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A SYNOPSIS OF STUDIES OF THE MONOTONIC AND CYCLIC BEHAVIOR OF CONCRETE-FILLED STEEL TUBE BEAM-COLUMNS

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1. Introduction

In current international practice, concrete-filled steel tube (CFT) columns are used in the primary lateral resistance systems of both braced and unbraced building structures. There exist applications in Japan and Europe where CFTs are also used as bridge piers. Moreover, CFTs may be utilized for retrofitting purposes for strengthening concrete columns in earthquake zones.

The CFT structural member has a number of distinct advantages over an equivalent steel, reinforced concrete, or steel-reinforced concrete member. The orientation of the steel and concrete in the cross section optimizes the strength and stiffness of the section. The steel lies at the outer perimeter where it performs most effectively in tension and in resisting bending moment. Also, the stiffness of the CFT is greatly enhanced because the steel, which has a much greater modulus of elasticity than the concrete, is situated farthest from the centroid, where it makes the greatest contribution to the moment of inertia. The concrete forms an ideal core to withstand the compressive loading in typical applications, and it delays and often prevents local buckling of the steel, particularly in rectangular CFTs. Additionally, it has been shown that the steel tube confines the concrete core, which increases the compressive strength for circular CFTs, and the ductility for rectangular CFTS. Therefore, it is most advantageous to use CFTs for the columns subjected to the large compressive loading. In contrast to reinforced concrete columns with transverse reinforcement, the steel tube also prevents spalling of the concrete and minimizes congestion of reinforcement in the connection region, particularly for seismic design. Numerous tests have illustrated the increase in cyclic strength, ductility and, damping by filling hollow tubes with concrete. Recent applications have also introduced the use of high strength concrete combined with high strength thin-walled steel tubes with much success. When high strength concrete and thin-walled steel tubes are used together, the more brittle nature of high strength concrete is partially mitigated by the confinement from the steel tube, and local buckling of the thin steel tube is delayed by the support offered by the concrete. Progress in concrete technology has made it possible to utilize concrete strengths over 15 ksi in CFT beam-columns.

A number of additional economical benefits stem from the use of CFTs. The tube serves as formwork in construction, which decreases labor and material costs. In moderate- to high-rise construction, the building can ascend more quickly than a comparable reinforced concrete structure since the steelwork can precede the concrete by several stories. The cost of the member itself is much less than steel and roughly equivalent to reinforced concrete on a strength per dollar basis for low to medium strength concrete (Webb, 1993). When compared to steel moment resisting frames, in unbraced CFT frames, the amount of savings in steel tends to grow as the number stories increases (Morino et al., 1996). On the other hand, relatively simple beam-to-column connection details can be utilized for rectangular CFT members. This also results in savings for the total cost of the structure and facilitates the design process. In addition, the steel tube and concrete act together to provide natural reinforcement for the panel zone, which reduces the material and labor costs of the connections. With the use of high-strength concrete, CFTs are stronger per square foot than conventional reinforced concrete columns (Webb, 1993). In high-strength applications, smaller column sizes may be used, increasing the amount of usable floor space in office buildings. The smaller and lighter framework places less of a load on the foundation, cutting costs again. These advantages have secured an expanding role for this versatile structural element in modern construction.

A primary deterrent to widespread use of CFTs is the limited knowledge regarding their behavior. A number of factors complicate the analysis and design of concrete-filled steel tubes. A CFT member contains two materials with different stress-strain curves and distinctly different behavior. The interaction of the two materials poses a difficult problem in the determination of combined properties such as moment of inertia and modulus of elasticity. The failure mechanism depends largely on the shape, length, diameter, steel tube thickness, and concrete and steel strengths. Parameters such as bond, concrete confinement, residual stresses, creep, shrinkage, and type of loading also have an effect on the CFT's behavior. Axially loaded columns and, in more recent years, CFT beam-columns and connections, have been studied worldwide and to some extent many of the aforementioned issues have been reconciled for these types of members. However, researchers are still studying topics such as the effect of bond, confinement, local buckling, scale effect, and fire on CFT member strength, load transfer mechanisms and economical detailing strategies at beam-to-CFT column connections, and categorization of response in CFTs and their connections at all levels of loading so as to facilitate the development of performance-based seismic design provisions. It should also be noted that, despite a recent increase in the number of full-scale experiments, the majority of the tests to date have been conducted on relatively small specimens, often 6 inches in diameter or smaller (see Tables 1 through 6). This is due to the load limits of the testing apparatus and the need to run the tests economically. Whether these results can be accurately extrapolated to the typically larger columns used in practice remains a pertinent and debatable question, although recent research in Japan has begun to address this important issue (Morino et al., 1996).

This section summarizes the behavior of CFT columns, beams, and beam-columns, as well as providing a brief summary of the behavior of beam-to-CFT column connections under a variety of loading conditions. The conclusions of the majority of experimental investigations, as well as selected related analytical investigations conducted throughout the past few decades, are presented and discussed. Following the summary of CFT behavior are individual summaries of significant CFT studies. Key experimental information from these studies is tabulated in Tables 1 through 6. The scope of this report covering member behavior is limited to CFTs that are filled completely with concrete, and CFTs that make no use of reinforcing bars, shear connectors, internal ribs, or other interior connectors. The scope is otherwise comprehensive.

2. Axially Loaded CFT Columns

Columns loaded axially in compression (either concentrically or eccentrically) will behave in one of two distinct ways. Columns with a small L/D ratio (short columns) are governed by cross-section strength. These types of columns reach their ultimate capacity when both the steel and the concrete reach their strength limit point, i.e., yielding of the steel and crushing of the concrete. Eccentric loading will have little effect on this type of column. The second type of behavior pertains to columns with a larger L/D ratio -- intermediate or long (slender) columns. These columns are governed by stability and fail by either elastic or inelastic column buckling.

A load applied eccentrically will tend to cause buckling to occur earlier than an equal load applied concentrically (concentric also implies that the column is perfectly straight). Straight columns under purely concentric axial loading rarely, if ever, exist in practice. Therefore, a more realistic approach to examining CFT column behavior incorporates the bending moment caused by geometric imperfections or eccentricities.

Short CFT Columns. *General Behavior.* When a concentric axial load is applied to a short concrete-filled steel tube column (assuming the load is applied uniformly across both materials), the steel and the concrete will both begin to deform longitudinally. At these initial strains, Poisson's ratio of the steel exceeds Poisson's ratio of the concrete (0.28 versus 0.15 to 0.25) (Gardner and Jacobson, 1967). This results in a greater lateral expansion of the steel, and little interaction between the two materials occurs. During this stage of loading, the steel and the concrete sustain load independently of one another. Thus, the longitudinal stress along the steel tube remains approximately constant. At a strain of approximately 0.001, microcracking in the concrete begins to occur and the lateral expansion of the concrete increases and begins to approach the constant lateral expansion of the steel. The concrete expansion reinitiates interactive contact between the two materials, which induces bond stresses to develop, and results in biaxial stresses in the steel and triaxial stresses in the concrete. This causes the longitudinal stresses in the steel tube to change as a function of the transfer of force between the steel and concrete. The strain at which interaction and subsequent confinement occurs varies typically from 0.001 to 0.002. Knowles and Park (1970) state that confinement occurs suddenly at a strain of about 0.002 (about $0.95f_c'$), after the concrete begins a rapid volumetric expansion (dilation). Other authors suggest a more gradual increase in confinement beginning shortly after microcracking at a strain of 0.001, and reaching full confinement at 0.002 (Tsuji et al., 1991; Zhang et al., 1991).

Once confinement occurs, the steel tube experiences circumferential stresses from the lateral pressure of the expanding concrete in addition to the longitudinal stresses. If the steel tube has not yet yielded, this biaxial state of stress effectively decreases the amount of additional axial load the steel can sustain before yielding occurs. If, on the other hand, the steel tube is in a state of yielding when the biaxial stress state initiates, the steel will be unable to sustain the longitudinal yield stress. In either case, the effective elastic stiffness of the steel tube decreases and the tube sheds some of its axial load to the concrete. While the rapid expansion of concrete has a deleterious effect on the longitudinal steel capacity, the load-carrying capacity of the concrete component of the CFT is enhanced. The confinement of the steel tube augments the axial strength of the concrete core. For circular sections in particular, this increase in axial strength of the concrete actually outweighs the corresponding decrease in steel strength, resulting in an overall increase in the capacity of the CFT section.

While circular sections can effectively develop circumferential tension to exert lateral pressure on the concrete, the flat sides of rectangular sections provide little perpendicular pressure to restrain the expanding concrete (Furlong, 1967). Only the corners of rectangular tubes can exert confinement and this effect is negligible. Therefore, the strength of the rectangular CFTs in excess of its nominal axial load capacity is attributed primarily to the strain hardening of the steel tube. However, as will be described in subsequent sections, the encasement of the concrete in rectangular sections does have benefits, namely increased ductility and toughness.

