**

Confined concrete was modeled using the “Concrete01” material as well. This material is used to model the concrete core confined by the transverse reinforcement. Mander’s model [46] was used to determine the material parameters. Based on the model, the longitudinal compressive concrete stress $f_c$ is given by:

$$f_c = \frac{f'_{cc} x r}{r - 1 + x^r} \quad (6.1)$$
where $f'_{cc}$ is the compressive strength of confined concrete; and

$$x = \frac{\epsilon_c}{\epsilon_{cc}}$$  \hspace{1cm} (6.2)$$

$$r = \frac{E_c}{E_c - E_{sec}}$$  \hspace{1cm} (6.3)$$

where $\epsilon_c$ is the longitudinal compressive concrete strain and $\epsilon_{cc}$ is the strain at the confined compressive strength of concrete, which is given by Equation 6.4:

$$\epsilon_{cc} = \epsilon_{co} \left(1 + 5 \left(\frac{f'_{cc}}{f'_c} - 1\right)\right)$$  \hspace{1cm} (6.4)$$

where $E_c = 57000 \sqrt{f'_c}$ (psi) is the elastic modulus of concrete, and $E_{sec}$ is the secant modulus of concrete determined from Equation 6.5.

$$E_{sec} = \frac{f'_{cc}}{\epsilon_{cc}}$$  \hspace{1cm} (6.5)$$

The four required parameters for confined concrete to be used in “Concrete01” material are: 1) strain at peak stress, $\epsilon_{cc}$; 2) ultimate strain, $\epsilon_{cu}$; 3) peak stress, $f'_{cc}$ and 4) ultimate stress, $f'_{cu}$.

The confined compressive strength of concrete can be determined from Equation 6.6:

$$f'_{cc} = f'_c \left(-1.254 + 2.254 \sqrt{1 + \frac{7.95f'_{cc}}{f'_c} - \frac{2f'_{l}}{f'_c}}\right)$$  \hspace{1cm} (6.6)$$

where $f'_{l}$ is the effective lateral confining stress which is defined in Equation 6.7.

$$f'_{l} = \frac{1}{2} k_e \rho_s f_{yh}$$  \hspace{1cm} (6.7)$$
where \( k_e \) is the confinement effectiveness coefficient, defined in Equation 6.8, and \( \rho_s \) is ratio of volume of transverse reinforcement to the volume of confined concrete core by the transverse reinforcement, Equation 6.9.

\[
k_e = \frac{1 - \frac{s'}{2d_s}}{1 - \rho_{cc}} \quad (6.8)
\]

\[
\rho_s = \frac{4A_{sp}}{d_s s} \quad (6.9)
\]

The parameters are clear vertical spacing between transverse reinforcement \( s' \), diameter of confined concrete core measured between the center-line of the transverse reinforcement \( d_s \), ratio of area of longitudinal reinforcement to area of core of section \( \rho_{cc} \), cross-sectional area of the transverse reinforcement (spiral) \( A_{sp} \), center-to-center vertical spacing of transverse reinforcement \( s \), and nominal yield stress of transverse reinforcement \( f_{yh} \).

The ultimate strain for confined concrete, \( \epsilon_{cu} \), is determined from Equation 6.10 [59]:

\[
\epsilon_{cu} = 0.004 + 1.4\rho_s f_{yh}\epsilon_{su} \frac{f'_{cc}}{f_{cc}} \quad (6.10)
\]

where \( \epsilon_{su} \) is the strain at maximum stress of transverse reinforcement. The ultimate stress, \( f'_{cu} \) is obtained using Equation 6.1 at \( \epsilon_c = \epsilon_{cu} \).

6.2.3 Reinforcing Steel Bars

The response of longitudinal reinforcing steel bars was modeled by the material “Steel02” available in OpenSees. The backbone of this material is shown in Figure 6.2. The input parameters for this material are: yield stress \( f_y \), initial modulus \( E_s \), which can be directly
obtained from monotonic tension test of steel bars, hardening ratio \((b)\), and the parameters \((R_0, C_{R1}, C_{R2})\) which control the transition from elastic to plastic branches. The recommended values for the transition parameters in this study are 18, 0.925, and 0.15, respectively. Moreover, the cast-iron sleeves were modeled using the same material, “Steel02”, based on the data reported by Jansson [36] and detailed in Chapter 5.

