Composite Action in CFT Components and Connections

CHARLES W. ROEDER, DAWN E. LEHMAN and RYAN THODY

ABSTRACT

Concrete-filled steel tube (CFT) components provide an attractive alternative to conventional methods of construction. The steel tube serves as formwork and reinforcement to the concrete fill. The fill delays local buckling of the tube and increases the compressive strength, stiffness and ductility of the CFT member if composite action is achieved. As such, the benefits of using CFT components include their efficiency, economy and ability to provide rapid construction. The primary obstacles to using CFT components relate to achieving adequate shear-stress transfer between the two materials needed for composite behavior and to developing robust, constructible connections. There is considerable interaction between this stress transfer and the connections joining CFT members to other structural elements. In many cases, natural bond stress, primarily friction, is adequate to ensure good structural performance of the CFT member, but in other cases mechanical interlock or bearing between the steel and concrete is required. While some mechanical transfer elements may supplement the natural bond transfer, others may destroy this beneficial effect. Additionally, natural bond may be dramatically enhanced by bending of the CFT member and by using appropriate connections to join other members to the CFT element. Therefore, proper quantification and understanding of natural, enhanced and mechanical sources of bond stress are needed to develop robust CFT structural components and their connections.

Keywords: bond stress, columns, composite construction, concrete-filled steel tubes, hollow structural sections, tubes.

INTRODUCTION

Concrete-filled steel tubes (CFT) have been used to support large axial forces induced by gravity or lateral loads. CFT construction is economical because the steel tube serves as a formwork, and it reinforces the concrete fill at the optimal location. The concrete fill stiffens the steel tube and restrains or delays local buckling. The diameter of the tube is sometimes very large, and diameter-to-thickness (d/t) ratios for the tube have sometimes exceeded 100. CFT construction has been used on buildings, such as the Gateway Tower in Seattle shown in Figure 1a, and bridge piers, shown in Figure 1b.

CFT offers great potential as an economical and practical system for providing seismic resistance and rapid construction for a wide range of structural systems. However, there are limitations in the use of CFT because of (1) uncertainty in composite action or interaction between the steel tube and the concrete fill and (2) inherent difficulties in connecting CFT members to other structural elements. The full benefits of CFT construction are achieved only if the steel and concrete work together to develop composite action. Stress transfer between the steel and concrete is necessary to achieve composite behavior, and the alternate methods of achieving this stress transfer and resulting composite action are evaluated. While the issues of composite action and connection of other members to CFT columns appear to be fundamentally different concerns, this paper will show considerable interaction between them and addresses both.

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NATURAL BOND STRESS

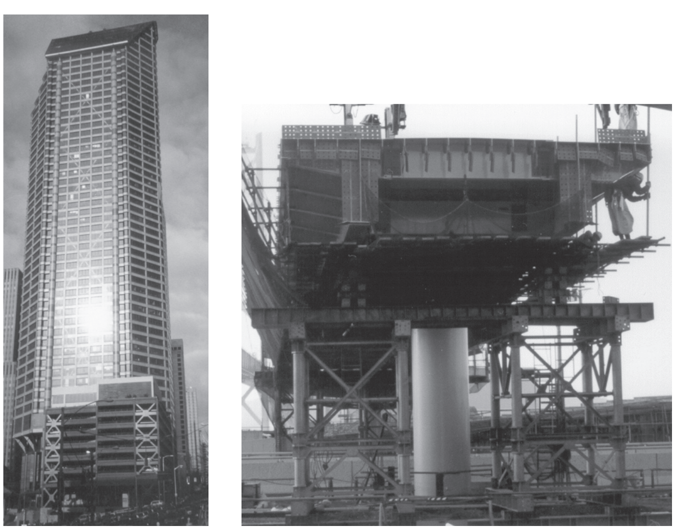
Achieving composite action requires sufficient shear strength at the interface between steel and concrete, which is termed bond strength. There are two general mechanisms used to achieve this strength: (1) natural bond, which includes chemical adhesion and frictional resistance, and (2) mechanical bond, which typically requires supplemental devices or significant irregularities in the steel to permit bearing of steel on concrete. If possible, relying on the natural bond stress between the steel and concrete, rather than mechanical bond-enhancing mechanisms, facilitates construction of CFT elements, since it results in less interference during concrete placement.

A number of research studies have examined the response of CFT sections that rely solely on natural bond stress (Roeder, Cameron and Brown, 1998; Virdi and Dowling, 1975; Furlong, 1967, 1968; Shakir-Kalil, 1991, 1993a, 1993b; Morishita, Tomii, and Yosimura, 1979a, 1979b; Morishita and Tomii, 1982; Tomii, 1984). These past studies have used push-out tests (typical setup shown in Figure 2) to evaluate bond-stress capacities. These past studies show that the bond stress between the steel tube and the concrete fill exhibits multiple stages, but it is primarily a frictional resistance dependent on the surface roughness of the steel and the contact pressure between the steel and concrete. Secondary benefits are initially provided from adhesion or chemical bonding between the two materials, but these secondary benefits are typically overcome during lower levels of loading.

Most past research expressed push-out test results in terms of an average bond stress over the entire bond area and specimen length. These tests have studied the impact of several variables on the average bond strength, including the strength of the concrete, the shape of the tube (square or circular), diameter and slenderness of the tube, the length of the bonded section, roughness of the interface, and the test setup. Bond stress is inherently variable, and there is great value in understanding why this variability occurs.

The degree of variability in natural bond stress is largely insensitive to the concrete strength, as shown in Figure 3a, since the primary bond stress resistance mechanism is friction, and the coefficient of friction is insensitive to strength. Some variability must be expected with respect to the length of the specimen, because of the nearly exponential bond stress variation over length computed in finite elment analysis prior to slip shown in Figure 4a. After progression of slip, the bond stress distribution approaches a uniform distribution over the slipped length, and the force transfer between steel and concrete approaches linearity as shown in Figure 4b. This behavior is verified by ANSYS finite element computer analysis (Santos, 1997) and experimental results (Roeder et al., 1998). The average stress computed from push-out tests is lower than the peak bond stress prior to slip but is larger than the bond stress over the majority of the specimen length as shown in Figure 4a. As seen in Figure 4a, the peak bond stress is quite large in a local area, and the distribution of bond stress is approximately exponential prior to initiation of slip. At increased load, local slip initiates when the static-friction capacity is exceeded. The extent of slip is limited by the remaining unslipped region. The magnitude of the maximum average bond stress depends on the coefficient of friction, the contact surface length, and the contact stress; however, it must be less than the peak local bond stress value prior to slip.

Natural bond stress is significantly larger for circular CFT than for rectangular or square CFT as shown by comparison of measured results in Figure 3a. As a result, circular CFT more readily develops its full composite resistance than does rectangular CFT. Although the geometry and material properties influence the peak and average bond strength for axially loaded specimens, the most influential parameter is the interface condition of the tube, as illustrated in Figure 3b. In that figure, only data from test series in which the interface condition was intentionally varied are plotted (Virdi and Dowling, 1975, Shakir-Khalil, 1993). The interface conditions were differentiated using a numerical scale from 1 (machined) to 4 (intentionally roughened). Comparing these data and the data on Figure 3a, there are several points of interest:



(a) (b)

Fig. 1. Examples of CFT applications; (a) highrise building (Gateway Tower, Seattle), (b) bridge pier.

