CONCRETE FILLED STEEL TUBE CONNECTIONS FOR PRECAST CONSTRUCTION IN SEISMIC REGIONS

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ABSTRACT
Concrete filled steel tubes (CFSTs) are composite elements which consist of a steel tube with concrete infill, and offer an efficient alternative to conventional reinforced concrete construction including rapid construction and reduced material and labor costs. However, the use of CFSTs in the US is limited in-part due to a lack of standard connection details. This paper focuses on the development of practical connections for CFSTs for use in moderate and high seismic regions with a specific emphasis on connections to precast concrete components. Two types of connections are being investigated, including column-to-foundation and column-to-cap beam connections. Extensive numerical parameter studies and experimental work resulted in straight forward design and corresponding expressions for a column-to-foundation connection in which the steel tube is embedded into the foundation concrete. Development of the column-to-cap beam connection is more recent and is discussed in-depth here. This connection offers many unique design considerations including congested joint reinforcing and limits on geometry associated with the integration of precast super-structure components. Three categories of the CFST column-to-cap beam connection are being evaluated; an embedded connection similar to the proposed foundation connection, a connection in which headed reinforcing bars are welded to the inside of the steel tube and extended into the cap beam, and a traditional jacket RC connection in which a short independent cage of transverse and longitudinal column reinforcing extends from the steel tube into the cap beam. All connections were developed and evaluated for use with precast bent caps for the optimization of accelerated bridge construction. Numerical and experimental results indicate that the proposed connection types can achieve adequate strength and ductility when subjected to extreme lateral loading.

Keywords: Accelerated Construction, Connections, Designing and Testing Related to Seismic, Research

INTRODUCTION

Concrete filled steel tubes (CFSTs) are composite structural elements which provide large strength and stiffness while permitting accelerated bridge construction (ABC). The steel tube serves as formwork and reinforcement to the concrete fill, negating the need for reinforcing cages, shoring, and temporary formwork. In relation to ABC, the placement of the concrete fill may be further enhanced using self-consolidating concrete (SCC), so that concrete vibration is not required.

The steel tube is placed at the optimal location to resist bending forces, thereby maximizing strength and stiffness while minimizing weight and material requirements. In addition, the steel tube provides optimal confinement and much greater shear strength than spiral reinforcement, which is typically used for circular reinforced concrete columns. In addition, the concrete fill restrains local tube buckling, supports compressive stress demands, and offers large stiffness to meet functionality seismic performance objectives and non-seismic load requirements. Shear stress transfer must occur between the steel tube and concrete fill to ensure full composite action, which increases efficiency, resistance, and ductility, all of which are desirable properties for seismic design[1,2,3,4,5].

Although CFSTs offer many advantages in rapid construction and improved structural performance, connections between CFSTs are often different and more complex than those used in steel or reinforced concrete construction due to the composite nature of CFSTs. Prior numerical and experimental research resulted in straight forward design and corresponding expressions for an embedded column-to-foundation connection[4]. Results from that research are presented briefly here. The primary focus of this paper is the development and experimental investigation of robust CFST column-to-cap beam connections capable of sustaining cyclic lateral loads while minimizing damage and degradation. The study focus is on precast bent caps, since this benefits ABC, and practical design expressions are developed for these connections based upon the experimental research.

CFST COLUMN-TO-FOUNDATION CONNECTION

A foundation connection in which the steel tube is embedded into the foundation concrete has previously been developed, and is illustrated in Fig. 1[4,6,7,8,9]. This connection is capable of transferring the plastic moment capacity of the CFST, and can provide large lateral deformation capacities when appropriately designed as is illustrated by hysteresis in Fig. 2. The connection employs an annular ring which is welded to the base of the steel tube, and projects both inside and outside of the steel tube to provide anchorage and efficient shear and moment transfer to the surrounding concrete and reinforcement, as is illustrated by the compression struts in Fig. 1. There are no internal shear connectors, dowels, or reinforcing bars penetrating from the tube into the foundation; the force transfer is solely accomplished by the anchorage provided by the tube. The foundation is designed to normal depth, design loads, and flexural reinforcement.

Two methods for constructing the foundation connection have been developed and experimentally evaluated; a monolithic method in which the steel tube and annular ring are temporarily supported in the foundation concrete and the foundation and CFST column are cast simultaneously, and a grouted method in which the construction of foundation and CFST column are isolated (illustrated in Fig. 1b). The second method achieves the objectives of ABC by separating the construction of the foundation from the construction of the CFST column. Using this method, the footing is cast with a recess formed by a light weight corrugated pipe with an inner diameter slightly larger than the outer diameter of the annular ring as shown in Fig. 1b. The tube and ring are placed into the void after the foundation is cast, and the recess between the tube and corrugated pipe is filled with high strength fiber reinforced grout to anchor the column into the foundation. The fiber reinforced grout used in the connection should be non-shrinkage according to ASTM C 1107, and should meet durability requirements according to ASTM C666 and ASTM C1012. These requirements are specified in NCHRP Report 681 for emulative grouted connections in precast construction[10]. Detailed information regarding the grout and fiber properties as well as mixing and construction procedures are provided in reference material[6,7,8,9]. For both options, the steel tube is filled with low shrinkage self-consolidating concrete to complete the CFST column, and no vibration is required[5].

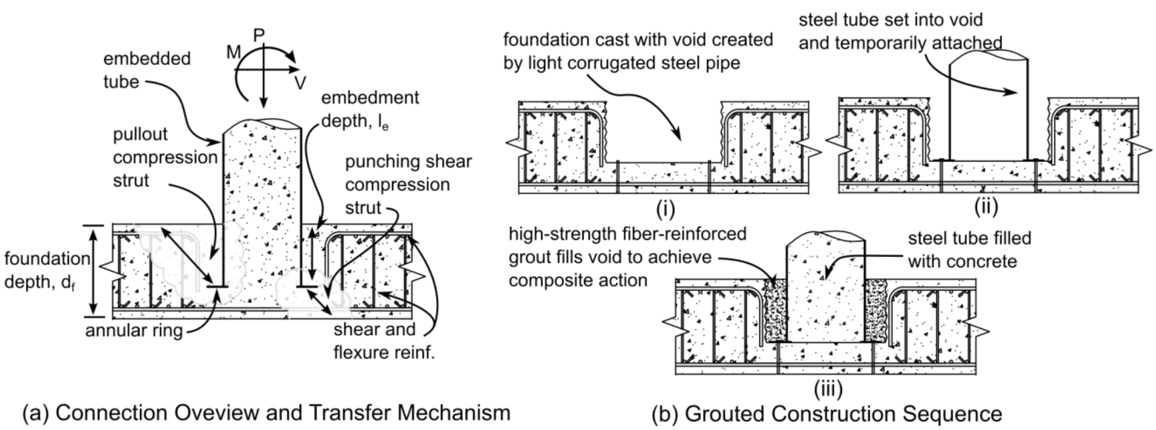

Fig. 1 CFST column-to-foundation connection

EXPERIMENTAL BEHAVIOR

The compilation of the experimental programs to evaluate the CFST column-to-foundation connection consisted of a series of 19 large-scale specimens which simulated approximately a half scale bridge column[4,6,7,8,9]. The diameter and thickness of the steel tube in a majority of the specimens were 20-in. and 0.25-in. respectively; resulting in a diameter-to-thickness ratio (*D/t*) of 80. This exceeds the limiting *D/t* ratio specified in ACI 318[11], but meets the requirements in the AASHTO LRFD[12] design specifications and the AISC Steel Construction Manual[13]. The annular ring in all specimens extended 16*t* (4-in.) and 8*t* (2-in.) from the outer and inner diameter of the steel tube respectively. The dimensions of the footing as well as the primary flexure reinforcing were selected to provide adequate strength for the foundation to minimize the influence of footing size on the failure mode, resist M_P of the CFST without yielding, and to represent a scale model of a typical bridge footing. The imposed displacement history for a majority of the specimens was based on the ATC-24 protocol[14], and a majority of the specimens were subjected to approximately 10% of the gross compressive load capacity of the CFST column.