When high strength concrete is utilized, the low dilatation of concrete prevents any significant confinement effect. Thus in most experimental studies, the axial load capacity of short columns with high strength concrete was often less than their nominal cross section strengths, unless the *D/t* ratio was small.

The load-deflection behavior of short columns exhibits different trends depending on their post-peak response. Circular CFT columns generally show strain-hardening or elastic-perfectly plastic type of load-deflection curves. For rectangular columns, a degrading (i.e., softening) load-deflection curve is commonly observed. The degradation of the load-deflection curves is also typical for stub columns with high strength concrete.

Failure Mechanism: CFTs with Thick-Walled Tubes. Short columns can be further subdivided into two categories based on the *D/t* ratio (the ratio of the tube diameter to the tube thickness). Concrete-filled steel tubes with "thick" walls will exhibit the more standard mechanism of failure. The concrete becomes confined at a strain of approximately 0.002 and additional axial strength is achieved. However, if the strength of the steel exceeds approximately 55 ksi (the stress corresponding to a longitudinal strain of approximately 0.002), the concrete will likely reach its compressive strength limit and may crush before the steel yields, which is an undesirable mode of failure (Furlong, 1967). In addition, this might cause elastic local buckling of the steel tube. SSRC (1979) thus specifies a steel yield strength limit of 55 ksi for composite columns. For lower strength steels, the failure of thick-walled short columns begins with the yielding of the steel. As the yielding of the cross-section of the tube proceeds, the concrete begins to fail by crushing. With confinement, the concrete can continue to sustain additional load until the steel tube fails (usually by extensive local buckling or full plastification of the cross-section) marking the ultimate strength of the section. The location of failure is usually mid-height for square and rectangular CFT specimens. For square CFTs, local buckling generally spreads to all four flanges. However, for rectangular CFTs, the longer flanges are more susceptible to local buckling. This causes steel yielding in the transverse direction along the shorter sides (Shakir-Khalil, 1991). The failure of circular CFTs also may take place at the mid-height with extensive local buckling, and local buckling then spreads to the ends (Schneider, 1998).

Failure Mechanism: CFTs with Thin-Walled Tubes. Thin-walled specimens fail either by elastic or inelastic local buckling of the steel tube, or by a shear failure in the concrete (at a strain of approximately 0.005) followed by local buckling of the steel tube, which is in a state of yielding. In either case, the longitudinal strains in the member are not large enough to allow significant confinement of the concrete core to occur. The local buckling of the steel is, however, delayed by the influence of the concrete core. The concrete forces the steel to buckle in an outward mode, which provides three advantages. First, when buckling occurs, the distance between the top and bottom flanges of the steel tube increases rather than decreases (as it would without the concrete core) which prevents the section modulus from decreasing significantly. The second advantage is that the concrete tends to spread the local buckling over a larger region, mitigating severe strain concentrations which tend to cause cracking. Third, delaying the local buckling effectively stabilizes the tube wall in the elastic range and often enables full development of the yield stress prior to buckling. In their tests for thin-walled high strength CFT columns, O'Shea and Bridge (1997c, 1997d) observed two types of failure patterns. The columns failed either by local buckling combined with concrete crushing or by sudden failure without any local buckling. Concrete crushing took place beyond an axial strain of 0.003. Prion and Boehme (1989) found, in their tests of thin-walled CFTs using high strength concrete, that the steel yielded, then the concrete failed by shearing, after which the capacity of the section fell off considerably, and local buckling occurred in the steel tube at the location of the shear failure in the concrete. Luksha and Nesterovich (1991) noticed a similar type of failure in their tests of large diameter tubes. Tubes with *D/t* ranging from 54 to 104 began failing at 90% of ultimate with the formation of buckles along the tube. At incipient failure, the concrete sheared causing the steel to buckle completely. This shearing/local buckling type of failure is undesirable because it is often sudden and could be catastrophic (Knowles and Park, 1969). A limiting value of *D/t* should be adhered to in order to prevent such failures. SSRC (1979) establishes the following limits on the *D/t* ratio. For rectangular tubes:

$$\frac{D}{t} \leq \sqrt{\frac{3 \cdot E}{f_y}}$$

and for circular CFTs:

$$\frac{D}{t} \leq \sqrt{\frac{8 \cdot E}{f_y}} .$$

Eccentric Loading of Short CFT Columns. Short columns subjected to eccentric loading or having initial out-of-straightness show insignificant lateral displacement for a given applied moment. These columns are essentially governed by the axial load effects alone. In their eccentric axial tests on thin-walled circular high strength concrete CFT stub columns, O'Shea and Bridge (1997c, 1997d) found that ductility improves with increasing eccentricity. Bridge (1976) also found that square CFT columns with larger eccentricity have a more stable post-peak response. This is because for small eccentricities, more concrete is effective in resisting the loads. Thus, softening exhibited in the post-peak response of concrete is reflected in the overall response of the column. Confinement has been shown to be negligible in both circular and rectangular tubes for *e/D* (eccentricity to diameter) ratios greater than 0.125 (Neogi et al., 1969; Bridge, 1976).

<u>Long CFT Columns</u>. *General Behavior*. If the CFT column is sufficiently slender, stability rather than strength will govern the ultimate load capacity and second order effects become more critical. Overall column buckling will precede strains of sufficient magnitude to allow large volumetric expansion of the concrete to occur. Hence, for overall buckling failures, there is little confinement of the concrete and thus little additional strength gain. Many authors have agreed that a slenderness ratio (*L/D*) equal to 15 generally marks a rough boundary between short and long column behavior. Neogi et al. (1969) originally proposed this value for eccentrically loaded columns (eccentrically loaded columns, rather than concentrically loaded columns, are the case of interest since this type of loading will cause an earlier buckling of the member). Chen and Chen (1973), Bridge (1976), and Prion and Boehme (1989) confirmed the *L/D* value of 15. Knowles and Park (1969) proposed a KL/r_c value of 44 (approximately equal to an *L/D* of 12), above which confinement does not occur. In addition, the AIJ (1990)

specification also has a minimum *L/D* ratio of 12 for long columns. However, Zhong et al. (1991) specified a lower value of *L/D* equal to 5 above which confinement does not occur.

Both elastic and inelastic flexural buckling can occur in CFT columns. CFTs that fail by inelastic buckling are referred to here as intermediate CFT columns and CFTs that fail by elastic buckling are referred to as long or slender CFT columns.

Failure Mechanism: Intermediate CFT Columns. Intermediate columns will undergo some steel yielding and/or concrete crushing before buckling occurs. As the steel yields and the concrete crushes, the stiffness of the member decreases and its capacity to withstand buckling decreases. Rectangular tubes tested in biaxial bending by Shakir-Khalil and Mouli (1990) yielded first in the compression corner of the tube at approximately 90% of the failure load, followed by tensile yielding of the opposite corner, leading to an overall buckling failure. By increasing the size of the tube or the strength of the concrete, additional strength may be attained. Conversely, increasing the yield strength of the steel tube or increasing the length has detrimental effects. Both of these factors will increase the relative contribution of the steel and decrease the concrete contribution, which begins to negate the benefits of filling the tube with concrete.

Failure Mechanism: Slender CFT Columns. The method of failure of long concrete-filled steel tube columns is characterized by overall elastic buckling of the member (Shakir-Khalil and Zeghiche, 1989). This type of column has a sufficiently large *L/D* ratio to cause buckling before any significant yielding occurs in the column. Tsuda et al. (1996) observed the same type of failure in their tests of slender concentrically loaded columns. The columns with an *L/D* greater than 18 did not reach their plastic axial strength and failed by flexural buckling.

Eccentric Loading of Long CFT Columns. As the length of a column increases, it becomes more sensitive to eccentric loading effects and initial out-of-straightness. Lateral deflections of the column increase the likelihood of a buckling failure rather than a cross-section strength failure. Eccentric loading will impart external and internal moments on the section. The eccentricity of the load itself produces a primary moment. Bending of the member chord (P-δ effect) induces secondary moments in the cross section due to the lateral deflection of the column. The failure mechanism of long columns due to eccentric loading will be similar to the concentric case, except that failure will occur at a smaller load and the failure load will decrease with an increase in eccentricity. Shakir-Khalil (1991) tested long CFT columns under eccentric loading and observed that CFT columns become more sensitive to initial imperfections when the eccentricity is low. However, Tsuda et al. (1996) found that as the *L/D* ratio increases, the behavior of both circular and square columns is less affected by eccentricity. In the same experimental study, CFT columns with an *L/D* ratio greater than 18 did not attain their full plastic moment capacity, and they failed due to flexural buckling. This indicates that slender CFT columns under eccentric axial loading are susceptible to elastic flexural buckling.