![Constitutive model for “Steel02” material](image)

**Figure 6.2: Constitutive model for “Steel02” material**

### 6.2.4 Low-cycle Fatigue Modeling

Fracture due to low-cycle fatigue of longitudinal reinforcing bars is one of the most common failure mechanisms of a well-confined RC columns subjected to a strong seismic event. During these events, large plastic strain reversals occur in the longitudinal steel bars with strain amplitudes more than 0.06 in some cases [45]. Hawileh et al. [33] conducted an experimental study on ASTM A706 and A615 steel bars and reported that as little as seven full cycles of...
0.06 strain can result in low-cyclic fatigue fracture of bars.

Coffin [21] and Manson [47] proposed a model to estimate the general fatigue life of a material subjected to any strain reversals. The model relates the strain amplitude to the number of cycles to failure and is expressed in Equation 6.11:

\[ \epsilon_a = \frac{\Delta \epsilon_a}{2} = \frac{\sigma_f}{E} (2N_f)^b + \epsilon_f' (2N_f)^c \]  

Where \( \epsilon_a = \) total strain amplitude, \( \Delta \epsilon_a = \) total strain range (\( \epsilon_{max} + |\epsilon_{min}| \)), \( \sigma_f = \) fatigue strength coefficient, \( E = \) steel modulus of elasticity, \( 2N_f = \) number of half cycles to failure, \( b = \) fatigue strength exponent, \( \epsilon_f' = \) fatigue ductility coefficient, \( c = \) fatigue ductility exponent.

The first term in Equation 6.11 represents the elastic strain component (high-cycle fatigue) and the second term represents the plastic strain component (low-cycle fatigue). Koh and Stephens [38] found that for most low-cycle fatigue analyses, the elastic part can be neglected and Equation 6.11 can be simplified to:

\[ \epsilon_a = \frac{\Delta \epsilon_a}{2} = \epsilon_f' (2N_f)^c \]  

\( \epsilon_f' \) and \( c = \) constants that can be experimentally determined to develop a fatigue-life model for a given material. They are typically determined using strain-based uniaxial tension/compression tests. In the present study, these constants were taken from the reported data of Hawileh et al. [33] since they conducted experiments on ASTM A615 Gr. 60 steel bars with different bar sizes.

The “Fatigue” material which is available in OpenSees was used to define the low-cycle fatigue of the column longitudinal steel bars. The material uses a modified rainflow cycle
counting algorithm to accumulate damage in a material using Miners Rule. The “Fatigue” material was wrapped around “Steel02” material used to define the constitutive response of the longitudinal steel bars. Once the “Fatigue” material reaches a damage level of 1.0, the force (or stress) of the “Steel02” material become zero. The input parameters for this material are the Coffin-Manson constants, $\epsilon' f$ and $c$, which were taken as 0.101 and -0.428, respectively, as reported by Hawileh et al. [33]. Although bar buckling typically occurs prior to low-cycle fatigue fracture, it is not incorporated into the determination of failure.

6.2.5 Bond Slip Modeling

When the column section experiences moment or shear force demand, axial tensile and compressive stresses are developed in the steel bars. The bar tensile stresses must be transferred into the footing through bond between the bar and footing concrete. Strain accumulates at the column-footing interface, which causes the bar to deform or slip relative to the footing concrete. This results in concentrated rotation at the column-footing interface, which causes additional displacement to the column. Previous research studies have observed that bond-slip rotation at the column-footing interface can contribute as little as 15-20% [73] and as much as 50% [40] to the lateral displacement of a column. Therefore, developing analytical models for columns depends on a reliable bond slip modeling; not based on constant bond stress distribution.