As the testing program was so large, only the hysteretic performances of selected specimens are discussed here to demonstrate the influence of tube embedment depth on connection behavior. The moment drift behaviors of inadequately and adequately embedded specimens are shown in Fig. 2a and Fig. 2b respectively, while typical behaviors and failure modes are shown in Fig. 3[4]. The moments have been normalized to the theoretical plastic moment capacity of the CFST component as calculated using the plastic stress distribution method (PSDM), which is shown as a dashed line in each of the subfigures of Fig. 2. This method is illustrated in reference material[4]. In summary, the ductility of inadequately embedded connections was ultimately limited by foundation damage due to a conical pullout of the CFST from the foundation, as shown in Fig. 3a. In general, the failure mode of adequately embedded connections was characterized by ductile tearing of the steel tube which initiated as a result of local tube buckling as is illustrated in Fig. 3c. Furthermore, adequately embedded specimens exhibited a minimal decrease in resistance as a result of severe local buckling which generally initiated at around 4% drift, and had virtually no foundation damage at the end of testing as is shown in Fig. 3c. The drift levels achieved by the adequately embedded specimens at failure are significantly larger than those observed from similar size reinforced concrete pier and column base connections[4,6,7,8,9].

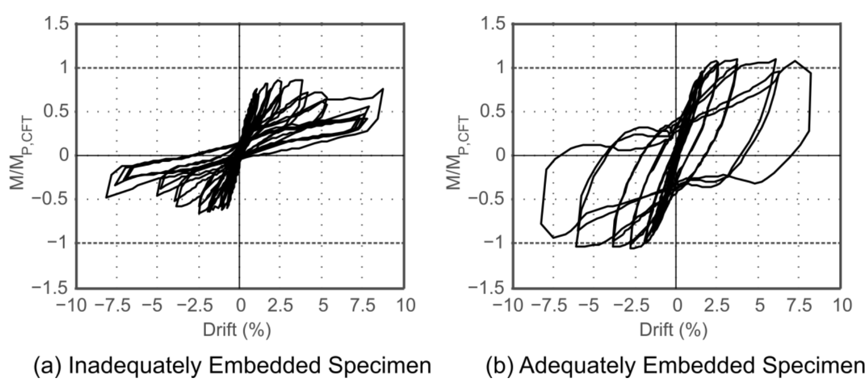

Fig. 2 Typical Moment-Drift Response from Adequately and Inadequately Embedded Specimens[6]

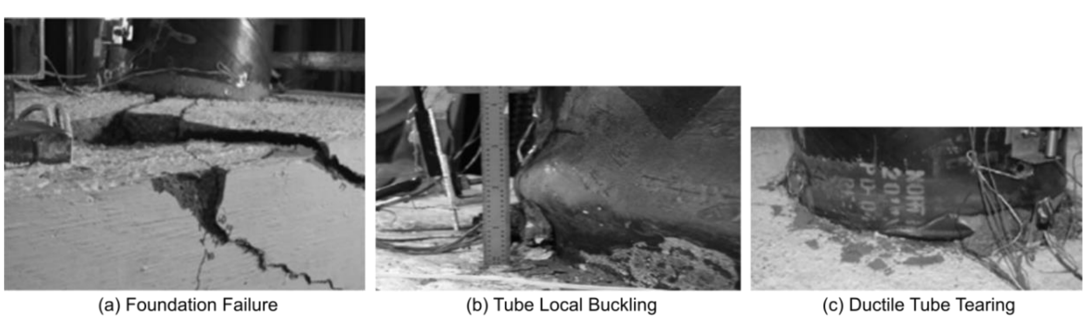

Fig. 3 Photos of Foundation Connection Behavior[6]

DESIGN EXPRESSIONS

The experimental results were used to develop design expressions for a CFST column-to-foundation connection capable of transferring the full moment capacity of the CFST. Specifically, expressions were developed to:

- dimension and detail the annular ring
- determine the required embedment depth of the tube to eliminate the conical pullout failure mode
- determine the amount of shear reinforcing required in the foundation
- determine the required depth of concrete below to the tube to prevent concrete punching failure.

These expressions are not discussed here for brevity; however detailed explanations are available in reference material[4].

CFST COLUMN-TO-CAP BEAM CONNECTION

While the numerical and experimental analyses conducted on the CFST foundation connection resulted in design expressions to support the use of CFST columns in highway bridges, full realization of the system requires the development of a range of cap beam connections. This connection offers unique challenges including congested joint reinforcing and limits on the width and height of the cap beam, which are parameters that have not been previously evaluated. Furthermore, the optimization of ABC requires exploring connections which are compatible with precast superstructure elements. To achieve these objectives, the continuing phase of this research is focused on the development robust CFST cap beam connections capable of sustaining cyclic lateral load demands while mitigating damage and degradation.

The proposed CFST column-to-cap beam connections are illustrated in Fig. 4. There are three connection types: (1) embedded ring connections (Fig. 1a), (2) welded dowel connections (Fig. 1b), and (3) reinforced concrete connections (Fig. 3c). This provides a suite of connections for designers, each option offering advantages as the project may require.

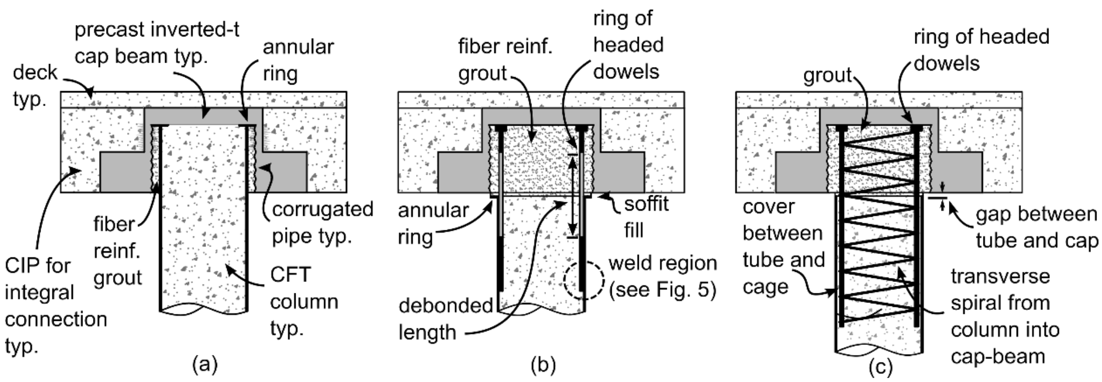

Fig. 4 Proposed CFST Column-to-Precast Cap Beam Connections. (a) Embedded Ring Connection (ER), (b) Welded Dowel Connection (WD), and (c) Reinforced Concrete Connection (RC)

Fig. 4a shows a full strength embedded ring connection (herein referred to as ER); this connection is similar to the embedded flange column-to-foundation connection evaluated in previous research[4]. The connection uses a grouted connection detail, with a void cast into a precast cap beam. A circular ring is welded to the steel tube to provide anchorage and transfer stress to the concrete and reinforcing in the cap beam. The flange extends a distance 8 times the thickness of the tube (8t) both inside and outside of the tube. The external projection of 8t is smaller than previous recommendations for the embedded foundation connection[4]. The precast cap beam is placed onto the column after the column is set, and the recess between the tube and corrugated pipe is filled with high strength fiber reinforced grout.