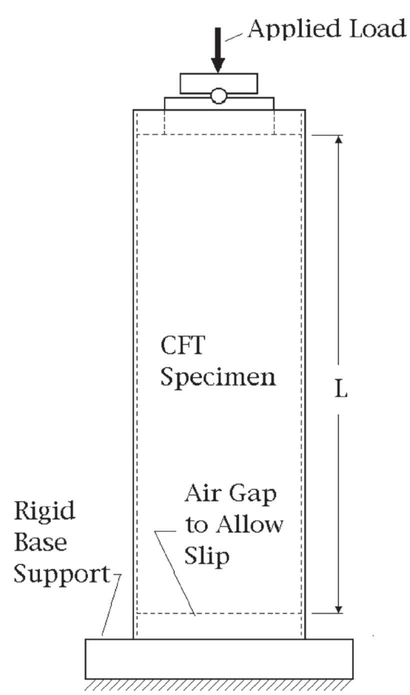


Fig. 2. Push-out test (Roeder et al., 2001).

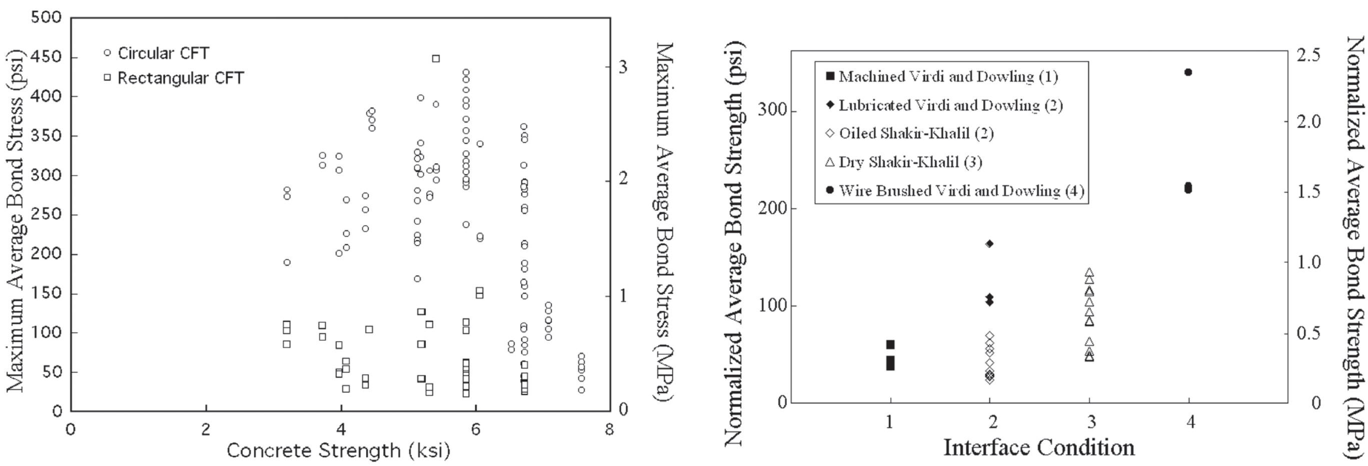
(1) The range of the average bond stresses is similar in Figures 3a and 3b, although Figure 3b only plots a subset of the data; (2) the data plotted in Figure 3b show that the minimum and maximum average bond stress values are approximately 75 to 350 psi (0.5 to 2.5 MPa) for circular CFT and the variability depends more heavily on the interface condition (i.e., the lowest bond stress is computed for the machined specimens) than the researcher, test setup or specimen length. Rectangular CFT has bond stress values one-half to one-third the values obtained with circular CFT.

Similarly, the average bond stress depends on the degree of the concrete shrinkage and vibration of the concrete (Roeder et al., 1998; Han and Yang, 2001). The bond transfer between the steel tube and the concrete fill depends on the radial displacements due to the pressure of the wet concrete on the shell and the shrinkage of the concrete core, together with the internal surface irregularities of the inside of the tube. CFT columns have axial symmetry, and the pressure of the wet concrete will cause a radial enlargement of the tube, as approximated by Equation 1

$$\Delta_1 = (pd^2)/(4E_s t) \tag{1}$$

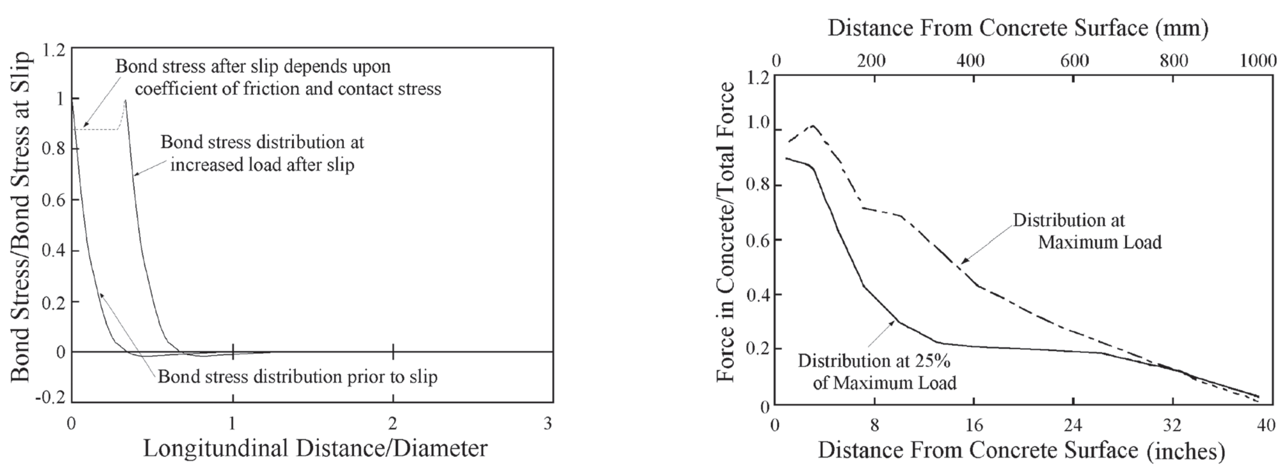
where

Δ_1 = radial enlargement
p = internal pressure of wet concrete
d = diameter of the steel tube
t = wall thickness of the steel tube
E_s = modulus of the steel



(a) *(b)*

Fig. 3. Measured average bond stress capacity: (a) as function of concrete strength (Roeder et al. 1998); (b) as function of interface condition.



(a) *(b)*

Fig. 4. Measured bond stress distribution at different load levels.

Shrinkage of the concrete during curing produces a radial reduction of the concrete core of

$$\Delta_2 = (-cd)/2 \qquad (2)$$

where

Δ_2 = radial reduction of concrete core due to shrinkage
c = shrinkage of a unit volume of concrete

The shrinkage depends on the concrete mix and admixtures, the curing procedures and the diameter. When these deflections are considered, three possible states exist at the interface:

State A: $$\Delta_1 + \Delta_2 > 0 \qquad (3)$$

State B: $$\Delta_1 + \Delta_2 < -\Delta_3 \qquad (4)$$

State C: $$0 \geq \Delta_1 + \Delta_2 \geq -\Delta_3 \qquad (5)$$

where

Δ_3 = amplitude of the rugosity, or roughness, of the interior of the tube

This evaluation is approximate, but it clearly illustrates the issues at hand.