<u>*Stiffness of Axially Loaded CFT Columns*</u>. The stiffness of concrete-filled steel tubes is complicated by the concrete core and the interaction between the two materials. The modulus of elasticity, the moment of inertia, and the effective area for tensile loading are well known for steel, but these properties are difficult to predict for concrete because of its inhomogeneity. They vary depending on the concrete strength, the occurrence of tensile cracking, and long-term load effects, among other things. SSRC (1979) has proposed a modified elastic modulus that is the sum of the moduli of elasticity for the steel and concrete, with a reduction factor of 0.4 imposed on the initial stiffness of the concrete to account for creep and tensile cracking. Other authors

have proposed summing the individual rigidities of each component (Hajjar et al., 1997a). The overall stiffness of the CFT, though, will be mostly influenced by the steel tube, since the steel has a much higher modulus of elasticity than the concrete.

Cyclic Behavior of CFT Columns. Relatively few tests have been conducted to study the cyclic behavior of axially loaded CFT specimens. The most notable tests in this regard have been performed in the U.S. and Japan on CFT bracing members subjected to alternate cycles of tensile and compressive loads. Experimental and analytical work considering the cyclic behavior of columns has been primarily limited to the more common and deleterious effects of cyclic shear and bending (see Sections 3, 4, and 5) rather than cyclic axial compression loads.

Liu and Goel (1988) compared hollow and concrete-filled rectangular tube braces. They showed that the addition of concrete increased the number of cycles to failure and the amount of energy dissipated. Of course, in tension, only the steel effectively resists the axial force. In compression during cyclic loading, the concrete primarily augments the buckling strength of the steel tube by delaying and reducing the severity of local buckling.

Failure Mechanism. As rectangular bracing members are cycled, the member is perturbed at incipient buckling, causing the compression flange to buckle locally in an outward direction. This is followed by an inward pinching of the webs, which forms longitudinal cracks at the corners that propagate along the member until failure occurs. Failure is delayed until the concrete at the hinge point crushes, which occurs only after many cycles of loading (Liu and Goel, 1988). Kawano and Matsui (1989) tested circular CFT bracing members under repeated axial loading. It was determined that the concrete infill delays local buckling and provides high deformation capacity. The failure of the CFT members took place with local buckling at various locations along the length and then tension cracking at the top of one of these local buckling bulbs.

Other Effects. *Type of Loading*. A concrete-filled steel tube can be loaded in three basic ways: load the steel only, load the concrete only, or load both materials uniformly. Tests by Gardner and Jacobson (1967) showed that loading the steel tube alone does not increase the failure load above that of a hollow tube. Ideally, loading the concrete alone would be the most efficient method. In the absence of bond, the steel would be used only to confine the concrete and would contain no longitudinal stresses. Steel used in this way is approximately twice as effective as steel resisting purely axial stresses. Orito et al. (1988) proposed a type of member, an unbonded steel tube concrete member, that behaved in such a manner. They showed that a CFT with an antifriction material applied to its inside resulted in greater compressive strength and a delayed yielding of the steel which was subject to only circumferential stresses. O'Shea and Bridge (1997c, 1997d) conducted similar tests with thin-walled tubes. They found that the steel tubes were mainly subjected to circumferential stresses until the concrete started to crush. However, as the concrete pressed into the steel tube, longitudinal stresses developed in the steel tube. Thus, in many cases, some bond will exist between the two materials, inducing an axial stress in the steel, creating a biaxial state of stress and decreasing the amount of confinement provided by the tube. In fact, Gardner and Jacobson (1967) showed that loading the concrete alone did not increase the ultimate load of the section beyond that obtained by loading both materials equally. Tests by Prion and Boehme (1989) confirmed this. However, simultaneous loading of both materials is the likely scenario in construction.

Residual Stresses. The level of residual stresses in steel tubes is highly dependent upon the manufacturing process and the shape of the cross section (Sherman, 1992). Hot-formed circular or rectangular structural tubes, either seamless or welded, have negligible residual stresses. Since residual stresses in hot-formed sections result from differential cooling of the member, structural tubes, which cool relatively uniformly as a result of their shape and enclosed nature, contain only a small amount of residual stress. Structural tubes manufactured by cold-forming, however, are much more likely to contain residual stresses. Residual stresses in cold-formed tubes are a result of stresses induced in the manufacturing process that exceed the elastic limit of the steel. Welded cold-formed tubes will contain higher residual stresses than seamless tubes, and cold-formed rectangular tubes will have higher residual stresses than circular tubes. The residual stresses may be classified as membrane and through thickness. Membrane residual stresses may develop both longitudinally and circumferentially, and they can reach on the order of 80% of the yield stress of steel tube. These types of residual stresses vary from tension to compression across the cross section, and their effect on the overall behavior of steel tube is generally insignificant (Sherman, 1992). On the other hand, through thickness residual stresses occur in the radial direction and can exceed the yield stress of steel tube (Bridge, 1976). The effect of through thickness residual stresses are taken into account implicitly in the stress-strain results of coupons of the steel cut longitudinally from the tube. These residual stresses will decrease the proportional limit of the material, producing an earlier decrease in the elastic stiffness of the section and effectively rounding the stress-strain curve (Sherman, 1992). Furlong (1968) discovered extensive residual stresses in his tests of rectangular cold-formed welded tubes. Proportional limits were less than 50% of nominal yield strength. For sections containing residual stresses, two common methods have been incorporated to account for the decreased strength. Gardner (1968), Knowles and Park (1969), and others have established the yield stress as the stress corresponding to 0.2% longitudinal strain of the steel rather than nominal yield to better account for residual stresses. Also, some authors have established reduced elastic moduli to account for the loss of stiffness. Finally, to avoid the problem of residual stresses altogether, many investigators have used annealed steel tubes, in which the tube is heated to a high temperature and then cooled it slowly and uniformly to relieve any built-in stresses.

Creep and Shrinkage. Early tests performed by Furlong (1967) showed that creep has an influential effect on the long-term behavior of CFTs, although the effect is somewhat mitigated by the tubular steel confinement. Furlong (1967) found that slow loading rates could decrease the strength up to 15%. Nakai et al. (1991) compared plain concrete specimens and CFTs to determine the effects of creep and shrinkage over a period of 6 months. They found that the amount of shrinkage due to drying was negligible compared to the plain concrete. Creep, however, did produce an increase in the longitudinal strains over time. The authors obtained creep coefficients (the ratio of the final strain to the initial elastic strain) of 1.44 to 1.61, which is about half of the value obtained for plain concrete. Terrey et al. (1994) also investigated creep and shrinkage by testing plain concrete and CFT columns. They found the shrinkage strains to be insignificant for CFTs. The test results indicated that the creep coefficient of CFTs is approximately 50 to 60% that of plain concrete. In addition, a final creep coefficient (i.e., after a long period of time) of 1.2 was obtained for CFTs. However, larger diameter tubes may be more susceptible to the effects of shrinkage on bond (Roeder et al. 1999), as discussed later.

Bond. Two types of bond may exist between the steel and the concrete: microlocking and macrolocking (Virdi and Dowling, 1980). The first type, microlocking, refers to the bonding of the concrete with surface irregularities ("roughness") on the inside of the tube. Microlocking provides the initial stiffness of the load-interface deflection curve and defines the ultimate bond strength. At the ultimate bond strength, the concrete at the interface crushes and the stiffness decreases substantially. The second type of bond, macrolocking, is a mechanical interaction between the concrete and steel due to the nonuniformity of the tube, i.e., out-of-straightness or out-of-roundness. Macrolocking provides frictional resistance which enables some bond to be maintained beyond the ultimate bond strength after local crushing of the concrete at the interface. Virdi and Dowling (1980) established a characteristic bond strength of 150-160 psi and concluded that surface preparation and the amount of compaction are the only significant parameters that will increase the amount of bond. Parameters such as concrete strength, length of the concrete/steel interface, the tube thickness, and the tube diameter had only negligible effects on the amount of bond. Shakir-Khalil and Zeghiche (1989) performed push-out bond tests on rectangular steel tubes filled with concrete and found that there was less bond than for reinforcement bars or even circular steel tubes. The effect of shrinkage and the relative flexibility of the rectangular tube walls reduced the bond strength. Bond has also been shown to be less in rectangular sections than circular sections due to the shrinkage of the concrete, which will have a greater effect on the less uniform rectangular section.

Roeder et al. (1999) stated that the bond between steel and concrete depends on three factors, including radial enlargement of wet concrete due its pressure on the steel tube, roughness of the tube wall, and shrinkage of the concrete. They identified three states of bond depending on the relative magnitude of the aforementioned factors. In state A, the amount of radial enlargement due to concrete pressure is larger than the shrinkage of the concrete. State B has the condition that shrinkage of the concrete is greater than the summation of radial displacement and the amplitude of surface roughness, which indicates the loss of contact between the steel and concrete. State C is common in practical applications, and in this case the shrinkage of concrete is greater than radial displacement, while the difference between the two is smaller than the amplitude of the surface roughness. Roeder et al. (1999) also indicated that chemical adhesion enhanced the initial bond between the steel and concrete. Moreover, they found that microlocking between steel and concrete results from mechanical interlocking of concrete with surface irregularities as well as the friction between steel and concrete due to lateral pressure of concrete. In their push-out tests, Roeder et al. (1999) obtained an exponential bond stress distribution along the column length under low axial loads, and as slip starts to occur the bond stress distribution became more uniform. Roeder et al. (1999) also tested several push-out specimens under eccentric loading, determining that eccentric loading improved the bond strength up to approximately 2.5 times the bond strength from the concentric loading tests. Roeder et al. (1999) provided an equation for the bond strength of circular CFTs that is a function of the *D/t* ratio.