The connections illustrated in Fig. 4b and 4c utilize T-headed reinforcing dowels that extend from the CFST column into the cap beam to provide axial, moment, and shear transfer. These connections can be integrated into precast elements using a void similar to that described for the grouted CFST connection as shown in Fig. 4.

Fig. 4b shows a welded dowel connection (herein referred to as WD). The WD connection utilizes headed dowels to resist the flexural demand. The shear transfer to the tube is accomplished by welding the dowels to the steel tube using a flare bevel groove weld as illustrated in Fig. 5. The dowels are developed into the cap beam using a high-strength, fiber-reinforced grouted connection. Welding the dowel directly to the tube, as opposed to embedding the dowel directly into the connection maximizes the moment capacity of the dowel connection. A soffit fill depth is included between the steel tube and cap beam. A flange with an outer diameter of D+8t is welded to the exterior of the steel tube to increase compressive bearing area on the soffit fill (this dimension is indicated in Fig. 15). As illustrated in Fig. 4b, the dowels can be de-bonded in the column-to-cap beam interface region to increase the deformation capacity of the connection.

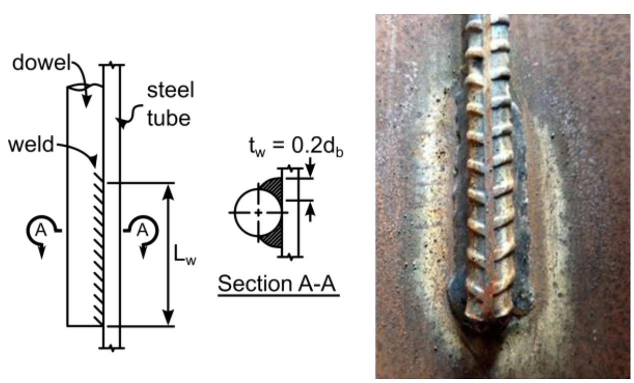

Fig. 5 Flare Bevel Groove Weld Between Longitudinal Dowel and Steel Tube

Fig. 4c shows a reinforced concrete connection (referred to as RC connection) in which a short independent cage for both transverse and longitudinal reinforcing extends from the

CFST column into the cap beam, and cover is provided between the reinforcing cage and steel tube within the column. A gap is left between the steel tube and cap beam to help focus the plastic hinging location between the CFST component and the cap beam[15].

SPECIMEN DESIGN

Eight large scale specimens were designed to experimentally evaluate the performance of the proposed connections under constant axial and reversed cyclic lateral loading. Two sets of specimens were designed and constructed; one set to evaluate the performance for loading in the transverse direction of the bridge, and one set to evaluate performance for loading in the longitudinal direction of the bridge. Specimen geometries are illustrated in Fig. 6 and Fig. 7 for loading in the transverse and longitudinal directions respectively, while specimen cross sections in the connection region are illustrated in Fig. 8.

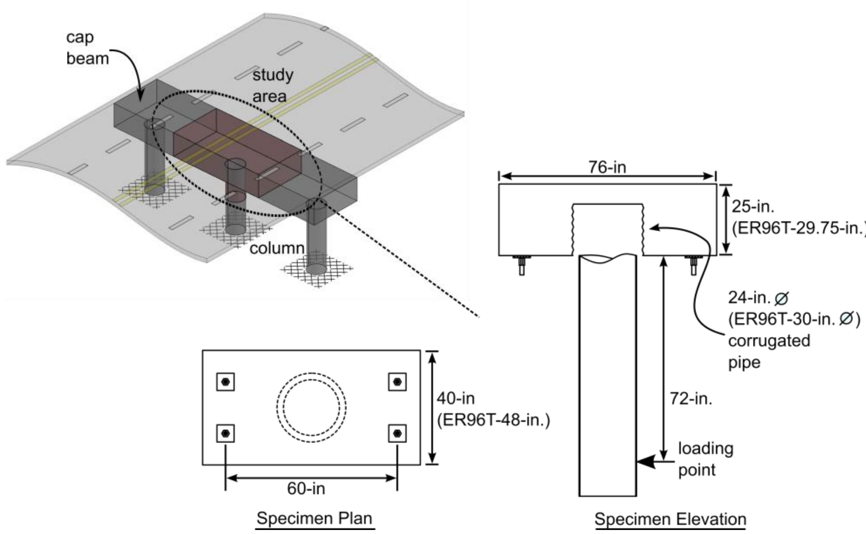

Fig. 6 Transverse study region and scaled specimen geometry.

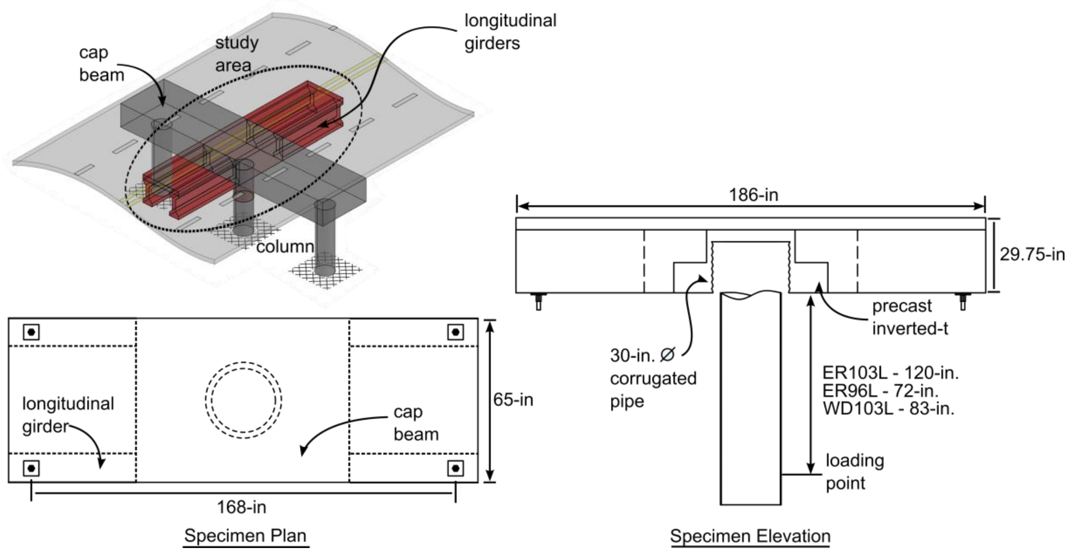

Fig. 7 Longitudinal study region and scaled specimen geometry

Four 20-in. diameter and one 24-in. diameter CFSTs were selected to evaluate the performance of the proposed connections for loading in the transverse direction (resulting in scale factors of 48% and 57%, respectively), while two 25.75-in. CFST and one 24-in. diameter CFST were selected to evaluate performance for loading in the longitudinal direction (resulting in scale factors of 61% and 57%, respectively). All tubes had a thickness of 0.25-in, resulting in D/t ratios of 80, 96, and 103 for the 20-in, 24-in, and 25.75-in. tubes, respectively. Specimen nomenclature used here refers to the connection type, as illustrated in Fig. 4, followed by the D/t ratio, and a letter to denote the direction of loading (T for transverse and L for longitudinal), i.e., ER96T describes an embedded connection with D/t = 96 for loading in the transverse direction of the bridge.