In State A the concrete exerts pressure on the interface after the shrinkage is complete, and the initial bond strength is provided by an adhesion between the steel and the concrete combined with friction due to pressure at the interface. With increasing shear, the initial adhesion (or chemical bond) capacity is exceeded and local slip occurs. This slip deformation may be very small, but friction is the shear transfer mechanism after this occurs. In State B separation between the two materials exists after shrinkage and relative rigid body motion occurs with little bond strength or resistance when one material is pushed relative to the other. In this state, bond stress is developed only when stresses and strains caused by applied loading induce frictional contact at the surface. The effective bond stress in Stage B may be negligible. State C is an intermediate condition. Adhesion is of reducing significance and the mechanical bond or interlock of the surfaces progressively reduces in a relatively unpredictable manner as State B is approached.

The surface roughness or rugosity of steel is typically small, and as a result, the natural bond stress achievable in most practical applications of CFT is small. However, some manufacturing methods such as spiral welding of the tube may increase the roughness or irregularity on the inside of the tube, and thus increase the natural bond stress achieved with the CFT application. This natural bond stress also may be increased if expansive or low shrinkage concrete is employed, and it is significantly larger with circular tubes than square or rectangular tubes (see Figure 3a). This natural bond stress decreases as the diameter or *d*/*t* ratio of the tube increases. Statistic models of bond stress have been developed for straight seam circular tubes, and the (statistically reliable) predicted average bond stress is never greater than approximately 200 psi (1.5 MPa), and it becomes negligible as the *d*/*t* ratio exceeds 50 to 60 or for large diameter tubes (Roeder et al., 1998).

COMBINED BENDING AND AXIAL DEMANDS — AN EXCEPTION TO THE NATURAL BOND STRESS LIMITATION

Recent tests have shown that there is a clear exception to the natural bond limitation for components subjected to large bending demands in addition to axial stresses. Tests by Thody (2006) were conducted to study the impact of the surface condition of the steel on the stiffness, strength and deformability of flexural CFT elements. Figure 5a illustrates a test setup where 20-in.-diameter (508 mm) spiral-weld tubes with large *d*/*t* ratio of 80 and high-strength vanadium-alloy steel ($F_y \approx$ 525 MPa or 76 ksi) were subjected to cyclic bending moment demands. In the test program, a series of eight tubes were tested, with four specimens designed specifically to evaluate the influence of the surface and slip condition on bond stress, shear transfer and composite behavior. [The other four specimens studied the effect of variation in the properties of the spirally welds and galvanization on the engineering response of the specimens and are described elsewhere (Thody, 2006)].

The steel tubes were spirally welded with the double submerged arc process from both the outside and inside of the tube. The specimens were filled with a low-shrinkage, self-consolidating concrete with a specified compressive strength of 13 ksi (90 MPa). The steel tubes were approximately 20 ft. (6.1 m) long, and the concrete was placed without vibration. The specimens were subjected to 3-point loading with a span length of 18 ft (5.48 m) and were not subjected to an axial load.

The reference specimen, CFT1, was constructed with no special surface treatment (without capped ends or grease interior) to represent a "normal" bond condition. Past test data suggest that the bond stress under "normal" conditions should be very small because of the large diameter and *d*/*t* ratio of the tube (Roeder et al., 1998). Specimen CFT2 was not filled with concrete but instead was a hollow steel tube (CFT2) to provide a baseline control specimen for demonstrating composite behavior. Specimen CFT3 had capped ends to ensure that slip between the steel and concrete could not occur. Specimen CFT4 was thoroughly greased over the entire interior surface of the tube just prior to concrete placement to minimize the potential friction and natural bond stress at the interface. Table 1 provides a summary of the test specimens and general test results. Shear connections or mechanical devices were not employed in any of these tests, and the concrete fill had no reinforcement other than the steel tube. The specimens were subjected to cyclic inelastic

Table 1. Summary of Flexural Tests

Specimen	Steel-Concrete Interface or End Condition	Ultimate Failure Mode	Maximum Moment Resistance (kN-m)	Stiffness Ratio $(E_sI_{eff})_{0.9M_y}$ to E_sI_g	Drift Level (%) at Various Benchmarks: Initial Yield	Local Buckling	Initial Tearing	At 20% Reduction in Resistance: + Drift %	– Drift %
CFT1	Normal	Tearing at Buckle	1155	0.52	0.9%	1.7%	2.7%	3.1%	–3.1%
CFT2	Hollow	Tearing at Buckle	816	0.39	0.8%	1.2%	N/A	1.5%	–2.1%
CFT3	Capped Ends	Tearing at Buckle	1182	0.51	1.0%	1.9%	3.1%	3.1%	–3.2%
CFT4	Greased	Tearing at Buckle	1150	0.53	0.7%	1.9%	2.6%	2.9%	–2.9%

deformation histories based upon the ATC-24 test protocol (ATC, 1992).

Table 1 shows that the specimens with concrete fill provided significantly greater ductility, resistance and stiffness than the hollow steel tube. Figure 6 show the measured moment-drift plots for the specimens CFT3 and CFT4, since they represent the extremes expected in composite behavior for these tests. Specimen CFT3 should have ideal bond stress since it has capped ends and full restraint against slip, while Specimen CFT4 should have the least natural bond and composite action with the greased steel-concrete interface. Comparison of these figures and the data of Table 1 indicate that the greased specimen developed nearly the same total composite action in that it had similar maximum resistance, stiffness and inelastic deformation capacity values as the specimen with capped ends. The filled specimens all developed the predicated ultimate capacity of the composite section. This is an important observation because it indicates that the binding action associated with bending is sufficient to prevent relative slip of the concrete and steel thereby ensuring composite action was achieved.

The composite specimens exhibited similar behaviors. Cracking of the concrete was localized to within a narrow region subject to maximum bending moment and occurred at relatively small inelastic deformations (always less than 1% drift). Yielding of the steel tube was typically noted at drift levels in the range 0.7% to 1.0%, and visible local buckling of the tube was observed at drift levels in the range of 1.7% to