Previous studies have used several approaches to model the bond slip at the joints of beam-column or column-footing. Wehbe et al. [73] developed a procedure to include bond-slip rotation in the responses of RC columns using a modified moment-rotation relationship. The moment-rotation is included in the model using rotational springs elements at the member ends. The procedure to calculate bond slip rotation is illustrated in Figure 6.3. The proce-
The method assumes that the bond slip rotation occurs about the neutral axis of the member. It also assumes that the bond stress over the bar embedded length is constant, which results in a linear or bilinear bar strain distribution.

![Diagram showing bond stress and strain profiles](image)

**Figure 6.3: Wehbe’s method for computing bond-slip rotation**

Moment-curvature analysis is used to determine the tensile strain and stress in the extreme reinforcing bar and the neutral axis location, \( c \). Then the slip, \( \delta \), in the extreme bar can be calculated by integrating the strain from the footing surface over the development length \( l_d \), Equation 6.13. For this specific strain profile, Equation 6.13 can be simplified as follows in Equation 6.14.

\[
\delta = \int_0^{l_d} \epsilon(x) dx \quad (6.13)
\]
\[ \delta = \begin{cases} \frac{\epsilon s l_1}{2} & \text{if } \epsilon_s \leq \epsilon_y \\ \frac{\epsilon y l_1}{2} + \frac{(\epsilon_s + \epsilon_y) l_2}{2} & \text{if } \epsilon_s > \epsilon_y \end{cases} \quad (6.14) \]

The lengths \( l_1 \) and \( l_2 \) are computed using Equation 6.15 and Equation 6.16, respectively.

\[ l_1 = \frac{f_s d_b}{4u} \quad (6.15) \]

\[ l_2 = \frac{(f_s - f_y)d_b}{4u} \quad (6.16) \]

where \( u \) is the bond stress which is given earlier in Chapter 2 in Equation 2.1. Once the slip, \( \delta \), is determined, the rotation at the column-footing interface can be computed using Equation 6.17.

\[ \theta = tan^{-1}\left(\frac{\delta}{c - d}\right) \quad (6.17) \]

The parameters are location of neutral axis determined from moment-curvature analysis \( c \) and distance from the center of extreme bar to the opposite concrete edge of the column \( d \). Since the moment rotation response is based on an initial moment-curvature analysis of the column section, the effect of axial load variation during the column analysis is not captured in this procedure.

A different approach was proposed by Zhao and Sritharan [77]. They proposed a constitutive stress-slip model for reinforcing steel bars using existing experimental data from the literature. The model can be used in fiber based analysis of RC members at the member ends. This model is integrated into the analysis using additional element such as “zero-length section” element which is available in OpenSees. The researchers obtained good correlation between the measured and calculated local and global response of some column and joint
tests. The model was investigated for conventional CIP connections only and it assumes that the concrete in the section be subjected to significant deformations beyond the strain capacity of the confined concrete. The researchers justified this by the location of the section at the member end which is provided additional confinement from the adjacent member. However, this approach is not appropriate for modeling any steel bar slip at intermediate levels along the structural member such as the bar slip above the grouted sleeves in GS precast columns.

Another simple approach was developed by Tazarv [72] using a modified stress-strain relationship for reinforcing steel fibers. The modified material is used in the fiber section to implicitly include the bond-slip effect in fiber-based analysis. The modification is imposed on the strain of the reinforcing bars. The effective strain of the steel bar at the column-footing interface is modified to include a combination of bar force-deformation component and bar bond force-slip component. The approach does not require additional elements or pre-nonlinear analysis.

In the present study, a macroscopic approach was followed. Regions where bar-concrete or bar-grout interface are expected to have bar slip were modeled with local bond slip relationship which were discussed in Chapter 5. For CIP columns, the region would be along the footing depth and was represented by the model proposed by Eligehausen et al. [25]. For GS precast columns, there are three zones of bar slip:

1. Footing region: modeled with bilinear shear stress-slip curve following the model proposed by Eligehausen et al. [25].