All of the specimens were constructed using pre-cast cap beams cast with a recess formed by light-gauge corrugated metal pipe, and the columns were grouted into place using high strength fiber reinforced grout. The specimens were cantilever columns anchored into a cap beam as illustrated in Fig. 6 and Fig. 7. Specimen cross sections in the connection region are illustrated in Fig. 8. Joint shear reinforcing in the welded dowel and reinforced concrete connection specimens was scaled from a prototype bridge and checked against the California Department of Transportation Seismic Design Criteria[17], while vertical shear reinforcing in the joint region of the cap beam for the ER connection was designed according to recommendations provided in reference material[4]. Flexural reinforcement in the cap beam was designed to resist 1.2 times the theoretical flexural strength of the CFST columns.

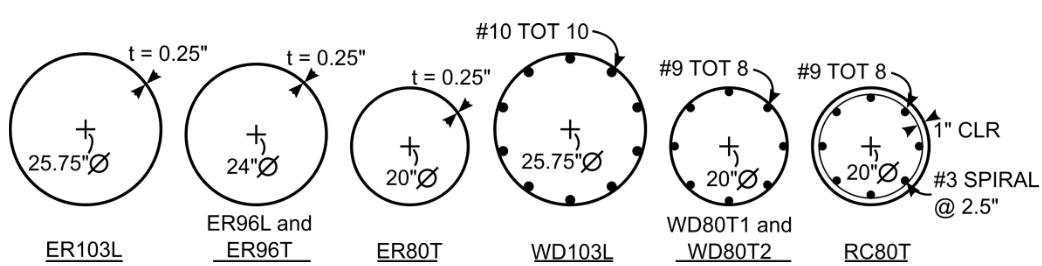

Fig. 8 Column cross sections in connection region

Embedded Ring (ER) Connection

Two transverse and two longitudinal specimens were designed to evaluate the performance of the proposed ER CFST column-to-cap beam connection. The specimens were designed: (1) to investigate the performance for smaller cap beam widths than had previously been evaluated for the embedded foundation connection, (2) to evaluate a smaller exterior annular ring projection of 8t (in contrast to 16t that had been used on the prior foundation connections studied), (3) to evaluate the influence of using API or ASTM grade tube steel, (4) to compare a straight seam and spirally welded tube, and (5) to evaluate the performance for loading in the transverse and longitudinal direction of the bridge.

Specimen ER80T was designed with embedment depth of 18-in. (0.9D), and utilized a 61-ksi ASTM A1018 spiral welded steel tube with an annular ring with a 2-in. projection both inside and outside of the tube. Specimens ER96T and ER96L were embedded 20-in (0.83D) into the cap beam (note the lesser relative embedment depth was possible because of the lower steel strength), and both utilized a 53-ksi API 5L X-42 grade straight seam tube with an annular ring that projected 51-mm (2-in.) inside and outside of the tube. Specimen ER103L was embedded 20.25-in. (0.8D) into the cap beam, and utilized a 69.3-ksi ASTM A1018 spiral welded steel tube with an annular ring with a projection of 2-in. inside and outside of the tube.

Welded Dowel (WD) Connection

Three specimens were designed using the welded dowel connection detail: one specimen with fully bonded bars (WD80T1), and two specimens with bars de-bonded along the length (WD80T2 and WD103L). In all cases, the longitudinal reinforcing in the connection region was selected with a target longitudinal reinforcing ratio of 3%, resulting in eight evenly distributed No. 9 bars in WD80T1 and WD80T2 and ten evenly distributed No. 11 bars in WD103L as illustrated in Fig. 8. The bars in all welded dowel specimens were embedded $12d_b$ into the cap beam per ACI 318 requirements for the development of headed reinforcing bars (ACI, 2011), and $24d_b$ into the CFST column. The bars were welded to the inside of the steel tubes using flare bevel groove welds formed by requirements of AWS D1.4 designed to exceed F_{ub}, where F_{ub} is the ultimate steel strength of the reinforcing bars. All of the specimens used flanges that projected 2-in. from the exterior of the steel tube and a 1-in. thick soffit fill, which extended below the surface of the cap beam. Specimen WD103L also included transverse No. 5 hoops with the intention of providing additional confinement to the soffit fill and joint region. PVC pipe was used to de-bond the longitudinal reinforcing bars in specimens WD80T2 and WD103L for lengths of 22-in. and 24-in., respectively. The de-bonded lengths were calculated using a moment-curvature analysis to achieve a connection rotation demand of 10% drift prior to fracture of the longitudinal reinforcing.

Reinforced Concrete (RC) Connection

One specimen (RC80T) was designed to evaluate the behavior of the reinforced concrete connection. As illustrated in Fig. 8, the longitudinal reinforcement consisted of eight evenly distributed No. 9 headed bars in an effort to achieve a longitudinal reinforcing ratio of 3%, and to allow for comparison to the welded dowel connections. The bars were embedded $12d_b$ into the cap beam per the ACI 318 development requirements for headed reinforcing, and $30d_b$ into the CFST column per development requirements for deformed bars[11]. Transverse column reinforcing was scaled from the prototype column, resulting in a No. 3 spiral at a spacing of 2.5-in. as shown in Fig. 8. A clear cover of 1-in. was provided between the steel tube and the transverse reinforcing.

EXPERIMENTS

The proposed connection types were experimentally evaluated in the structural testing lab at the University of Washington[16]. Specimen geometry was consistent with that defined in Fig. 6 and Fig. 7. The specimens were tested using a self-reacting test frame with a horizontal actuator to apply the lateral load and a Baldwin Universal Testing Machine (UTM) to apply a constant vertical lateral load as shown in Fig. 9.

The imposed displacement protocol was based on the ATC-24[14] protocol, and the specimens were subjected to 10% of the gross compressive load capacity of the CFST column. The specimens were instrumented using strain gages, linear potentiometers, string potentiometers, and an Optotrak motion capture system, however only the global moment-drift behavior measured using a load cell on the horizontal actuator and a string potentiometer placed at the center of loading is presented here. The location of this string potentiometer is indicated in Fig. 9. All specimens were tested in an inverted configuration due to constraints of the available testing apparatus.