Morishita et al. (1979a, 1979b) and Tomii et al. (1980a, 1980b) conducted push-out tests on circular and square CFTs by loading the steel tube alone. They found that the concrete strength did not have any influence on the bond strength for square shapes, while in circular tubes, bond strength decreased when high strength concrete was utilized. They also determined that mean bond stress remained constant even if large values of slip occurred between steel and concrete. Morishita et al. (1979a, 1979b) specified bond strength values of 28.46 to 56.92 psi and 21.34 to 42.69 psi for the circular and square CFTs, respectively. Morishita et al. (1982) performed push-out tests on square CFTs under constant axial load and cyclic shearing force. It was found that when the shear forces acting on the columns increased, the magnitude of mean bond stress improved, and as the amount of slip, increased the bond stresses became constant. In addition, the concrete strength was again found to have little effect on the mean bond stress.

Despite the large number of push-out tests available in the literature, they provide limited information for bond strength and bond stiffness. The scatter of these two quantities obtained from the push-out tests is large (Hajjar et al., 1998a, 1998b). However, these tests provided insight to the shape of the load-slip behavior. In addition, push-out tests also do not always represent well the loading conditions in an actual composite CFT frame. In buildings, the loading is usually applied to a CFT column at the connection through the steel girders. The load may be applied to the steel tube alone or to the steel tube and concrete core simultaneously depending on the type of connection. Moreover, these regions are the places where the highest bond demand exists in a frame. Therefore, connection tests are often more suitable to investigate bond transfer between steel and concrete. Dunberry et al. (1987) tested CFT columns framed by steel girders at the mid-height. They found that the rotation of girders in the lower part of the connection caused a pinching effect, increasing the bond strength and the rate of load transfer from the steel to the concrete. Dunberry et al. (1987) also determined that capping and grouting the end of CFT columns caused some portion of load transfer to be achieved in the upper part of the column rather than restricting the load transfer mechanism to the connection region. This caused more favorable load transfer from the steel to the concrete. Shakir-Khalil and Mahmoud (1995) performed simple beam-to-column connection tests similar to Dunberry et al. (1987). The load transfer between the steel and concrete was completed within a distance of D above the connection.

3. Pure Bending (CFT Beams)

Concrete-filled steel tubes subjected to pure bending behave much like hollow tubes. In fact, several authors suggest using the plastic moment capacity of the steel tube as a lower bound for the strength of a CFT in pure bending. The steel contributes a large portion of the stiffness and strength since it lies at the periphery of the section where the material has the most influence. In general, CFT beams fail in a ductile manner. A limited number of tests have been performed regarding CFTs under pure flexural bending since their primary application thus far has been as columns.

Strength. The tensile resistance of a CFT depends primarily on the steel alone. Therefore, moment resistance is highly influenced by the steel tube. The only contribution of the concrete to moment resistance occurs due to the movement of the neutral axis of the cross section toward the compression face of the beam with the addition of concrete. This effect can be enhanced by using thinner tubes or higher strength concrete (Furlong, 1967). Tests by Bridge (1976) showed that the concrete core only provides approximately 7.5% of the capacity in member under pure bending. However, Kitada (1992) reported (without quantifying) that substantial increases in strength, and especially ductility, could be achieved by filling hollow steel tubes with concrete.

Pure bending tests of CFTs by Lu and Kennedy (1994) indicated an increase in moment capacity due to concrete infill for the square and rectangular beams of between 10 and 35% as compared to hollow tubes. The amount of improvement was larger for the thinner tubes. In addition, the maximum curvature improved substantially. This was attributed to the high deformation of the top flange of the tube as local buckling was delayed by contact with the concrete. Lu and Kennedy (1994) also examined the effect of moment gradient on the flexural strength by varying the shear span of the specimens. They found that it has little effect on the moment and deformation capacity. The steel and concrete strains along the depth of the beam were similar, so that the slip between the steel and concrete was negligible and has little influence on the moment capacity.

Elchalakani et al. (2001) obtained similar results as Lu and Kennedy (1994) for circular CFT beams and found that concrete infill enhanced the moment capacity between 3 and 37%, with larger improvement for the thinner sections. The slip between the steel and concrete was insignificant, and an *a/D* ratio of 2.7 was specified to provide full load transfer without any slip. Elchalakani et al. (2001) also presented a *D/t* ratio limit of 112 for circular CFT beams to achieve plastic flexural strength.

Failure Mechanism. In pure bending tests by Lu and Kennedy (1994) with medium strength concrete and low *D/t* ratios, the specimens exhibited a linear moment-curvature response followed by a nonlinear stiffness degradation region, approaching the maximum moment asymptotically. The failure took place by local buckling in the compression flange of the tube, concrete crushing in the locally buckled area, and often yielding of the tube in tension. Beams containing high strength concrete or beams having high *D/t* ratios begin failing when the concrete fractures in shear after the steel has begun to yield. The concrete shearing causes further stretching and then a subsequent rupture of the steel tube. Local buckling in the compression region also occurs near failure (Prion and Boehme, 1989). The beams tested by Prion and Boehme (1989) showed very ductile behavior and did not seem affected by the slip that occurred between the steel and concrete (i.e., the ultimate moment capacity was not lowered). The CFT beams with large *D/t* ratios tested by Elchalakani et al. (2001) exhibited local buckling distributed along the compression flange, and failure took place with tensile fracture at the bottom fiber of the section that had the most severe local buckling.

Stiffness. The stiffness of a CFT beam depends to some degree on whether or not bond exists at the interface of the two materials. In the absence of bond, there will be no interaction between the materials, little or no concrete augmentation, and the composite stiffness will depend heavily on the stiffness of the steel tube. Tests by Furlong (1968) showed that specimens exhibited a lower stiffness than that calculated assuming plane sections remain plane in bending (i.e., that bond between the materials exists). He concluded after testing greased and non-greased specimens that little or no bond existed at the interface between the steel and the concrete in his tests. The only interaction between the two materials was the physical pressure. Although Prion and Boehme (1989) questioned the existence of bond in beams, their analytical results assuming strain compatibility most accurately predicted the behavior of specimens in pure bending. Lu and Kennedy (1994) found that the flexural stiffness of CFT beams was approximately 1.1 times greater than that of hollow tubular beams, on average. Therefore, they recommended using the flexural stiffness of steel for the composite section as a conservative approach. With respect to initial stiffness early in the loading history, Gourley et al. (1994) suggest adding the steel and concrete flexural rigidities to get an estimate of the initial composite flexural rigidity, essentially assuming initially perfect bond between the materials.

Cyclic Loading. Concrete-filled steel tubes in bending dissipate a significant amount of energy with only a slight decrease in strength as the loading is cycled. While the strength of CFTs during subsequent cycles is not greatly affected by the slip between the two materials, beam specimens do show a loss of stiffness due to a lack of bond and cracking of the concrete after the first cycle (Prion and Boehme, 1989). Beyond the first cycle, the stiffness of the CFT is primarily due to the steel alone since the concrete is cracked on both faces of the beam. As the specimen is cycled, the gap in the tensile zone of the concrete increases as the member approaches zero curvature, and the member strength and stiffness decrease. As the gap closes upon reverse loading, strength and stiffness increase again. This sometimes results in some pinching behavior, with the stiffening occurring due to strain-hardening, compressive concrete re-engaging, and possibly interaction between the tube and the core (Prion and Boehme, 1989). With each subsequent cycle, the tensile cracks in the concrete form more quickly, which also contributes to an overall decrease in strength and stiffness.

4. Combined Axial Load and Bending (CFT Beam-Columns)

Typically, beam-column tests are differentiated from eccentric loading tests by the magnitude of the induced moment and the type of failure. Eccentric load tests refer to tests where a moment was introduced to accentuate initial out-of-straightness, or tests in which the moment was of relatively small magnitude (e.g., less than 20% of the ultimate moment capacity). The eccentricity hastens the onset of buckling in a typical failure. Beam-column tests, on the other hand, have moments of significantly larger magnitude, warranting careful consideration of the interactive failure due to combined moment and axial load. These moments may be introduced transversely, via loading of a connected member, or by a number of other methods.