2. Field dowel region: modeled with linear shear stress-slip curve which is proposed in Chapter 5 for high steel bars embedded in grouted sleeves.
3. Factory end region: modeled with bilinear shear stress-slip curve which is proposed in Chapter 5 for normal steel bars embedded in grouted sleeves.

The material used in OpenSees to model the shear stress-slip behavior of aforementioned three regions was “Steel01” material which is shown in Figure 6.4. the yield stress and strain, \( f_y \) and \( \epsilon_y \) were substituted by the maximum local shear stress, \( \tau_g \), and local yield slip, \( \delta_1 \), respectively. The modulus, \( E_p \) was used to be approximately zero to avoid numerical divergence.

![Constitutive model for “Steel01” material](image)

Figure 6.4: Constitutive model for “Steel01” material

### 6.2.6 Elastic-No Tension Material

The contact surface between the column longitudinal steel bar and the footing dowel bar was modeled with “Elastic-No Tension” material, also called “ENT” material. The material requires one input parameter which is the compression modulus, \( E \). The selected modulus was based on the measured value of steel modulus of the reinforcing bars of each column model.
6.3 Analytical Models of Columns

Previous researchers have attempted different approaches to model RC members. General three-dimensional (3-D) solid finite element model was used to model RC columns [27]. The model incorporated interface elements to capture the slip between the longitudinal steel bars and surrounding concrete. Other researchers used 3-D models to model pull-out tests of steel bars embedded in concrete [65, 44] which also incorporated interface elements to describe the bar slip. In these studies, local bond slip models such as that developed by Eligehausen et al. [25] were used to describe the constitutive low of the interface elements. These analytical models are computationally expensive.

Therefore, two-dimensional fiber-based models were used to lower the computational cost significantly. These models usually utilize one force-based beam element with several integration points to model the RC column. In these models, bar slippage were modeled using several methods such as those described in the previous section. The most common method is Wehbe’s method [73]. In that method, a rotational spring is employed at the location where the slip is expected to occur. Moment-rotation relation, which is discussed in previous section, is assigned to the spring. This method and other bond slip methods lack the physical representation of the actual column behavior.

In the present study, a 3-D discrete fiber-based model is proposed. Figure 6.5 and Figure 6.6 shows a conceptual view of CIP and precast columns, respectively. Several elements and fiber sections are plotted for convenience but they do not represent the actual numbers used. Note the approach utilizes existing 3-D fiber-based elements, the novelty is in the discrete representation of the reinforcement and how compatibility is enforced. Figure 6.7 and Figure 6.8 depicts side-view schematics for analytical modeling of CIP and precast columns, respectively.
The central part of the model is a nonlinear beam-column element with fiber sections that contain concrete fibers only. Along the column length, the fiber sections consist of confined and unconfined concrete for the concrete core and cover, respectively. Along the footing length, the fiber sections consist of confined concrete fibers only since it is difficult to create a circular section (confined concrete) enclosed by a square fiber section which represent the remaining footing. The fiber sections in the column shaft are discretized with a radial discretization scheme with 16 radial core divisions, 24 transverse core divisions, 2 radial unconfined cover divisions, and 24 transverse cover divisions. Fiber sections in the footing are discretized with 60 square divisions in each direction.

The elements that surround the central portion are truss elements that represent the longitudinal steel bars in the concrete cross section. The horizontal elements that connect the central portion to the truss elements are elastic beam elements which were created with high cross sectional area and high stiffness (rigid elements).

Bond slip springs were used along the longitudinal dowel-concrete interface in the footing region for CIP and GS precast columns. Also, the springs were used in the interface between the longitudinal steel bars and the grouted sleeves for GS precast columns. The springs were zero-length elements with a “Steel01” material to define the corresponding constitutive models which are discussed in Chapter 5. The maximum local shear stress, $\tau_g$ for the springs was adjusted based on the element discretization scheme.