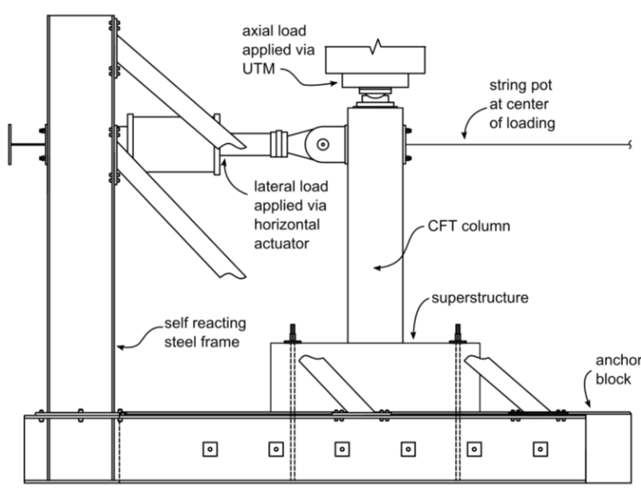

Fig. 9 Test apparatus

The moment drift behaviors of select specimens are plotted in Fig. 10, while the failure modes of select specimens are illustrated in Fig. 11. Only a brief description of the experimental behavior is presented here for brevity; more detailed descriptions are provided in reference material[16]. The moments have been normalized to the theoretical plastic moment capacity of the CFST calculated using the PSDM to allow for comparison of the specimens.

The ER connections exhibited larger stiffness and comparable strength to the WD connections. The larger stiffness of the embedded connection specimens is a result of the location of the tube as well as the confinement of the concrete fill. The comparable strengths are a result of the fact that the ER and WD connections had similar effective reinforcing ratios and moment arms. The RC connection developed significantly less resistance than the ER or WD connections due to a significantly smaller moment arm. The failure mode of all the ER connections was characterized by ductile tearing of the steel tube near the CFST column-to-cap beam interface as illustrated in Fig. 11a. These connections exhibited local buckling near this interface at drift ratios ranging from 3%-4%, however this did not influence the lateral load carrying capacity of the CFST column. In general, the welded dowel connections exhibited large ductility, however the failure modes of WD80T1 and WD80T2 was ultimately characterized by cap beam failure as illustrated in Fig. 11b. None of the bars in WD80T1 fractured during the experiment, and large cracking developed in the cap beam in drifts ranging from 7-8%. In contrast, one bar at the extreme fiber was fractured at 10% drift in WD80T2, and only moderate cap beam damage was observed for cycling up to this point. WD103L was cycled to 12% drift with no decrease in resistance or damage to the superstructure. The final state of specimen WD103L is illustrated in Fig. 11c. No bar buckling and only limited soffit crushing was observed in this specimen, as the transverse hoops in the joint region provided confinement (the location of the transverse hoops are indicated in Fig. 15). Note that transverse hoops were not included in the WD80T1 and WD80T2 specimens, and were included in WD103L based on the observed failure modes of the two WD80T specimens.. The failure mode of the RC connection was characterized by bar fracture and soffit crushing as illustrated in Fig. 11d, as six out of the eight connection bars fractured during cycling from 10% to 12% drift, and limited cap beam damage was observed.

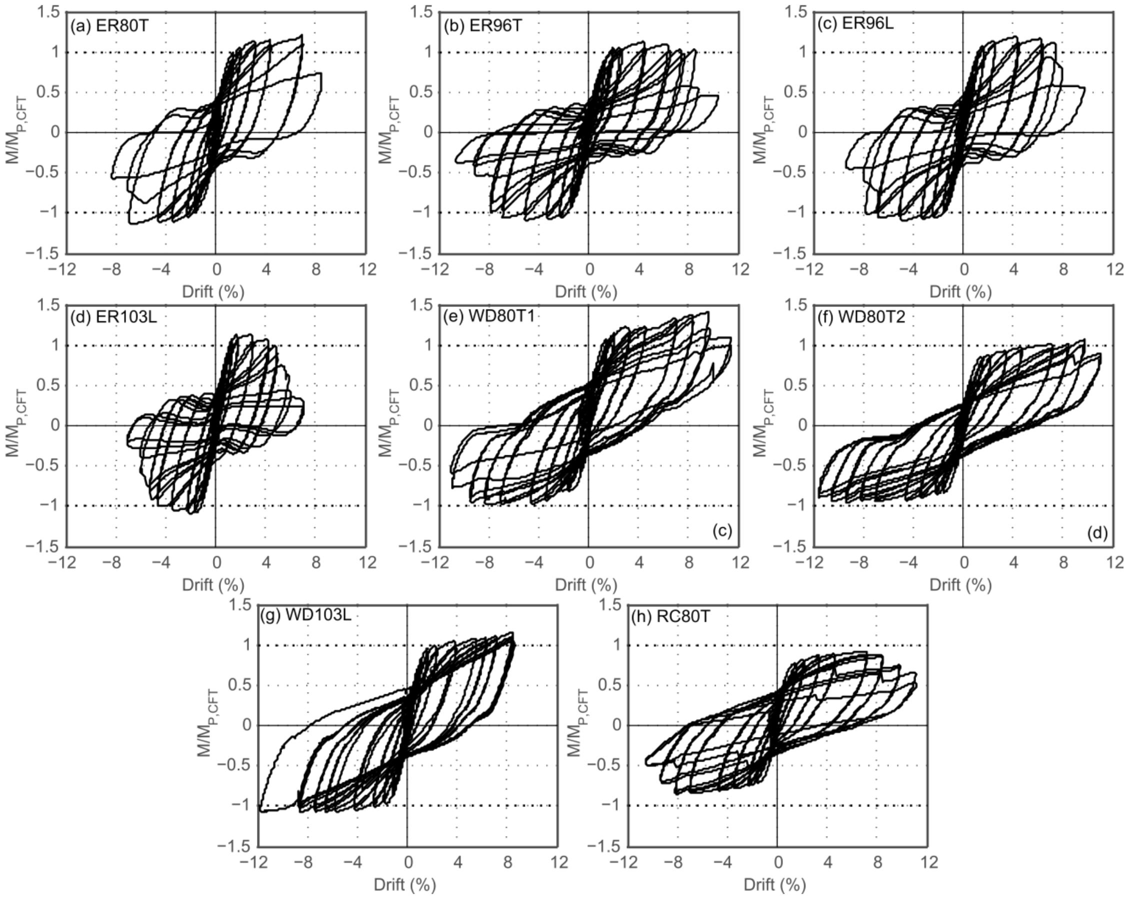

Fig. 10 Experimental moment-drift behaviors

(a) ER96T (b) WD80T1 (c) WD103L (d) RC80T

Fig. 11 Final state of select specimens

DESIGN EXPRESSIONS

The experimental results and observations were used to develop practical engineering expressions for the proposed CFST column-to-cap beam connections. The connection should be designed as one of the following options:

1. An embedded CFST connection (ER connection) in which the CFST column is embedded into the cap beam as illustrated in Fig. 4a and Fig. 12.
2. A welded dowel connection (WD connection) in which a ring of partially debonded vertical headed reinforcing bars are welded inside the CFST column and extend into the cap beam as illustrated in Fig. 4b and Fig. 15.

3. A grouted dowel connection (RC connection) in which a ring of headed reinforcing bars is developed into the steel tube and extend into the cap beam.