General Behavior. Several key parameters influence the behavior of beam-columns. Among them are the D/t ratio, the axial load ratio (P/P_o), and the L/D ratio or the slenderness of the member. The D/t ratio determines the point of local buckling, and it affects the ductility of the section. A smaller D/t ratio will delay the onset of local buckling of the steel tube. Tubes with high D/t ratios (above approximately 50) will often exhibit local buckling even before yielding of the section occurs. A low D/t will provide greater ductility, illustrated by the long plateau in the moment-curvature diagrams for such columns (Tomii and Sakino, 1979a). Tomii and Sakino (1979a) also showed that beam-columns with low D/t ratios (24 and 33) can sustain the maximum moment after local buckling. Beam-columns with higher D/t ratios (44) began to lose capacity as the curvature increased, although only under large axial loads did the capacity drop significantly. Ichinohe et al. (1991) determined that for circular CFT specimens with a D/t less than 53, the moment increases after local buckling without strength degradation.

The axial load ratio is a second important parameter in CFT beam-column behavior. The relationship between the moment and axial load, as illustrated by interaction diagrams, is typically a curve that bulges outward for low axial compressive loads (i.e., with the maximum moments exceeding M_o) and then approaches P_o approximately linearly, showing a rapid decrease in the moment capacity for high axial compressive load ratios. As axial compression is added to a CFT member in bending, the contribution of the concrete begins to increase, utilizing the composite action of the section to a greater extent. The axial load increases the strength of the concrete in a manner similar to the confinement effect discussed in Section 2. As with the *D/t* ratio, the axial load ratio has an effect on ductility. Large values of P/P_o lead to rapid moment capacity deterioration and brittle failures. If the steel on the compression side has buckled (which is more likely as the *D/t* ratio increases), a more brittle failure typically ensues. It can be seen that a combination of a high *D/t* and high P/P_o leads to undesirable modes of failure. Because of this, Tomii and Sakino (1979a), for example, limited their studies to beam-columns with P/P_o less than or equal to 0.5.

Finally, the *L/D* ratio will have a significant effect on the performance of the member. A number of authors (e.g., Chen and Chen, 1973; Bridge, 1976; Tomii and Sakino, 1979a; Fujimoto et al., 1996; Tsuda et al., 1996) have presented interaction diagrams (both from experimental and analytical research) that show the moment-axial load relationship for different *L/D* ratios. For a given cross section, the maximum moment (M_o) will remain approximately the same with an increase in length, but the maximum compressive axial force (P_o) will drop markedly.

With respect to material strength, Varma et al. (2000) conducted beam-column tests on square CFTs with high strength materials. He observed a reduction of initial stiffness and moment capacity with an increase in *D/t* ratio. The yield strength of the steel and the axial load ratio did not have any significant influence on the initial stiffness. However, when the yield strength of the steel increased, the moment capacity was enhanced. The amount of improvement was larger for low *D/t* ratios due to better resistance to local buckling. The steel strength also had little influence on the ductility seen in the CFT beam-columns. However, both high axial load levels and large *D/t* ratios caused a reduction in ductility, often more severe than is seen with normal strength materials. Failure took place due to local buckling of the flanges and extensive concrete crushing. In most of the cases, yielding of steel tube occurred prior to the specimen achieving its peak load. In the post-peak region, local buckling moved to the webs and tension cracking of the corners took place. Nakahara and Sakino (1998) also determined that, compared to using normal strength concrete, introduction of high strength concrete reduces ductility.

<u>Cyclic Behavior</u>. Concrete-filled steel tube beam-columns typically perform better under cyclic loading than comparable hollow tubes and reinforced concrete members. CFT members show very full hysteresis loops indicating large energy dissipation. Compared with a reinforced concrete member with the same slenderness ratio, steel ratio, and axial load ratio, the CFT exhibits a higher value of ultimate axial load and a higher amount energy dissipation (Huang et al., 1991).

As with monotonic loading, when high strength concrete was used for CFT beam-columns subjected to cyclic loading, Nakahara and Sakino (2000a) concluded that the response is governed partially by the level of axial loading on the member. Decreasing the *D/t* ratio increases ductility when the axial load ratio is high. However, they found that the effect of *D/t* ratio on ductility is less evident in the case of low axial load ratio, and less evident than for monotonic loading. Varma et al. (2000) tested square CFT columns with high strength materials under cyclic shear and also found that the *D/t* ratio had a less significant effect on ductility than for monotonic loading, but that high axial load levels caused a reduction in energy dissipation capacity and ductility. However, the secant stiffness of the specimens was larger when the axial load level was high due to higher contribution of concrete.

Failure Mechanism. The concrete in a CFT beam-column subjected to cyclic flexural loading contributes little strength in bending, but it does increase the capacity of the section by delaying the onset of local buckling of the steel tube (Kawaguchi et al., 1991). In their tests of beam-columns, Prion and Boehme (1989) noticed a pinching of the hysteresis loops similar to their beam tests due to the opening and closing of the concrete cracks during steel yielding (see Section 3). In the cyclic shear tests on square CFT columns with high strength materials (Varma, et al., 2000), steel yielding in tension and concrete crushing occurred either prior to or at the same time as local buckling of the flanges, which took place near the peak load. Following that stage, local buckling of the webs and corners of the tube occurred and the specimens failed with tension cracking of the steel tube corners. After local buckling in the web, the strength of the specimens started to deteriorate more dramatically than lower strength specimens.

5. Shear in CFTs

Tests of concrete-filled steel tubes subjected to shear or shear in combination with axial load and bending have been conducted primarily in Japan. The test results have demonstrated that CFTs have excellent shear resistance under both monotonic and cyclic loading.

Monotonic Behavior. The behavior of a CFT member under a shear-type loading is dependent upon essentially the same parameters as beam-columns, including the *D/t* ratio, the axial load ratio, and the shear span ratio (*a/D* ratio) which is analogous to the *L/D* ratio for beam-columns. Based on the shear span ratio, shear behavior can be divided into two types. For a small shear span ratio (0.83 to 1.0), diagonal shear cracking indicative of shear failure occurs in specimens that are also subjected to axial load (Tomii and Sakino, 1979a). For shear span ratios of 2.0 to 3.0, columns exhibit a flexure-type failure with plastic hinges forming at the specimen ends.

The *D/t* ratio and P/P_o ratio have much the same effect as discussed for beam-columns in Section 4. The point of local buckling typically occurs at or near the peak shear stress, but well in advance of the maximum rotation and occurs earlier as the *D/t* ratio of the section increases. Overall shear resistance (moment capacity) decreases with an increase in the *D/t* ratio or an increase in the axial load ratio. Such columns will behave in a brittle manner.

Cyclic Behavior. Concrete-filled steel tube members subjected to shear forces display a large amount of energy dissipation and ductility. Circular members tend to have more stable hysteresis loops and a greater ductility than rectangular tubes. However, experiments have shown that rectangular tubes tend to behave as circular tubes after a few cycles, as the buckling of the steel tube at the point of maximum shear transforms the critical regions from rectangular to circular in shape (Kawaguchi et al., 1991, Sakino and Tomii, 1981; Sakino and Ishibashi, 1985).

Members with relatively thin tube walls show some strength deterioration with successive cycles of loading, but still display a large amount of energy dissipation (Tomii, 1991). The strength deterioration results from the buckling of the steel tube and subsequent crushing of the concrete. Members with thick tube walls exhibit deformation behavior similar to thin-walled tubes. However, these members resist local buckling and concrete crushing well into the plastic range of strains (Council on Tall Buildings and Urban Habitat, 1979) providing greater overall shear resistance.

Axial load has been found to have little appreciable effect on the shear-carrying capacity of CFTs (Tomii et al., 1972). CFT specimens subjected to a high axial load ($P/P_o = 0.5$), tend to show a stabilization of the hysteretic loops and even a slight increase in shear resistance. As described earlier, rectangular sections buckle at the critical region and become circular in shape. The circular shape provides an increased confinement of the concrete, increasing the shear resistance. Sakino and Tomii (1981) also observed a considerable amount of axial shortening for columns with a P/P_o of 0.5 due to the combination of steel local buckling and concrete crushing. Values of axial shortening ranging from 27% to 34% of the section depth were measured.

Cyclically loaded rectangular specimens with an *a/D* ratio of 1.0 fail in shear, as opposed to the cyclically loaded specimens with *a/D* ratios of 2.0 and 3.0, which fail in flexure, much like the monotonic specimens (Sakino and Tomii, 1981; Sakino and Ishibashi, 1985). Short beam-columns show considerable energy absorption and display less strength deterioration than the longer columns that fail in flexure. Both lengths exhibit an initial decrease in capacity and then a slight increase as local buckling in the critical regions transforms the shape of the tube from rectangular to circular.

6. Torsion in CFTs

As with shear, few tests of CFTs under torsional loading have been done. In the limited tests performed, concrete-filled steel tubes performed quite well under torsional loading. The nature of the steel tube itself is conducive to excellent torsional behavior. A closed section such as a tube has a much greater torsional resistance than a W-section with a similar area. Also, since torsional stresses increase with radial distance, the orientation of the steel (which has a much larger shear modulus, *G*, than the concrete) at the perimeter, where stresses are a maximum, idealizes the torsional resistance of the section.