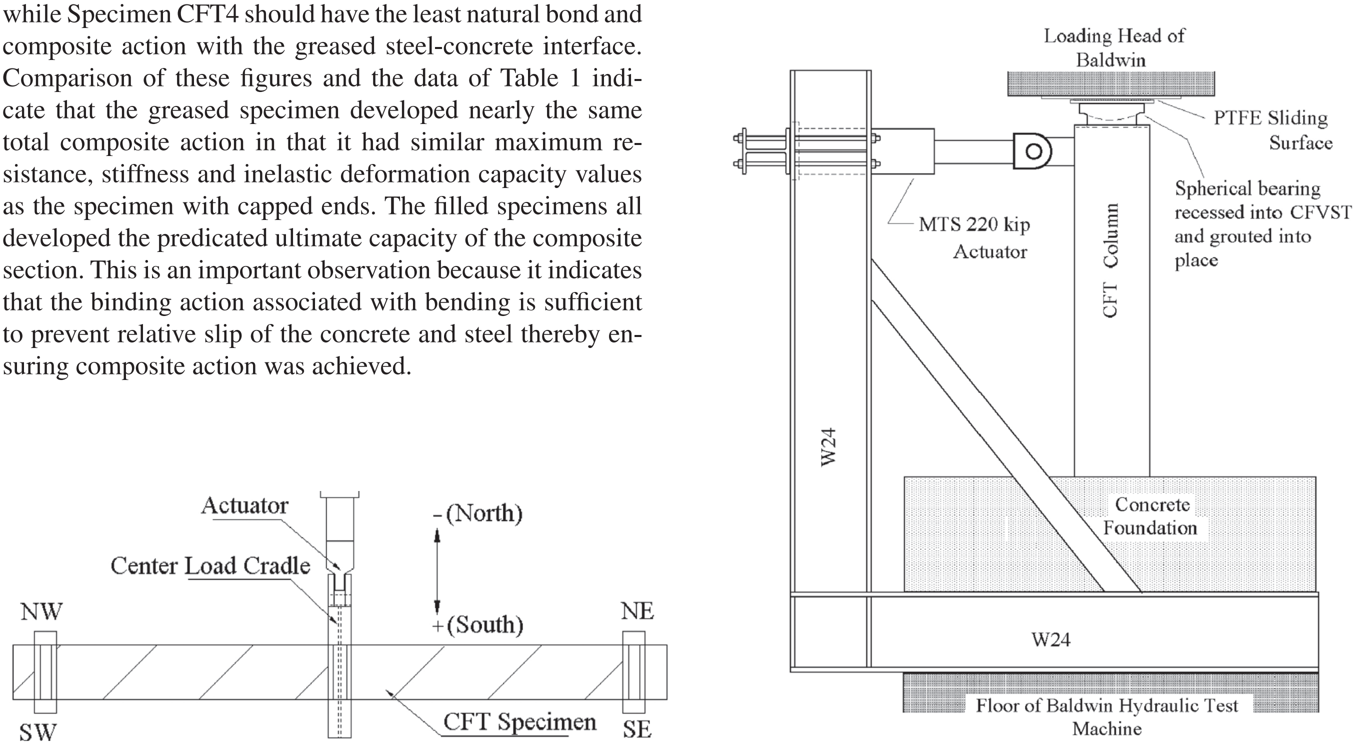
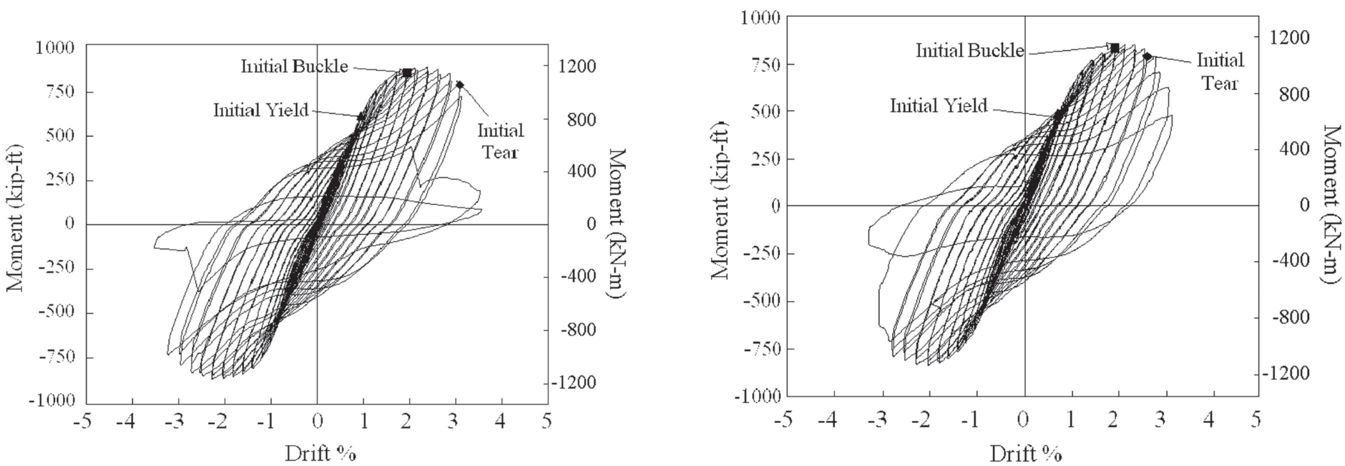


Fig. 5. Schematics of tests setups: (a) cyclic flexural test; (b) cyclic flexure with axial load.

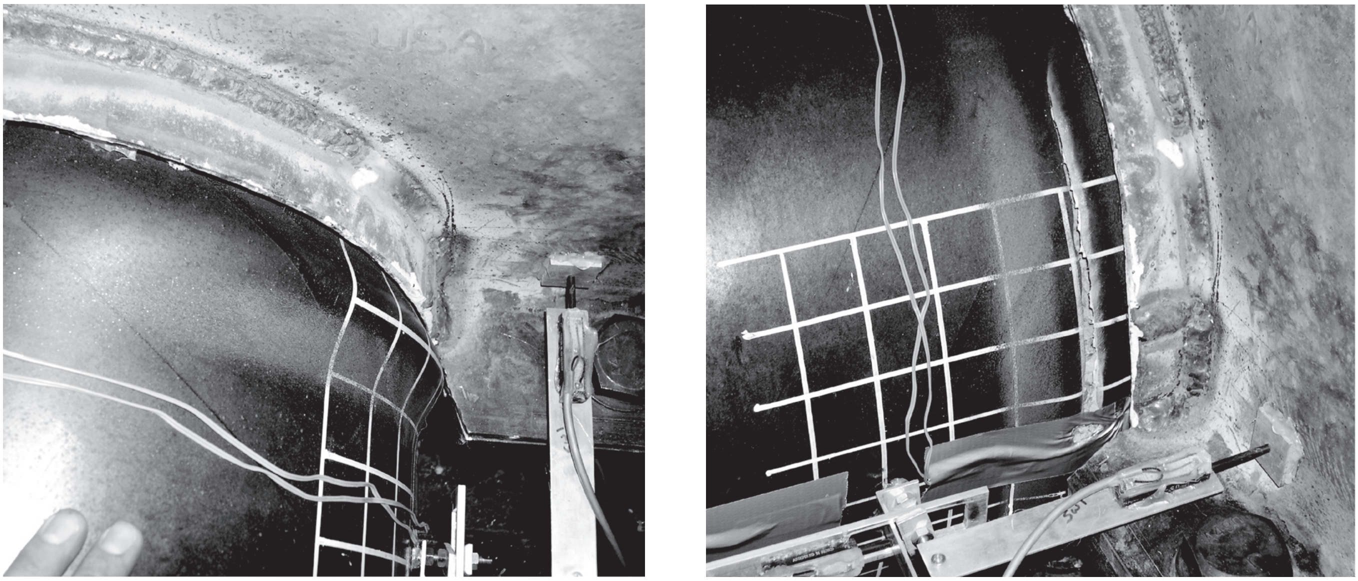
2.0%. The local displacement of the buckled region became increasingly larger with increased inelastic cycles, as shown in Figure 7a. As a larger buckling bulge occurred, large local strains were noted on the peak of the bulge and cracks began to develop in the steel at the peak of the buckling bulge at drifts in the range of 2.6% to 3.1% (Figure 7b). The cracks grew slowly in a ductile manner with increasing deformations, and powdered concrete seeped from the cracks during this time. This indicates that the concrete was completely crushed into a powdered form before severe buckling and steel cracking initiated. The crack length grew with increasing deformation, and ductile tearing progressed around the perimeter of the tube occurred at drift ratios between 3% and 4%. The ductility exhibited in these tests was considerable when it is recognized that the *d*/*t* ratio greatly exceeds that allowed by current AISC *Seismic Provisions for Structural Steel Buildings* and AISC *Specification for Structural Steel Buildings* (AISC, 2005a, 2005b).