Node “1” was fixed and loading was applied to node “2” as depicted in Figure 6.7 and Figure 6.8. Constant axial load was applied first followed by the lateral cyclic displacement scheme as was done in the experiment. Prior to axial load application, the stiffness of all bond slip springs were made rigid. The reason for that was to avoid inducing initial prestressed forces and deformations in the springs which would influence the column behavior.
Also, the initial deformations would violate the assumption of plane section remaining plane. The parameterization framework was used in OpenSees to update the value of the spring stiffnesses. The command “Parameter” was used to set the initial rigid stiffness value for the springs. Then the constant axial load was applied in 10 steps at Node “2”. After that the command “updateParameter” was used to update the spring stiffnesses to their original values. Finally, the lateral cyclic displacement protocol was applied at Node “2”.

Figure 6.5: Conceptual view of the CIP analytical model
Figure 6.6: Conceptual view of the precast analytical model
Figure 6.7: Side view of the CIP analytical model
6.4 Analytical Results of Columns

The measured and calculated hysteresis loops of all columns are shown in Figure 6.9 through Figure 6.14. The analytical 2-D model is presented for columns C-40-1 and G-40-1 for comparison with the analytical 3-D model. Details about 2-D model can be found in the reference [28]. Both analytical models estimated the lateral load capacity, initial stiffness, and
failure due low-cycle fatigue very well. For C-40-1 and G-40-1 columns, it can be seen that the 2-D model simulated the experiment well but overestimated the unloading stiffness. This was caused by the spring effect on the overall behavior where the spring unloading stiffness depends on an empirical factor, $\beta$, which was taken as 0.3. The 3-D model predicted bar fracture due to low-cycle fatigue better than 2-D model as seen in G-40-1 column.

To better compare the hysteresis behavior of 2-D and 3-D modeling procedures with the measured response, the energy dissipation was calculated and shown in Figure 6.15 through Figure 6.20 for cycles 1 and 2 for all columns up to a reliable drift level. The energy dissipation using 2-D approach was obtained only for C-40-1 and G-40-1 columns. It can be seen generally that energy dissipation from the modeling approaches is comparable to the measured response. Both modeling approached showed higher energy dissipation that the measured response. The 3-D model showed better results that 2-D model for both 1st and 2nd cycles.
Figure 6.9: Hysteresis curves for C-40-1

Figure 6.10: Hysteresis curves for G-40-1
Figure 6.11: Hysteresis curves for G-40-2

Figure 6.12: Hysteresis curves for G-40-3
Figure 6.13: Hysteresis curves for C-25-1

Figure 6.14: Hysteresis curves for G-25-1
Figure 6.15: Energy dissipation for C-40-1

Figure 6.16: Energy dissipation for G-40-1
Figure 6.17: Energy dissipation for C-25-1

Figure 6.18: Energy dissipation for G-25-1
Figure 6.19: Energy dissipation for G-40-2

Figure 6.20: Energy dissipation for G-40-3
CHAPTER 7: CONCLUSION

7.1 Summary

Accelerated bridge construction (ABC) is gaining significant attention in the US since it offers many advantages compared with conventional cast-in-place (CIP) construction such as reduced construction time improved product quality, reduced traffic interruptions, maximized work zone safety and reduced cost. A key feature in ABC methods is the use of precast concrete elements. Connection of these elements is typically completed by using mechanical couplers. Grouted sleeve (GS) coupler is one of the most popular mechanical splices in the market since it offers good construction tolerance and great load transfer between the precast elements.

Although ABC has been used in low seismic regions, their use in moderate to high seismic regions is limited, especially for substructure element connections, due to lack of seismic performance data for such connections. Connection regions for precast substructure elements typically coincide with plastic hinge zones. Thus, under earthquake events these connections are subject to high deformation demands.

Bridge columns with grouted sleeve connections have only been subject to a limited number of investigations in the US. However, research thus far has indicated some performance issues related to this type of connection detail for seismic applications. Given the demand for ABC and popularity of GS coupler connections, there is a need to develop improved connections details.