Torsional failure in a CFT is not abrupt or distinct, but is characterized by a large increase in torsional rotation at a fairly constant load. The failure is due to a combination of spiral cracking in the concrete and tensile yielding of the steel. The effect of axial load on the torsional response is for the most part detrimental. Lee et al. (1991) found that for axial loads up to one-half of the ultimate axial load, an increase in the axial load resulted in a corresponding increase in the torsional resistance of the member. Xu et al. (1991), however, found that the greatest resistance of the section occurred in the case of pure torsional load and any increase in the axial load resulted in a decrease in the torsional resistance.

7. CFT Connections

A wide range of beam-to-CFT column connections have been studied over the past several decades. Some type of connections transfer the load from the girders directly to the steel tube, while others transfer the load to both the concrete core and the steel tube. The connection types having girders welded to the steel tube are often suitable for simple connections (Dunberry et al., 1987; Shakir-Khalil and Mahmoud, 1995). However, for moment connections, these configurations impose high deformation demands on the steel tube, possibly causing fracture of the tube wall. This results in a deterioration of strength and stiffness. Thus, in some connection systems, it is common to distribute the girder force around the steel tube by means of internal and external diaphragm plates welded to the steel tube. For connections to circular CFTs, Schneider and Alostaz (1998) found that the connection types having extended plates or deformed bars passing into the concrete core improved both strength and stiffness. Among their specimens, the one with a continuous girder into the CFT column produced the most desirable cyclic response. For connections to rectangular CFTs, Peng et al. (2000) found that using split-tee connections with through-bolts or post-tensioned bolts provided excellent hysteretic performance.

8. Composite CFT Frames

Portal frames composed of rectangular CFT columns and wide-flange steel beams have been tested at several locations (Matsui, 1986; Kawaguchi et al., 2002), and stable hysteresis loops have been obtained. The concrete infill increased the local buckling strength of the member, and the concrete core continued to resist loads even after local buckling of the steel. This fact improved the strength of the frames relative to frames having hollow tube beam-columns. Fracture at the connections or extensive local buckling at the column ends were among the typical failure modes.

II. Summaries of Concrete-Filled Steel Tube Experimental and Analytical Studies

1. COLUMN, BEAM-COLUMN and FRAME TESTS

Klöppel, K. and Goder, W., 1957 (c; m)[1]

Introduction. The authors performed collapse load tests on hollow and concrete-filled steel tubes. They also established a formula for the design of CFTs. Three tests were examined in detail, with stresses and strains in both the steel and concrete tabulated for incremental values of concentric load. The paper was published in German, but much of the test data may be found in Knowles and Park, 1970.

Salani, H. J. and Sims, J. R., 1964 (c; m)

Introduction. The elastic and inelastic behavior of mortar-filled steel tubes in compression was investigated. Comparisons were made between the experimental values and theoretical values obtained from the tangent modulus formula.

Experimental Study, Results, and Discussion. The tubes in the study were composed of seamless, cold drawn, annealed steel and varied from 1 to 3 inches in diameter. Both hollow and mortar-filled tubes were tested. The ultimate load capacities of the 1 and 1.5 in. tubes were not augmented by the use of mortar. This may have been due in part to the low ratio of mortar to steel area. The 2 and 3 in. tubes showed an increase in strength when filled. All of the tubes failed by column buckling except for the 3 inch tubes which failed by local buckling.

Theoretical Discussion. The theoretical buckling loads for both the empty and mortar-filled tubes were computed by the tangent modulus formula:

$$P_{\mathrm{cr}} = \frac{\pi^2 \cdot E \cdot I}{L_{\mathrm{eff}}^2}$$

where E is the effective modulus of elasticity, and I is the effective moment of inertia. For both the empty and filled tubes, this modulus of elasticity value was obtained from the experimental results of a stress-strain compression test of a stub column. A least squares polynomial curve fit was employed to express the stress-strain relationship. The concept of a transformed section was avoided here because E_c varied inversely with the load.

Comparison of Results. With the exception of the 3 in. tubes which buckled locally, the theoretical buckling loads for the empty and mortar-filled tubes were within 16% of the experimental values.

[1] The type of element tested and the type of loading used are listed in parentheses after the authors (see the end of this section for this nomenclature).

<u>Gardner, N. J. and Jacobson, E. R., 1967</u> (c; m)

<u>Introduction</u>. The authors investigated axially loaded CFT compression members both experimentally and theoretically. The results were compared to ACI (American Concrete Institute 318-63) and NBC (National Building Code of Canada, 1965), as well as tests performed by Klöppel and Goder (1957). They attempted to predict the ultimate load of short CFTs and the buckling load of long CFTs. The data for the theoretical analysis of the long CFTs was obtained from an experimentally-derived load-deflection curve based on tests of a stub column with the same cross sectional dimensions. Furlong and Knowles wrote a discussion of this paper, parts of which were included in this summary.

<u>Experimental Study, Results, and Discussion</u>. At least two columns of each different tube size were tested and for each long column size a corresponding stub column was tested. Both the long columns and the stub columns first yielded in the longitudinal direction. Some interaction existed between the materials, as the CFT had a greater ultimate strength than the sum of the individual steel and concrete components. A limited investigation into the effects of varying the end conditions on the stub columns was conducted by alternately loading the steel only, the concrete only, and then both materials together. These results were tabulated but no general trends were cited in the discussion.

<u>Theoretical Discussion</u>. The authors discussed theoretical and design formulas from previous investigations, i.e., Considère, Russell, Klöppel and Goder, and others, and summarized some of their work.

Short Axially Loaded CFTs. A very detailed discussion of short column behavior was presented. The authors described the stress state of the CFT as load was applied. Initially, Poisson's ratio for steel (0.283) exceeded that of the concrete (0.15-0.25) and no confinement existed. As the load increased, the lateral strains in the concrete "caught up" to the strains in the steel, i.e., the Poisson's ratio of the concrete reached and then exceeded that of the steel. At this point, the tube began to restrain the concrete core and the hoop stress in the tube became tensile. At failure, the steel augmented the concrete strength as expressed by the equation

$$\sigma_{\mathrm{c}} = f_c^{'} + k \cdot \sigma_r$$

where k is an experimentally determined empirical factor with a value of about 4 and σ_r is the radial pressure on the concrete. The authors presented the following formula to represent the total load on the column:

$$P = A_c \cdot f_c^{'} + \frac{k}{2} \cdot A_s \cdot \sigma_{sc} + A_s \cdot \sigma_{sl}$$

The first term represents the concrete strength; the second, the strength contribution of the confined concrete due to the hoop stress in the tube; and the third term represents the steel strength under longitudinal compression. The bounds of this formula may be obtained by setting the hoop stress equal to yield (lower bound) or setting the longitudinal stress equal to yield (upper bound). In reality, though, the hoop stress will probably not reach half of the yield stress

(Knowles, discussion). Assuming compatibility, the axial stress in the steel may be determined if the strain in the concrete at crushing is known.

Long Axially Loaded CFTs. The buckling strength of a long column may be determined by the tangent modulus formula:

$$P_{cr} = \pi^2 \cdot \left(\frac{E_{st} \cdot I_s + E_{ct} \cdot I_c}{L^2} \right)$$

E_{st} and E_{ct} are the respective steel and concrete tangent moduli. The steel modulus was obtained from tension tests and the concrete properties were found by using a stub column test. This approach poses a problem because the loads carried individually by the steel and concrete must be known.

<u>*Design Formulation*</u>. ACI and NBC used the following formula to determine the allowable load:

$$P_a = 0.25 f_c^{'} \cdot \left(1 - 0.000025 \cdot \frac{L^2}{r_c^2} \right) \cdot A_c + \left(17000 - 0.485 \cdot \frac{L^2}{r_s^2} \right) \cdot A_s$$

The steel must have a yield strength of at least 33000 psi and $\frac{L}{r_s} \leq 120$. The results of this formulation are discussed in the following section.

<u>*Comparison of Results*</u>. Results were given for the experimental ultimate loads, the tangent modulus formula, and the ACI-NBC method. The long column tests revealed that the tangent modulus formula conservatively predicts the results within 0 to 16.8%. But the method required a stub column test to determine E_{ct}. The ACI-NBC implied a load factor at failure of 2.5, but experimental factors ranged from 3.37 to 5.13. For stub columns, the axial load showed good agreement. However, the authors cautioned against using their theoretical formulation because the full triaxial concrete strength was not developed in the tests. They stated that the additional section strength shown in the experiments was probably due more in part to the steel going into strain hardening, rather than the concrete being triaxially confined.