Slip measurements were made between the steel and the concrete fill at the ends of the test specimens. The maximum slip data values measured for CFT1 and CFT4 were 0.07 and 0.11 in. (1.9 and 2.8 mm), respectively. After completion of the test, destructive inspection of the specimen revealed that the primary cracking and concrete crushing occurred within



(a) *(b)*

Fig. 6. Moment-drift curves: (a) Specimen CFT3 with capped ends; (b) specimen CFT4 with greased interface.



(a) *(b)*

Fig. 7. Photos of behaviors noted in the CFT tests: (a) local buckling; (b) tears developing at the peak of the bulge.

approximately 4 in. (100 mm) of the central load point, as illustrated by the photo in Figure 8. The concrete fill was examined after the test was completed, and no damage or deterioration of the concrete fill at the steel-concrete interface was noted outside this crushed zone. The measured values and observed behavior supports the concept of binding action described later in this paper. The presence of bending in the tube resulted in full composite action even in Specimen CFT3, which was designed to prevent normal bond stress transfer.

The test results show that CFT elements primarily subjected to bending can achieve full composite action without adding supplemental bond stress transfer mechanisms and can meet the flexural performance requirements. Push-out tests that use an eccentrically applied axial load with bending moment show much larger bond stress measurements than concentrically loaded push-out tests, but much of this data is from embedded section bond stress pushout tests (Murrow, 1997). The reason for this effect can be understood in the context of the prior discussion. Members subject to flexure sustain curvature and transverse deflections along their length. If the steel and concrete fill are fully bonded, the transverse deflections and curvature are directly determined through conventional engineering expressions in that full composite action is developed. If the steel and concrete are not fully bonded, the curvature and transverse deflections of the steel and concrete differ, and the different deformations in the steel and concrete will result in binding at multiple locations (minimum of 3) on the inside of the tube. The binding produces a large contact force at these locations and prevents slip of the concrete to permit shear stress transfer between the steel and concrete.

These results and their theoretical arguments suggest that engineers should not consider adding supplemental bond stress transfer mechanisms in cases of combined bending and axial loads but should consider supplemental mechanisms for cases of large axial loads (with or without low bending moment demands). Therefore, it is not necessary to check bond stress with members subject to shear and flexure only. Evaluation of the bond stress for axial loads only should be considered using provisions such as described in the prior section and mechanical force transfer section that follows. This conservative approach is frequently employed in design, and it is rational based upon these test results.

MECHANICAL SHEAR TRANSFER

In view of the limited natural bond capacity in CFT members, many engineers employ mechanical shear transfer elements to achieve full composite action. Some specifications specifically require that the mechanical transfer elements be used for composite columns (ACI 318, 2005). In some structures, shear stud connectors were welded to the inside of the tube, as depicted in Figure 9a. While mechanical devices can provide a calculable slip resistance, shear studs connectors develop their full shear resistance by deformation as shown in Figures 9b and 9c. Deformation of the shear stud also causes slight deformation of the wall of the tube, which may result in local separation of the concrete and steel thereby reducing the natural bond stress at the steel-to-concrete interface. Even very slight deformation of the wall of the tube or damage to the concrete disturbs the contact between the wall of the tube and the concrete fill, resulting in loss of friction in this area.

Tests that have been performed on CFT with and without shear connectors (Roeder et al., 1998; MacRae, Roeder, Gunderson and Kimura, 2004) show that the resistance achieved

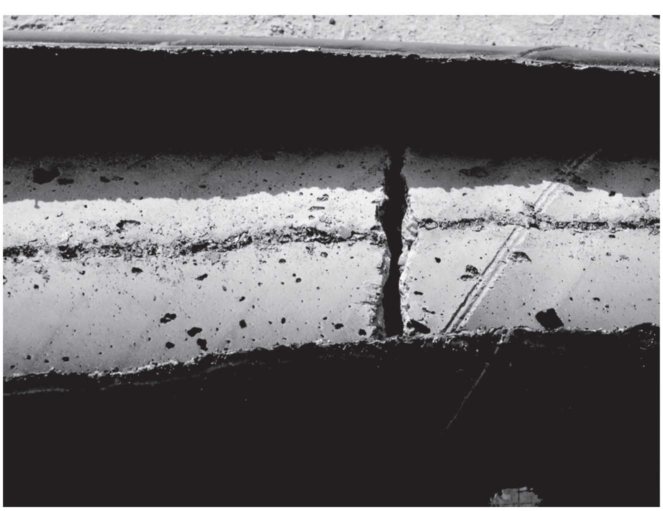


Fig. 8. Photo of the concrete core and damaged concrete region after completion of testing.

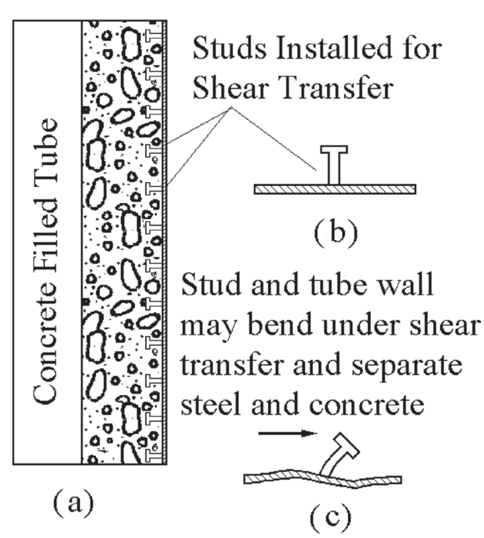


Fig. 9. Force transfer with typical shear stud connectors.

with shear connectors may be smaller than that achieved without any shear studs unless the studs are designed to provide the full stress transfer resistance. If shear studs are employed, the connections should be designed to achieve the full axial stress transfer, and a large number of connectors may be required. It is necessary to note that the shear connectors are not necessarily uniformly loaded when installed in a group (see Figure 4a), and this nonuniform load distribution increases the demand on individual studs. Finally, it must be recognized that installing a large number of shear connectors inside a steel tube can be costly and difficult; as a result, shear studs may not be the optimal choice for providing mechanical stress transfer in CFT applications.

If the slip or connector deformation is large, deterioration of the concrete at the bond interface will occur. Stiff mechanical transfer provides a binding action that prevents relative slip, and the resistance achieved with friction can be very large. The results by Thody (2006) and others suggest that it is desirable to use mechanical elements that have sufficient stiffness to limit slip, because binding action and slip restraint aid in ensuring interface friction and composite action.

Stiff mechanical transfer may be accomplished by several means. Annular rings or stiff bars such as shown in Figure 10 may be very effective if they are placed at regular (but not necessarily close) intervals over the length of the tube. The annular rings must be stiff to transfer force between elements, but not excessively stiff compared to the stiffness of the wall of the tube, because deformation of the wall of the tube is undesirable. The ring thickness need not be overly large to achieve the stiff transfer due to the strength and stiffness of the well-confined concrete. Connectors of this type have been used in some structural applications, but their use has been limited, because the internal rings are difficult to install.