The purpose of this study was to improve the seismic performance of GS precast column-footing connection details using the shifted plastic hinging (SPH) concept. The improved
detailing also aimed at reducing the damage in adjacent elements. The connection was tested and evaluated for use in ABC in moderate to high seismic zone regions. Based on the experimental results, design expressions and procedures were developed.

The study incorporated two phases of experimental testing of large scale circular bridge column models. In the first phase, four 0.42-scale columns were designed, fabricated and tested. They included two sets of columns, one with aspect ratio of 4.0 and the other with aspect ratio of 2.5. In each set, one column was CIP column as a reference and the other column was a precast column utilizing GS connection. In the second round, Two precast columns with GS connections were tested: one had a 0.42-scale and the other had a 0.33-scale. Both had aspect ratio of 4.0. The GS precast columns in both phases were designed using SPH methodology.

Along with column tests, the study incorporated uni-axial tests of the GS coupler components. The tests included monotonic tension test and strain penetration test. Along with the experimental uni-axial testing program, one-dimensional analytical modeling of the GS splices was proposed and bond slip constitutive models of the bar-grout interface were obtained by reverse analysis.

The study also included analytical investigations on the column models using OpenSees. A three dimensional discrete fiber-based model was proposed for both CIP and GS precast columns. The model featured more physical representations than the commonly used two-dimensional fiber-based models. The model made use of the proposed bond slip constitutive laws for GS couplers. The analytical results were compared to the experimental results to validate the modeling procedures.
Findings from the experimental tests and numerical studies on the large scale precast columns and grouted splices components led to the following conclusions:

1. The precast columns produced the expected lateral load capacity which was determined from section analysis above the coupler region.

2. The precast columns achieved significantly improved displacement ductility using the proposed GS connection detail compared with previous researched GS connections.

3. For shear-critical column, The failure mode, displacement ductility reduction, strain profiles indicated that SPH procedure should be more investigated.

4. Significant reduction of damage in capacity-protected element of the precast columns was noticed using SPH detailing with GS connections.

5. Precast columns achieved comparable dissipation energy compared with CIP columns which qualifies them for use in moderate-to-high seismic zones.

6. Bond slip in the footing of the precast columns had significant contribution to the overall column displacement but less that for CIP columns.

7. Flexural deformation component from the coupler region contributed between 10-20% to the total column displacement.

8. The failure mode of GS spliced steel bars was bar rupture away from the coupler region which indicated that the coupler was able to fully develop the ultimate stress in the spliced bars.
9. Strain penetration into the grouted couplers was found to sufficiently enough to induce significant rotation directly above the coupler region and increase the rotational capacity of the column thus increase the displacement ductility.

10. The proposed bond slip models for GS couplers were found more accurate than previous methods. The models were useful in the proposed analytical model for the precast column.

11. The 3-D discrete analytical models simulated the experimental tests very well and were better than the commonly used 2-D models.

12. The 3-D proposed model achieved significantly faster computations than conventional continuum 3-D finite element models.

13. The 3-D proposed model also exhibited a better physical representation than other available 2-D fiber-based models since it did not incorporate modified stress-strain relationships or additional analyses which are necessary parts in 2-D fiber-based models.

7.3 Recommendations and Future Work

Based on the experimental observations, authors recommended several things:

1. More focus needs to be put on the use of SPH with GS connections in shear-critical columns. The priority should be given to well-confined columns; G-25-1 column in this study used the minimum transverse reinforcement ratio.

2. A debonded length of the longitudinal steel bars in the column above the couplers directly can be provided to increase the degree of improvement of using SPH detailing with GS connections.
3. Investigations on different aspect ratios and axial load intensities are recommended to extend the range of findings.

4. Repair of precast columns is an appealing subject that needs to be investigated for the connection detail in this study. Repairing such connection is a challenge since shifting the hinge above the damaged SPH is not going to recover the displacement ductility.

5. The concept of SPH can be extended for use in other connection types. An example for that can be the use of SPH with ultra high performance concrete (UHPC) for precast column-footing connections.
LIST OF REFERENCES


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