<u>Furlong, R. W., 1967</u> (c, bc; m)

<u>*Introduction*</u>. Experiments were performed to determine the accuracy of proposed theoretical estimates of the ultimate strength of beam-columns. CFT behavior was discussed in detail for concentric and eccentric loading situations. The author began with a discussion of some of the advantages and disadvantages of using CFTs. One advantage is the optimization of the steel strength and stiffness due to its location at the periphery of the cross section. A second advantage is that thinner steel sections may be used because the concrete core forces all local buckling modes outward, ensuring that the tube reaches its longitudinal yield strength before buckling. Finally, two economic advantages arise by using CFTs. Concrete has a lower cost to strength ratio and the tubes act as formwork which decreases construction costs. The author cited two main disadvantages: the questionable strength retention of the steel tubes in fires and

the lack of experimental evidence regarding connections and beam-columns. The prime objective of the paper was to provide experimental beam-column data and examine this type of element.

Experimental Study, Discussion, and Results. The tests were performed by applying a constant concentric axial load and increasing the moment to failure. Several conclusions were presented.

Creep. The author found that creep had an influential effect on the specimen behavior. For slower loading rates (i.e., 10 to 15 minutes at a given displacement), creep produced a load reduction of as high as 15%. Conversely, the author suggested that if tests are conducted continuously without pausing for measurements, an additional 10% gain in strength could be achieved.

Concrete-Steel Interaction. The author concluded that the two materials behave independently of one another. At strains below 0.001, the Poisson's ratio of concrete was one-half to two-thirds that of steel. This difference in lateral expansion tended to separate the materials. As strains increased, the concrete expanded laterally at a greater rate than the steel. Above 0.001, the concrete Poisson ratio began to approach that of steel. It was assumed that the steel had attained a confining state when the ratio of the measured circumferential and longitudinal strains in the steel sharply increased. This suggested an outward pressure from the concrete core. Not until about 90% of the load had been added did the ratio between circumferential and longitudinal strains divert substantially from the Poisson's ratio measured from the steel tubes alone in compression. This suggested that little benefit to the concrete arose from confinement activity of the steel encasement. The concrete core did, however, stabilize the steel wall of the tubes, preventing premature local buckling and allowing the tube to attain its full yield capacity.

Theoretical Discussion. *Stiffness of Axially Loaded CFT Columns*. The author suggested that both the steel and the concrete in columns under pure compressive loads should undergo the same amount of longitudinal strain. The modulus of elasticity, or stiffness, of the concrete tends to decrease around a strain of 0.0008. Steel, on the other hand, has a constant stiffness up to a strain of about 0.0010 to 0.0012. Its stiffness also decreases at a lower rate than concrete's stiffness beyond 0.0010. The steel will therefore begin to carry a higher and higher proportion of the load as the corresponding strains increase beyond 0.0008.

Strength of Axially Loaded CFT Columns. A lower limit for the strength of the cross section may be established by summing the force necessary to yield the steel in the longitudinal direction plus the amount of force carried by the concrete at the yield strain. Should the steel exert any confinement on the concrete, the amount of force the concrete carries before the steel yields will increase. The concrete will reach its ultimate strength at a strain value of about 0.002 and may begin to crush. To prevent the onset of crushing before steel yielding, steel with a yield stress above about 60 ksi should be avoided. Furlong recommended a maximum value of steel yield strength of 50 ksi to ensure the steel yields before the concrete crushes. The author also found that confinement of the concrete is much more likely in round sections because the steel may develop an effective hoop tension, whereas the flat sides of a rectangular tube are not effective in resisting perpendicular pressure.

Strength of CFT Beam-Columns. The lower limit to the pure bending capacity of CFTs is the plastic moment on the steel alone. The concrete core may increase the section's resistance to bending, but since the tensile resistance depends on the steel alone, in order for the ultimate moment to increase, the presence of the concrete must cause the neutral axis of the cross section to move toward the compression face. This would increase the tensile moment arm about the geometric centroid of the section, allowing additional capacity. A way to accomplish this is to use a thinner tube and/or higher strength concrete. The author formulated a load-moment interaction diagram in an attempt to predict the capacity of a CFT under combined loading. He plotted P_u/P_o versus M_u/M_o, where P_u and M_u are the measured ultimate loads and moments, respectively. P_o is the lower limit of axial load estimated by summing the yield strength of the steel and the load on the concrete at the time the steel yields (based on the strain at the yield load). M_o, the lower limit value for moments, is simply the plastic moment capacity of the steel tube alone. An estimate of the lower bound of the strength of a column is given by the equation:

$$\left(\frac{P_u}{P_o}\right)^2 + \left(\frac{M_u}{M_o}\right)^2 \leq 1.$$

<u>Comparison of Results</u>. The above interaction equation was very conservative for a number of experimental values, but it represents a reasonable lower bound in the absence of computers or a more rigorous approach. The author arrived at better results by considering the composite sections to be reinforced concrete sections and analyzing them as such using the capacity interaction equations specified in the ACI Building Code. The results were illustrated graphically in the paper. Although this method produced better results, the values were quite tedious to compute.

<u>Furlong, R. W., 1968</u> (c, bc; m)

<u>Introduction</u>. Design formulas and graphs (both the author's and those of AISC and ACI) were presented for columns and beam-columns based on axial load capacity, flexural strength, and flexural stiffness. Bond between the steel and concrete in CFTs was examined in detail, and creep and residual stresses were also examined. Experimental results from a range of different investigations, including the author's own, were compared with the theoretical and design formulations.

<u>Experimental Study, Discussion, and Results</u>. *Bond*. Initial eccentric load tests performed by the author exhibited a stiffness much less than the value computed based on a design formula using transformed areas. The assumption in the formula that plane sections remain plane in bending, and therefore that bond between the materials exists, may have contributed to this discrepancy. To examine this assumption, tests were performed to determine the influence of bond on the behavior of the specimens. Greased and non-greased tubes were tested to determine the strength contribution of the bond between the steel and concrete. For the axial load tests, the load-strain curves for “bonded” and “unbonded” were very similar. This was expected since the longitudinal strains in both materials should parallel one another. Surprisingly, however, the curves were also similar for the bending tests, which revealed that bond contributed little or no strength to the ungreased, or “bonded” member. Even at low stress the bond could not prevent sliding. Little change in behavior occurred between the “bonded” and “unbonded” specimens regardless of the tube shape, concrete strength, or wall thickness. The

author was confident that bond existed at the beginning of the test in the non-greased specimens. Therefore the bond must have broken at low loads to permit separation of the wall and the core. The only sustained interaction seemed to be the physical pressure between the two materials.

Residual Stresses. Tests on steel coupons taken from the tubes showed extensive residual stresses in cold-rolled and welded steel. The presence of residual stresses caused a "softening", or decreases in the elastic stiffness well before the yield point, translating into proportional limits that were in some cases as low as 50% of nominal yield.

Creep. Creep, as indicated by reductions of load over time, became prominent after longitudinal steel yielding. The author suggested that the ultimate strength under sustained loading could be 10% higher than the incremental test loads obtained.

<u>Theoretical Discussion</u>. *Stiffness*. Since it was shown that little interaction between the materials took place, the author presented a basic formula for computing the composite stiffness of a CFT member:

$$E \cdot I = E_c \cdot I_c + E_s \cdot I_s$$

The modulus of elasticity for concrete was computed by the standard ACI formula:

$$E_c = 33 \cdot w^{1.5} \sqrt{f_c'}$$

To account for the softening of the steel due to residual stresses, the author used a steel modulus of elasticity E_s equal to 25×10^6 psi. The author's final formulation based on the gross moment of inertia of the cross section was:

$$E \cdot I = I \cdot \left[25 \cdot 10^6 \cdot \alpha + (1-\alpha)^2 \cdot \left(57000 \cdot \sqrt{f_c'} - 25 \cdot 10^6 \right) \right]$$

where α is the ratio between the steel area and the gross area. A similar formulation was used to compute the axial stiffness, $E \cdot A$. The CFTs behaved much like reinforced concrete. Stiffness increased with the amount of axial load prior to bending. Then at about 50% of axial capacity, the measured flexural stiffness decreased because the stiffness of plain concrete drops sharply for stresses above $0.50 \cdot f'_c$. To account for the stiffness reduction of the concrete at higher axial loads, the author further modified the above stiffness function by the factor:

$$4 \cdot \frac{P}{P_o} \cdot \left(1 - \frac{P}{P_o} \right)$$

for axial load P between $0.50 \cdot P_o$ and P_o, where P_o is the squash load.

Strength. The author presented the formulas derived in Furlong (1967) and reiterated the accuracy obtained by treating the section as reinforced concrete when analyzing the interaction functions. He also presented quite accurate equations for long columns under pure axial loading, but these will have limited application in practice since moments almost always exist.

Comparison of Results. The accuracy of the theoretical formulation for stiffness was as much as 25% off from the experimental results, and the author suggested that his formulas, though convenient, be used only as a crude approximation.

Further Research. The author suggested the following topics of study: connection details, fabrication techniques, and the investigation of the behavior and economy of high-strength concrete (8000+ psi).

Gardner, N. J., 1968 (c; m)

Introduction. An experimental investigation of axially loaded CFTs using spiral welded tubes was described and the results were compared with the ACI-NBC design formulas and the values obtained from the tangent modulus method.