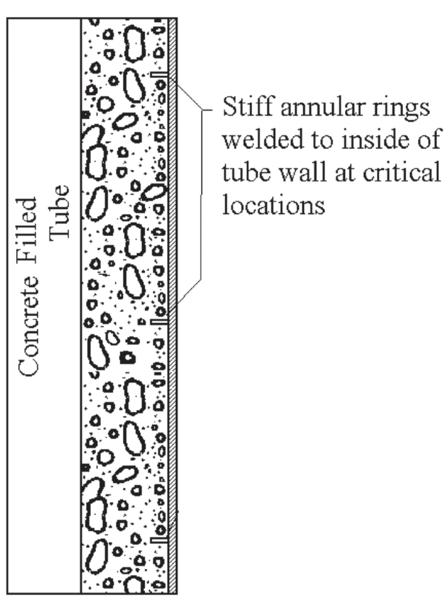


Fig. 10. Annular rings for shear transfer.

CONNECTIONS TO CFT COLUMNS

A more desirable way of accomplishing stiff mechanical transfer is accomplished by using conventional structural connections, to achieve this force or stress transfer. Figure 11 shows three typical structural connections, which accomplish these goals. With respect to composite action, these connections are similar in that (1) they all penetrate the tube with a stiff structural element to provide the binding, and (2) they restrain slip to ensure composite action in the CFT element without any additional transfer elements. In addition, they are designed to transfer the forces between members, as required by the loading and system configuration.

Figure 11a shows an internal diaphragm connection, such as those commonly used in Japan for moment resisting frame connections. These connections join the beam flanges to diaphragms, which penetrate the tube at both the top

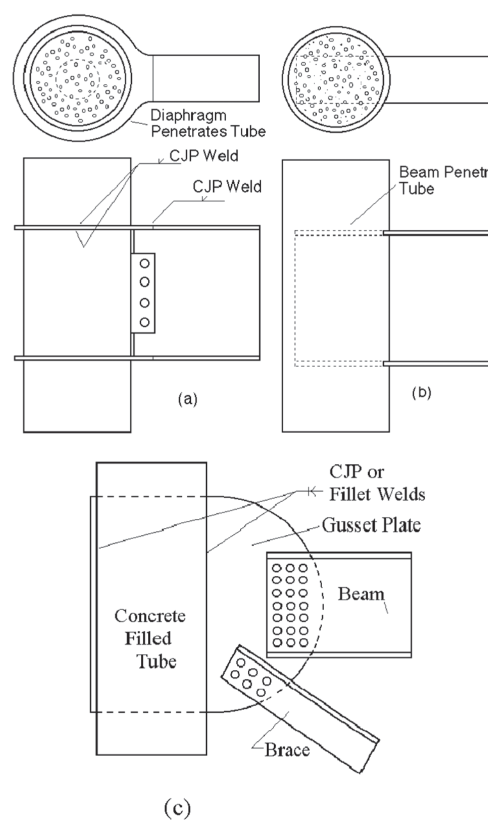


Fig. 11. Penetrating CFT connections: (a) internal diaphragm connection; (b) penetrating beam connection; (c) penetrating braced-frame gusset plate connection.

and bottom flange locations. Research has shown that these connections develop the full plastic moment capacity of the beam, develop the composite behavior of the column, and provide good ductility under seismic loading (Fukumoto, 2005). However, these connections are regarded as very expensive for U.S. practice, because complete joint penetration (CJP) welds are required around the full perimeter of the tube at four locations at each beam-column connection. Figure 11b shows an alternate penetrating connection for seismic resisting moment frames. This connection simply requires an I-shaped slot through the steel tube so that the steel beam can penetrate and engage the concrete fill. The tube must still be welded to the beam, but the weld is quicker and more economical than the four CJP tube welds required for the internal diaphragm connection. This connection was experimentally investigated (Schneider, 1996), and it has been shown to provide excellent ductility and inelastic performance combined with the full plastic resistance of the beam and the full composite resistance of the CFT member. It is a much more economical connection. However, there may be greater difficulty in achieving geometric control during fabrication and erection of this connection, because of less clear and precise fit-up points for the connection.

External connections such as shown in Figure 12 are possible, but in general their performance is inferior to the penetrating connections used with circular CFT. Reasonable performance has been achieved with rectangular CFT and through-bolted connections, such as illustrated in Figure 13. The through bolts in these connections also provide some binding action to limit relative slip between the steel and concrete, and good performance has been achieved (Ricles et al., 2004). However, care must be taken in designing these connections for seismic demands to ensure that the controlling yield mechanism is not the tensile yield capacity of the through bolts, since this yield deformation can lead to early tensile fracture of the bolts (Kanatani et al., 1988).

Concentrically braced frames (CBFs) are commonly used for seismic design, because they economically provide lateral resistance and stiffness needed to ensure good performance during small to moderate seismic loads. CBFs are more commonly used for seismic design in the United States in recent years, because of the problems noted with steel moment resisting frames during the 1994 Northridge earthquake. These problems were resolved through research performed during the SAC Steel Project (FEMA, 350; FEMA, 355), but steel moment resisting frames are today more expensive and complex than prior to the 1994 earthquake. CFT columns are very suitable for CBFs, because there are large axial load demands in the columns of braced frames, and CFT members have large axial stiffness and resistance capacities. However, there is significant difficulty in connecting the CFT column to the other structural members. The penetrating gusset plate connection in Figure 11c has been investigated in a recent research study (MacRae, et al., 2004). This connection has been shown to provide good force transfer and seismic performance of the system without any additional shear connectors or other mechanical stress transfer components inside the tube. This connection is also a penetrating connection and works by direct bearing through edge of the gusset on well-confined concrete. Ribs have been attached to the side of the plate or holes may be cut into the plate to increase bearing resistance where needed, and this connection appears to offer the benefits of direct bearing with an economical connection configuration.

The previous discussion has shown that connections with one or more penetrating elements can provide full composite action negating the need for additional mechanical shear connectors. The stiff penetrating elements block, bind and restrain the concrete fill against significant slip movement. The penetrating elements provide direct bearing force transfer to both the steel and concrete fill, and the resulting restraint provides natural bond stress (both frictional

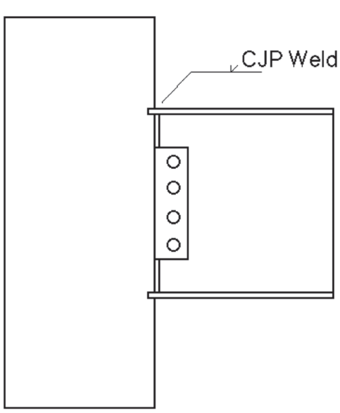


Fig. 12. Typical external steel-to-steel CFT connection.