Each of these options can be employed using cast-in-place (CIP) or precast superstructure cap beam. For precast construction, a void must be included in the precast elements through use of a corrugated pipe, which meets the specifications outlined below. The following sections summarize design expressions for the ER and WD connection types. Design of the RC connection is very similar to that of a jacketed reinforced concrete column, and has thus been omitted here. Additional information on design can be found in reference material[16].

MATERIALS

Materials for the specified connections shall conform to the Caltrans standards[17], with several specific provisions included in this section.

Grout

When precast components are used, the fiber-reinforced grout consisting of prepackaged, cementitious grout and meeting ASTM C-1107 for grades A, B, and C non-shrink grout is used. The grout conforms to several additional performance requirements including compressive strength, compatibility, constructability, and durability. The 28-day grout strength f'_g must exceed f'_c of the surrounding concrete components. Grout using metallic formulations shall not be permitted, and grout shall be free of chlorides. No additives should be added to pre-packaged grout. These requirements ensure the grout has properties that provide adequate strength and longevity. These requirements adapted from recommendations provided in NCHRP Report 681[10].

Fiber Reinforcing

Macro polypropylene fiber with a minimum volume of 0.2% is included to provide crack resistance and bounding characteristics between the tube and corrugated metal duct. Test results to date have not evaluated the use of alternative fibers such as steel fibers.

Corrugated Metal Duct

Corrugated metal ducts are used to provide voids in precast components. The ducts are galvanized steel according to ASTM A653. Duct diameter is selected based on construction tolerances. Plastic ducts should not be used as the purpose of the ducts it to be a bond crack arrestor, act as confinement and provide shear transfer from the grout to the outer concrete. The use of corrugated metal ducts for grouted connections is supported by this research as well as a wealth of seismic precast connection data[4,10].

Reinforcement

Reinforcing in the connection region shall conform to ASTM A706 Gr. 60 (or Gr. 80 if allowed) requirements for weldable reinforcing. ACI and AASHTO are moving towards

Gr. 100 reinforcing steel, however steel strengths on this magnitude were not evaluated in this research and are thus not recommended at this time. ASTM A706 places restrictions on the chemical composition of reinforcing bars to enhance welding properties.

CFST Tube Steel

Steel tubes may either be straight seam or spiral welded and must conform to either ASTM 1018 or API 5L requirements. Spiral welded tubes must be welded using a double submerged arc welding process, and weld metal properties must match properties of the base metal and meet minimum toughness requirements of AISC demand critical welds[13]. Selection of tube material designation (ASTM 1018 or API 5L) plays a role in the ductility of the full strength embedded CFST connection. API 5L grade steel has more strict requirements regarding chemical composition than ASTM 1018 steel, and can therefore provide additional ductility for both spiral welded and straight seam tubes[16].

EMBEDDED RING CONNECTION

The embedded ring connection utilizes a CFST fully embedded into the cap beam. The CFST pier or column controls the strength and ductility of this connection type, not the cap beam or other superstructure components. The precast cap is placed on the column after the concrete fill is set, and the recess between the tube and corrugated pipe is filled with high strength fiber reinforced grout as shown in Fig. 12.

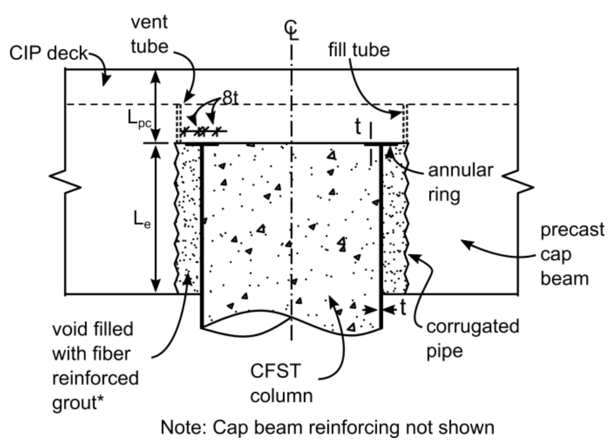

Fig. 12. Embedded Ring Connection

Annular Ring

The annular ring is welded to tube using complete joint penetration welds or fillet welds on both the inside and outside of the column designed to transfer the full strength of the tube to provide anchorage and stress transfer. The ring is made of steel of the same thickness and similar yield stress as the steel tube. The ring extends into and out from the tube 8 times the tube thickness to provide adequate anchorage as shown in Fig. 12.

Embedment Depth

The required embedment depth, L_e, of the CFST was determined using a conical pullout model discussed in detail in reference material[16]. The required embedment depth to eliminate the potential for cap beam failure is given in Equation 1 as:

$$L_e \geq \sqrt{\frac{D_o^2}{4} + \frac{DtF_{u,st}}{6\sqrt{f'_{c,cap}}}} - \frac{D_o}{2} \ (psi) \qquad [1]$$

where D_o is the outside diameter of the corrugated pipe, and D, t, and $F_{u,st}$ are the diameter, thickness, and ultimate stress of the steel tube, and $f'_{c,cap}$ is the compressive strength of the cap beam concrete in psi. The embedded depth, L_e, is illustrated in Fig. 12.

Punching Shear

Adequate concrete depth, L_{pc}, must be provided above the tube to eliminate the potential for punching shear failure in the cap beam as shown in Fig. 12. The ACI 318[11] provisions for footings in single shear were used as a basis to develop an expression for the minimum depth above the embedded CFST to avoid this failure mode. This expression is given in Equation 2 as:

$$L_{pc} \geq \sqrt{\frac{D_o^2}{4} + \frac{C_c+C_s}{6\sqrt{f'_{c,cap}}}} - \frac{D}{2} - L_e \ (psi) \qquad [2]$$

where C_c and C_s are the compressive forces in the concrete and steel due to the combined axial load and bending moment as computed by the PSDM. Note that the derivation of this expression can be found in references[4,16].

Cap Beam Flexural Reinforcing

Longitudinal flexural reinforcing in the column region is required to resist $1.25M_{p,CFST}$ to ensure the cap beam does not yield. Longitudinal flexural reinforcing is spaced uniformly across the width of the cap beam. To ensure continuity, a minimum of one layer of upper reinforcing must pass above the embedded CFST in the cap beam as illustrated in Fig. 13. Some longitudinal reinforcing in the bottom layer will be interrupted by the embedded corrugated pipe. The bottom layer of flexural reinforcing not interrupted by the corrugate pipe shall be designed to resist $1.25M_p$ of the CFST column. Interrupted bars should still be included as shown in Fig. 13.

Joint Region Shear Reinforcing

Vertical reinforcing, A_s^{jv}, shall be included in the joint region according to Equation 3, where A_{st} is the total area of the steel tube embedded into the cap beam, and A_s^{jv} is the total area of vertical reinforcing required within a distance L_e from the outer diameter of the corrugated pipe when a precast cap beam is used. Derivation of this equation is given in reference material[16].

$$A_s^{jv} = 0.65 A_{st} \quad [3]$$

Vertical stirrups or ties are distributed uniformly within a distance $D/2+L_E$ extending from the column centerline as shown in Fig. 13 and Fig. 14a. These stirrups can be used to meet other requirements documented elsewhere including shear in the bent cap.