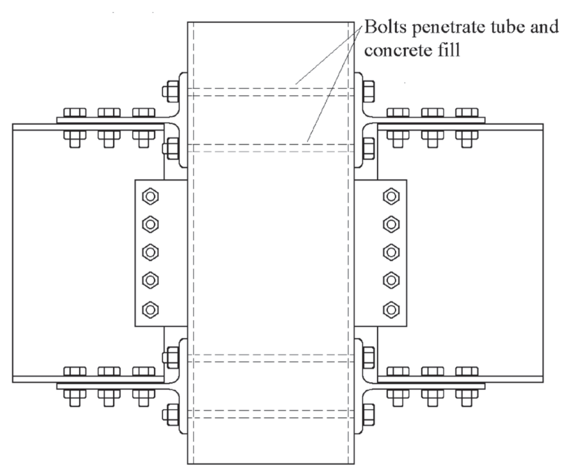


Fig. 13. Through-bolted connection for rectangular CFT.

resistance and adhesion) over a length of the tube. As a result, there is a great deal of interaction between the composite action and bond stress issues and the connections used to join CFT columns to other structural element.

Circular tubes provide significantly better performance as CFT members than rectangular tubes, because of better confinement of the concrete fill, significantly larger natural bond stress, and reduced potential for local buckling and deterioration of the tube. These observations are documented in research results such as Figure 3a, and large body of research performed in Japan and other countries (Morino, 1999). However, the circular geometry is not well adapted to many commonly used structural connections. Research in progress at the University of Washington is aimed at developing improved connections for circular CFT columns in structural applications to take advantage of their promising engineering properties. Several connections are under investigation with this research program, but a primary portion of the study has focused on CFT column-to-footing connections. CFT columns and bridge piers may have large plastic rotation demands at the column-to-foundation and the column-to-pier cap connections. Therefore, robust connections that are capable of transferring the full moment capacity and sustaining cyclic plastic rotation demands are required in high seismic zones.

A new CFT column-to-foundation connection has been proposed (Kingsley, 2005; Kingsley et al., 2005), and is illustrated in Figure 14. The connection is a hybrid of the embedded and base plate connections, and it has been evaluated experimentally using the test setup illustrated in Figure 5b. The proposed connection employs a annular ring or flange, which is welded to the base of the steel tube with a complete joint penetration weld as shown in Figure 15. It is a ring, rather than an end plate, and the ring extends slightly into the tube to provide blocking and binding to the concrete fill, as described earlier in this paper. There are no reinforcement or dowels within or penetrating into the tube. Therefore, the embedded tube and annular ring are solely responsible for the connection force and moment transfer. The connection offers economy, efficiency and continuity of the concrete fill of the CFT pier with the foundation not afforded by full plate connections. The flange serves as a temporary attachment for the tube, while concrete is being placed, and it also serves as a stress transfer rib for the composite connection.

The construction procedure is accomplished in one of two ways. First, the connection can be constructed by placing the footing in two lifts as illustrated in Figure 14a. The lower lift is cast, and the tube is then temporarily attached to the lower lift by anchor bolts. The remainder of the footing and concrete fill are cast in a second lift. The footing is reinforced with shear and flexural reinforcement as required for normal foundation design. This CFT connection was tested and excellent seismic performance resulted, as illustrated in Figure 16.

As an alternative, the footing can be cast in a single operation, and the connection can be constructed as illustrated in Figure 14b. With this alternative, a recess is formed into the concrete footing with light gauge corrugated metal. This recess must be of slightly larger diameter than the outside diameter of the annular ring for the steel tube. The tube is then fit into the recess and temporarily anchored as shown in the top part of Figure 14b. Fiber-reinforced grout is used to fill the recess around the perimeter of the tube, and the concrete fill is placed in the tube, using either traditional or self-consolidating concrete. This connection also has been tested and provides similar or superior performance to that

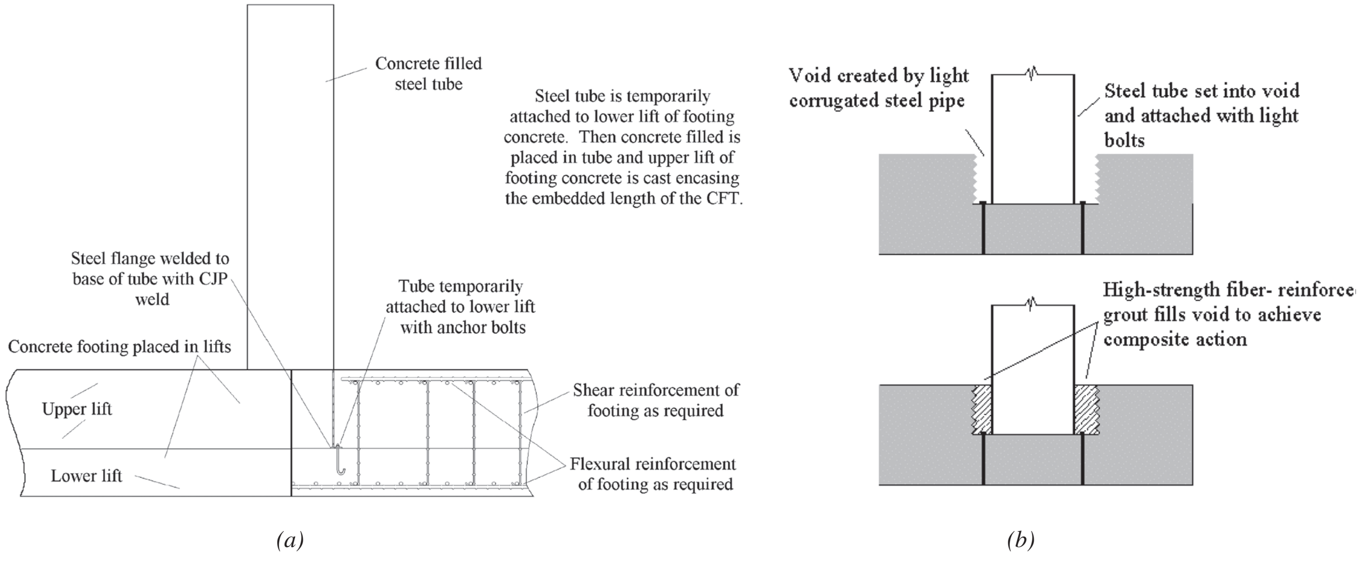


Fig. 14. Proposed CFT foundation connections: (a) embedded type; (b) separated construction type.

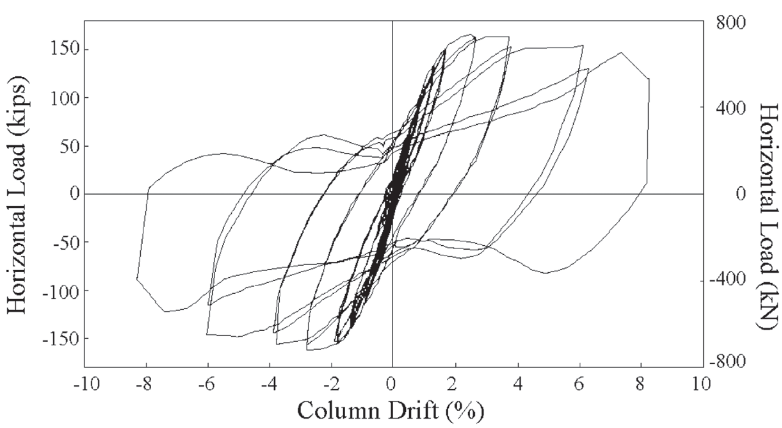
illustrated in Figure 16. This alternate connection permits the use of CFT columns or piers with precast caps, although the precast cap options have not been experimentally evaluated.