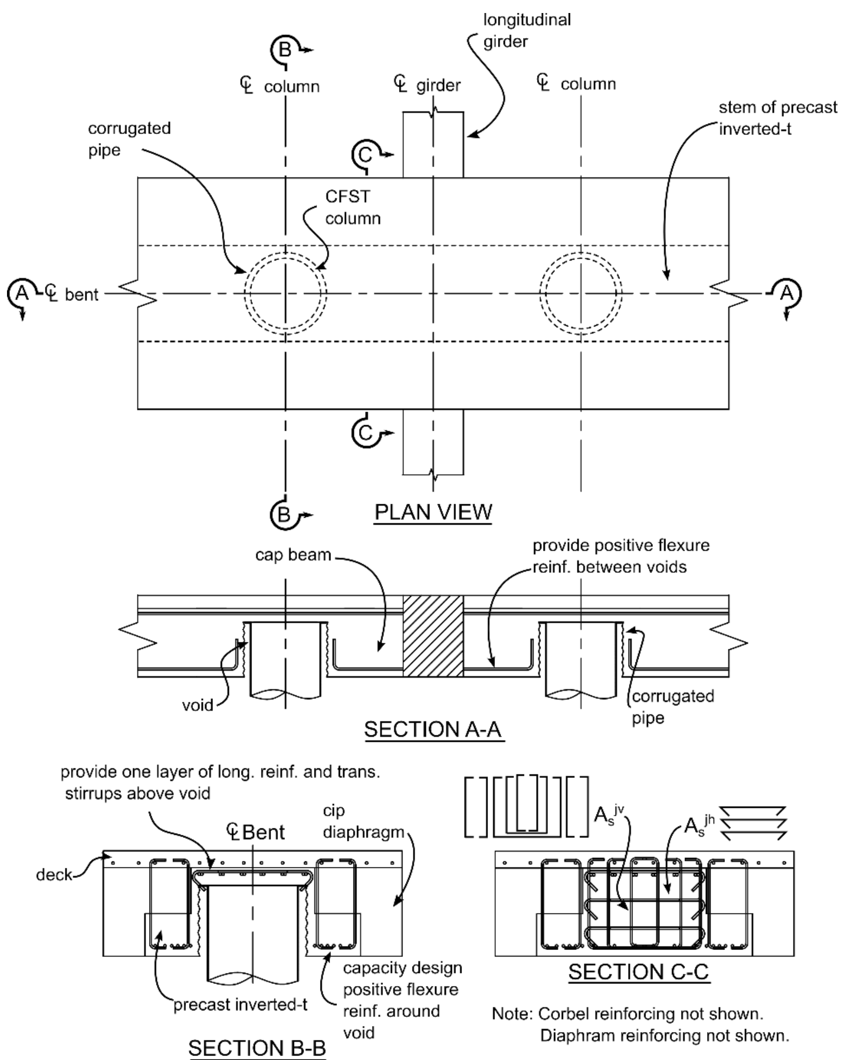

Fig. 13 Cap beam details for embedded connection

Joint Region Horizontal Stirrups

Horizontal stirrups or ties shall be placed transversely around the vertical stirrups or ties in two or more intermediate layers spaced vertically at not more than 18-in apart. The horizontal reinforcing area, A_s^{jh}, is determined using Equation 4 where A_{st} is the area of the steel tube embedded into the cap beam. The horizontal reinforcing shall be placed within a distance $D/2+L_E$ extending from the column centerline as illustrated in Fig. 14b.

$$A_S^{jh} = 0.1 \times A_{st} \quad [4]$$

In addition, the top layer of transverse reinforcing should continue across top of the void in the cap beam as shown in Fig. 14b.

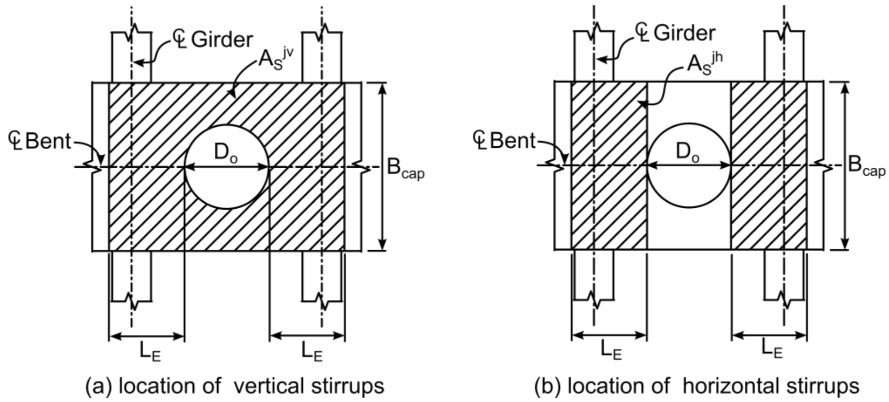

Fig. 14. Required location of (a) vertical and (b) horizontal stirrups for the embedded ring connection

WELDED DOWEL CONNECTION

The welded dowel connection utilizes a ring of headed reinforcing bars that are welded into the tube and developed into the cap beam. The strength is controlled by the reinforcing ratio of the longitudinal reinforcing which extends from the column into the cap beam. The welded detail is designed to carry the full strength of the reinforcing bar. The advantage of this connection is a shorter embedment length into the CFST column and a maximized moment arm. Design of this connection shall conform to requirements in the Caltrans standards[17], with several specific provisions included below.

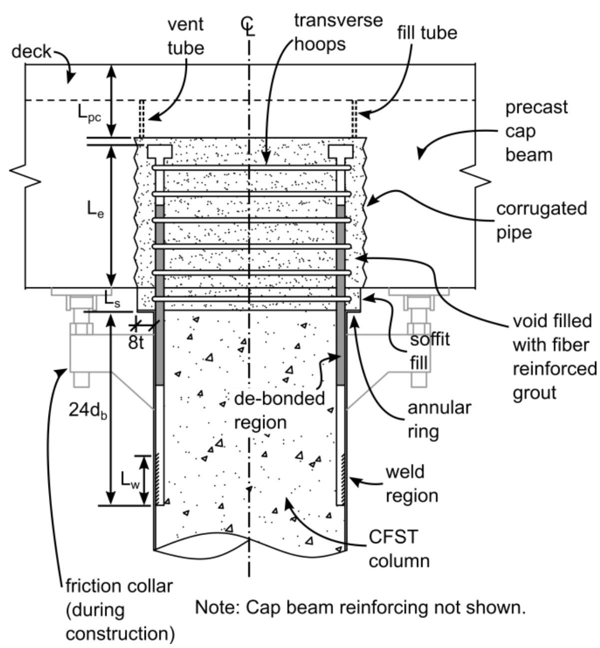

Fig. 15. Welded dowel connection

Annular Ring

The annular ring is welded to end of the steel tube to provide a larger area to transfer compressive stress from the steel tube into the soffit fill. In this connection the ring does not transfer tensile stresses but does provide some compressive force transfer. The ring is made from steel of the same thickness and yield strength as the steel tube. The ring projects outside of the steel tube a distance 8 times the thickness of the steel tube as illustrated in Fig. 15.