The experimental research results show that the CFT pier or column member and connection developed good ductility with large energy dissipation and inelastic deformation capacity. The connection developed the full composite resistance and ductility of the CFT column. Yielding occurred within the steel tube, and ultimate failure was buckling of the steel tube and tearing of the steel at the buckled region. This failure did not occur until nearly 8% displacement drift. The photo in Figure 16b shows the CFT column at approximately 6% drift; the tube has buckled but still had minimal deterioration in resistance. The tube retained its integrity through large inelastic deformations, and there were no signs of damage until nearly 4% drift. Thus, the CFT pier is expected to meet serviceability performance limit states at even relatively large drift levels. In comparison, damage to a reinforced concrete pier would be expected at drift ratios of 1.5% to 2%, with significant strength deterioration at 5 to 6% drift (Kingsley et al., 2005). It should be emphasized that the tube is quite slender with relatively thin walls and high strength steel (d/t = 80, and $F_y \approx$ 525 MPa or 76 ksi). Local buckling is well controlled, and excellent cyclic inelastic deformation capacity is achieved. Connections of this type allow CFT members, which may not only improve construction time and costs, but structural performance as well.

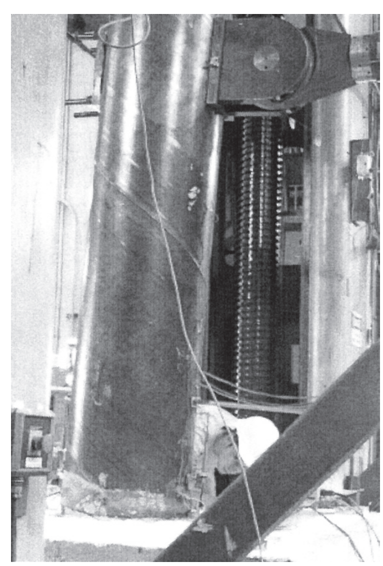
Fig. 15. Photo of welded annular flange.

RELEVANCE TO AISC SPECIFICATION

Chapter I of the AISC *Specification* (2005b) is the primary source of design information for CFT columns in the United States. The current provisions recognize that shear transfer is required between the steel tube and the concrete fill. It requires that the transfer be accomplished by "direct bond interaction, shear connection, or direct bearing," but no guidance in assessing the bond interaction is provided. This paper shows that circular CFT with relatively large bending



(a)



(b)

Fig. 16. Experimental behavior from typical connection test: (a) column shear vs. story drift; (b) photo of specimen at large drift levels.

moments will likely have enough bond stress or "direct bond interaction" to achieve full composite behavior. This paper also suggests that CFT with primarily axial load will require "direct bearing" such as provided by penetrating connections or annular rings unless the diameter and *d/t* ratio of the tube are small, because of limitations and variability in bond stress.

The AISC *Specification* makes little distinction between circular and rectangular CFT other than for slenderness limits. Circular tubes also better confine the concrete fill and have greater resistance to local buckling of the tube. Hence, circular CFT offers clear advantages to rectangular CFT, although circular CFT may result in greater difficulty with connections to other structural members. The AISC *Specification* recognizes the greater confinement provided by circular CFT by permitting $0.95f'_c$ for the concrete strength rather than the $0.85f'_c$ permitted for rectangular CFT and other applications. However, Figure 3a clearly shows that circular CFT have much larger bond stress transfer capability and more easily develop full composite action, and no distinction is made for this difference in the provisions.

Finally, the AISC *Specification* limits circular CFT columns to a ratio of $0.15E/F_y$, and with a nominal 70-ksi (480-MPa) yield stress, the slenderness limit is 62. However, this paper has summarized a number of tests with yield stress in excess of 70 ksi (480 MPa) and tube slenderness of 80. The specimens performed well with full composite resistance and good ductility as shown in Figures 6 and 16. Hence, it is reasonable to suggest that relaxation of this slenderness limit may be possible for some applications.

SUMMARY AND CONCLUSIONS

This paper has provided an overview of composite construction with CFT piers and columns. The focus of this work has been on inelastic cyclic deformation capacity and seismic design, but the conclusions apply equally well to other applications. The paper has summarized a range of past articles and reports as well as two recent experimental research efforts. The bond stress and interaction between the steel and concrete are the primary focus of this work. A number of key issues can be noted.

1. Shear transfer between the inside wall of the steel tube and the concrete fill is essential to providing optimal performance and composite behavior of CFT structural components. The natural bond stress achieved between the steel and concrete is contributed by friction, mechanical interlock and adhesion between the two materials. However, in the absence of mechanical shear transfer connectors, friction is by far the dominate effect.

2. Circular CFT offers significant benefits over rectangular CFT, since circular CFT provides better confinement of concrete fill, greater bond stress between the steel and concrete, and greater resistance to local buckling and deterioration of inelastic response during cyclic loading. These facts are documented by experimental comparisons such as Figure 3a and significant body of published research. However, the geometry of circular CFT results in greater difficulty in forming connections between the CFT member and other structural elements.

3. In the absence of mechanical restraint and in the presence of significant axial load, the utility of frictional bond stress is limited to relatively stocky (small *d/t* ratios), small-diameter CFT elements, because the benefits of friction are lost due to concrete shrinkage and the smaller radial stiffness of thin wall tubes. Past research (Roeder et al., 1998) shows relatively small bond stress capacity with diameters and *d/t* ratios greater than 12 to 16 in. (300 to 400 mm) and 50 or 60, respectively. However, friction is invariably adequate for circular CFT members with large bending moments, because binding action between the concrete fill and the steel tube develops large frictional transfer. Further, the detrimental effect of large diameter and large slender ratio is not apparent with large bending moments. As a result, it is recommended that the bond stress be checked only for axial load transfer.

4. If the natural (frictional) bond stress is inadequate to accomplish the required axial force transfer, mechanical transfer is required. Shear studs are commonly used in composite construction, but they are not a desirable method for transferring shear in CFT members, because shear studs are relatively flexible and they do not work well with natural bond. If shear studs are used, they should be conservatively designed to transfer the entire axial load component. Stiff, annular rings are better shear transfer elements for CFT, but they are difficult to install on the inside of the tube.

5. It is shown that a wide range of penetrating connections offer the benefits of annular rings. These penetrating connections provide direct bearing of the steel on the concrete. The concrete is well confined by the steel tube, and very large bearings stresses can be developed. Several different connections were described and summarized.

6. A brief summary of a new CFT column-to-foundation connection was provided. The connection is economical, simple and rapidly constructed. It provides excellent ductility and inelastic deformation capacity. It also offers promise of the use of precast concrete cap beams with the CFT members. Work continues on this connection.

7. Finally, the experiments described in this paper were all performed on spirally welded high-strength steel tubes ($F_y \approx 525$ MPa or 76 ksi), relatively large diameters (20 in. or 405 mm), thin walls, and relatively large d/t ratios (d/t = 80). These tubes are well outside the range currently permitted for seismic design in U.S. design provision. Despite this, the tubes performed well. They achieved considerable ductility and inelastic deformation capacity. Local buckling occurred, but it occurred at relatively large deformations after the concrete had crushed. Ultimate failure occurred as tearing of the steel in the buckled area. The spiral welds did not have a detrimental effect on the inelastic performance of the tube, as long as the spiral welds were of good quality with a matching metal tensile strength, and good CVN toughness characteristics.

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