Length Dowels Extend into the Cap Beam and Column

The headed reinforcing extends into the cap beam to fully develop the longitudinal dowels while also eliminating the potential for a conical pullout failure. The headed dowels must extend into the cap beam for the largest length calculated using Equation 5 and Equation 6. Equation 5 defines the required development length to develop reinforcing bars with mechanical anchors. Note that ACI is referenced because neither AASHTO[12] nor the Caltrans SDC[17] provide development expressions for headed bars. Equation 6 defines the required embedment length to eliminate a conical pullout failure similar to the tube embedment depth requirement defined in Equation 1. The derivation of Equation 6 is provided in reference material[16].

$$L_e \geq \frac{0.016\psi_e F_{y,b}}{\sqrt{f'_g}} d_b \qquad [5]$$

$$L_e \geq \sqrt{\frac{D^2}{4} + \frac{1.2 * F_{y,b} * A_{st,b}}{6\pi\sqrt{f'_{cc}}}} - \frac{D}{2} \qquad [6]$$

The longitudinal dowels must extend into the CFST for a distance adequate to develop the full strength of the dowels while limiting damage to the concrete fill. Results from welded dowel pullout tests (discussed in references[16]) suggest that the embedment can be as low as $18d_b$ for full dowel development, however a distance of $24d_b$ is recommended here to provide a reasonable factor of safety.

Vertical and Horizontal Joint Region Reinforcing

Cap beam detailing requirements specified in the California Department of Transportation Seismic Design Criteria V. 1.6[17] should be followed when designing the welded dowel connection.

Soffit Fill Depth

The soffit fill depth, L_s, is calculated according to Equation 7 to ensure that the annular ring does not come in to contact with the bottom of the cap beam at the maximum expected drift angle, θ_u where D is the outer diameter of the annular ring. This depth is illustrated in Fig. 15.

$$L_s \geq \sin(\theta_u)\left(\frac{D}{2} + 8t\right) \qquad [7]$$

Dowel De-bonded Length

Longitudinal dowels should be de-bonded from the concrete in the connection region with the intent of increasing connection ductility. The required de-bonded length to achieve a pre-determined connection rotation, θ_u, prior to bar fracture is calculated using Equation 8 or 9, where ϕ_u is a curvature limit corresponding to a maximum steel strain as obtained from a moment-curvature analysis. Half of the de-bonded length extends into the cap beam, and half of the de-bonded length extends into the CFST column as illustrated in Fig. 16.

$$L_{ub} = \frac{\theta_u}{\phi_u} \qquad [8]$$

$$L_{ub} = \frac{\tan\theta(D - t - d_b/2)}{0.7\varepsilon_u} \qquad [9]$$

Equation 9 is a simplified method for estimating the required de-bonded length of the longitudinal reinforcing to achieve a pre-determined drift ratio prior to bar fracture. Although this method does not require a moment curvature analysis, it results in larger de-bonded lengths than those calculated using a moment-curvature analysis, as required in Equation 8.

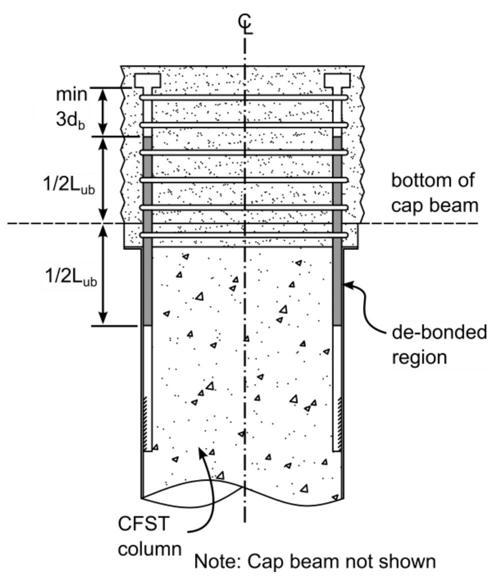

Fig. 16. Welded dowel connection de-bonding dimensions

Dowel-to-Steel Tube Welds

Longitudinal dowels are welded to the inside of the steel tube using flare bevel groove welds on both sides of the dowels, as illustrated in Fig. 5. The required weld lengths to develop the rupture capacity of the longitudinal dowels are specified in Equation 10 and are based on typical weld limit states for flare bevel groove welds where A_b is the area of the longitudinal dowel, $F_{y,b}$ is the yield strength of the longitudinal dowel, and F_{EXX} is the tensile strength of the weld metal. Equation 10a is based on failure of the weld metal, Equation 10b is based on yielding of the tube steel, and Equation 10c is based on rupture of the tube steel. A strength reduction factor of 0.9 has been included for yielding limit states in Equations 10a and 10b, while a strength reduction factor of 0.75 has been included based on a tube steel rupture limit state in Equation 10c.

$$L_w \geq \frac{5.6 A_b F_{y,b}}{F_{EXX} d_b} \tag{10a}$$

$$L_w \geq \frac{0.83 A_b F_{y,b}}{F_{y,st} t} \tag{10b}$$

$$L_w \geq \frac{1.11 A_b F_{y,b}}{F_{u,st} t} \tag{10c}$$

Use of Spiral or Hoop Reinforcement in the Joint Region

Transverse reinforcing in the form of spiral or individual hoops should be included around the longitudinal dowels which extend into the cap beam according to requirements in the California Department of Transportation Seismic Design Criteria V. 1.6. At least one hoop should be placed in the soffit fill depth if individual hoops are used as shown in Fig. 15. This reinforcing acts to confine the grout in the joint region and limit buckling of the longitudinal dowels[16].

CONCLUSIONS AND FUTURE WORK

An embedded CFST column-to-foundation connection in which the steel tube is grouted into the foundation concrete was briefly introduced. Experimental research showed that this connection is capable of transferring the plastic moment capacity of the CFST column while limiting damage to the foundation. The experiments provided valuable information and resulted in straight forward and practical design expressions for an embedded foundation connection.

Several new CFST column-to-cap beam connections were proposed and experimentally studied using increasing cyclic deformations. These connections included (1) an embedded ring connection in which an annular ring is welded to the top of the steel tube and embedded into the cap beam (2) a welded dowel connection in which a ring of headed dowels is welded to the inside of the steel tube and developed into the cap beam, and (3) a reinforced concrete connection in which a traditional reinforcing cage consisting of a ring of headed dowels with transverse reinforcing is developed into the CFST column and cap beam. All of the connections were demonstrated using a grouted connection detail, which can be integrated with precast cap beam components for ABC. A series of large scale specimens were tested to evaluate the behavior of the different connection types. The experimental results suggest that all of the connection types can achieve strength and ductility objectives within the unique constraints of a precast cap beam, and practical engineering expressions were developed for the proposed connections.

Additional research should be conducted to further refine the design expressions, and evaluate the behavior of the connections for a much wider range of parameters. An evaluation of possible repair methods would be beneficial to demonstrate the advantages of using CFSTs in highway bridge construction. Thus four primary areas of future work are recommended:

1) Utilize the detailed finite element models developed for the initial connection evaluation to conduct extensive parametric studies on the proposed connections.
2) Evaluate repair strategies for columns which have been moderately damaged following lateral load events.
3) Develop additional connections such that CFSTs are more versatile for bridge construction; specifically a CFST-to-pile connection is needed.
4) CFSTs should have larger torsional strength and deformability relative to RC columns. A research program aimed at evaluating the response of CFST columns and connections subjected to combined torsional, shear, flexure and axial loading is needed. In addition this program should develop a connection capable of transferring torsion to the superstructure for skewed bridges.

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