Experimental Study, Results, and Discussion. The procedure involved testing specimens under pure axial load to failure. The spiral welded tubes used in the test were manufactured by bending a narrow steel strip in a helix and welding up the tube along the junction of the steel edges. At the paper's publication, this type of tubing cost 40% to 50% less than equivalent cold drawn seamless tubing.

Residual Stresses. Care was taken to measure the residual stresses in the tubes. Large manufacturing stresses were realized. The results were graphed in the paper and the author suggested that the values serve as a pessimistic bound since the ratio of wall thickness to diameter (a factor in the amount of residual stresses present) of the tested tubes was greater than typical tubes in practice. He also recommended taking the stress-strain properties from compression tests of complete cross sections which gave a much better indication of residual stresses than coupon tests.

Axially Loaded Columns. The long columns failed by buckling and exhibited no abrupt loss of capacity at large strains, i.e., the columns behaved plastically much like the steel stress-strain curve. Overall, the spiral welded tubes behaved much like seamless tubes.

Theoretical Discussion. The author assumed the following as a lower bound for the strength of a stub column under axial compression (see also Gardner and Jacobson, 1967) :

$$P_o = A_c \cdot f_c^{'} + A_s \cdot f_y$$

This value was conservative for all experimental values. For long columns, as in the 1967 paper, the author used the tangent modulus formula:

$$P_{cr} = \pi^2 \cdot \left(\frac{E_{st} \cdot I_s + E_{ct} \cdot I_c}{L^2} \right)$$

E_{st} and E_{ct} are the respective steel and concrete tangent moduli. These values were obtained from the uniaxial compression test for the steel and by calculation from the triaxially loaded stub column for the concrete. Estimating the tangent modulus for steel in such a fashion was incorrect if the steel was loaded biaxially. The author found, as Furlong (1968) did, that at higher strains, Poisson's ratio for concrete exceeded that of steel and a hoop tension was induced in the steel tube. The yield criterion of the steel tube was expressed as the sum of the tensile hoop stress and the compressive longitudinal stress. Therefore, the longitudinal stress at which yield occurred under biaxial loading could be significantly less than that under uniaxial loading. Since the tangent modulus of the steel decreased rapidly near yield, using uniaxial properties for the steel was wrong since it was probably yielding much sooner.

Design Formulation. The ACI-NBC method as described in the summary of Gardner and Jacobson (1967), may be used with confidence for design in both seamless and spiral welded tubes (which behaved much like seamless).

Knowles, R. B. and Park, R., 1969 (c, bc; m)

Introduction. This paper investigated axially loaded CFTs and hollow tubes over a wide range of slenderness ratios, with particular attention paid to the effect of the slenderness ratio on the lateral pressure exerted by the tube on the concrete. The authors also looked at the effect of loading the materials together and individually (i.e., load the concrete and not the steel and vice versa). They examined concentrically loaded columns theoretically by the tangent modulus approach and they constructed a straight line interaction formula to estimate the behavior of eccentrically loaded CFTs.

Experimental Study, Discussion, and Results. *Concentrically Loaded Columns*. All of the hollow tubes tested under axial loads failed by inelastic flexural buckling; no local buckling was observed before the ultimate load was reached. Since local buckling is often sudden and catastrophic, the authors suggested that the ratio of the wall thickness to the diameter of the tube should be limited, although no specific values were given. The concrete-filled tubes failed in the same manner as the hollow tubes, with the region of plasticity always located at midheight. Tests of concrete cylinders showed a sudden and large increase in volume at a strain of around 0.002. In the CFT this increase in volume will cause the steel to exert a confining pressure and increase the strength of the concrete. The authors found that for concrete core slenderness ratios ($K{\cdot}L/r_c$) less than 44.3, confinement of the concrete will occur. For most of the author's tests, though, overall column buckling preceded strains of sufficient magnitude to cause volumetric expansion and confinement. It was noted that the square tube columns with small slenderness ratios did not gain additional strength due to confinement. Although it has been shown by other investigators that square tubes provide less confinement than circular tubes, square ties in reinforced concrete have produced good confinement results. The authors stated that the issue of square tube confinement has yet to be resolved.

Eccentrically Loaded Columns. Both the hollow and concrete-filled tubes failed by overall column buckling at midheight, where the moment was largest.

Theoretical Discussion. *Axially Loaded Columns*. A CFT can be loaded axially in three distinct ways: load the steel but not the concrete, load the concrete but not the steel, and load the steel and the concrete such that the longitudinal strain is the same in both materials. Each method produces a different section behavior. The first method essentially mimics the behavior of a steel tube alone, with the concrete failing at the maximum load the steel tube can carry. Tests by Gardner and Jacobson (1967) showed that loading the steel tube alone does not increase the failure load above that of a hollow tube. Under eccentric loading, the concrete may tend to delay the onset of local buckling and increase the bending resistance. Loading the concrete alone idealizes the use of the steel tube which provides a confining stress and does not resist axial load. Steel used in this way is approximately two times as effective as steel resisting longitudinal stresses. But in reality, some bond will exist between the two materials, inducing some longitudinal stress in the steel tube. This creates a biaxial state of stress which decreases the circumferential capacity and the amount of confining pressure that the steel is able to exert on the concrete. Gardner and Jacobson found that this type of loading does not increase the failure load above that obtained by loading both materials simultaneously. Loading both materials so that longitudinal strains are equal is the probable situation in actual structures and was the type of loading used in the experiments. The theoretical ultimate buckling stress of the axially loaded columns was obtained using the tangent-modulus formula:

$$f_{cr} = \frac{\pi^2 \cdot E_t}{\left(\frac{K \cdot L}{r}\right)^2}$$

where $K{\cdot}L/r$ is the slenderness ratio and E_t is the tangent modulus obtained by summing the respective tangent moduli of the steel and concrete. Hollow tube stub columns and short unconfined concrete cylinders were tested to obtain the tangent modulus-stress relationships used in the formulation. The authors presented a number of graphs illustrating their stress-strain results. The tangent modulus method produced generally accurate and conservative estimates of ultimate loads. The formula underestimated the strength for short columns, as expected. At small slenderness ratios, the concrete was able to reach high enough strains to allow expansion of the concrete and cause a corresponding lateral confinement from the steel, which increased the strength of the concrete. Also mentioned were additional methods of calculating the ultimate load. One method involves using the tangent modulus of the steel tube and assuming equal longitudinal strains in the concrete and steel. Using this measured longitudinal strain, the load in the concrete may be obtained from its stress-strain curve. This method tends to overestimate the capacity, however. A second approach is to use the tangent modulus obtained from CFT stub column tests. The authors did not recommend this because end conditions will affect the results. Friction at the ends will cause lateral pressure that is unaccounted for and the two materials may not be loaded equally. Therefore the authors recommended the method used in the paper.

Beam-Columns. Theoretical estimates of the bending capacity were determined by using a straight line interaction equation for the beam-columns:

$$\frac{P_u}{P_o} + \frac{M_u}{M_o} = 1$$

The value of P_o was obtained by testing an equivalent length column with no moment and M_o was obtained by a flexural test of a simply-supported CFT beam. P_u was the measured load at failure of the beam-column and M_u was calculated by multiplying P_u by the actual eccentricity (the sum of the initial applied eccentricity and the measured lateral deflection) at the ultimate load. The straight line interaction formula predicted a theoretical ultimate load equal to or less than the experimental for all of the tests except for the circular columns with large eccentricities (1.0 in. for these tests). A rigorous explanation of the overestimate was not given. The authors cautioned against using this straight line interaction formula for slender columns. They also felt it would be conservative for short columns.

Neogi, P. K. et al., 1969 (c, bc; m)

Introduction. The elasto-plastic behavior of pin-ended circular CFTs, loaded either concentrically or eccentrically about one axis, was studied numerically and then compared to experimental results performed by the authors and results published in other articles.

Experimental Study, Discussion, and Results. The main parameters of the test were the steel type, *L/D* ratio, and the eccentricity. All of the columns behaved in a ductile manner and no local buckling of the tubes occurred. The shape of the deformed columns remained symmetric throughout the test. The curvature was distributed uniformly over the length of the beam up to the maximum load. Beyond this point, the increase in curvature was concentrated at the midpoint of the section. As the load decreased, the moment at the center of the section continued to increase, probably due to the strength gain from the triaxial compression of the concrete. These results were important in comparing the analytical method.

Theoretical Discussion. As a CFT undergoes loading, the structural action changes as Poisson's ratio for concrete exceeds that of steel. Initially, for low strains, the steel expands at a faster rate (Poisson's ratio of 0.283 versus 0.15 to 0.25 for the concrete), thus providing no restraining effect on the concrete. Then at higher loads, the concrete expansion exceeds the steel expansion. The radial pressure induces hoop tension in the tube, changing the stress state from uniaxial to biaxial in the steel and from uniaxial to triaxial in the concrete. The steel transfers some of its longitudinal load to the concrete at this point since it cannot sustain as much load in the presence of a hoop stress. In this state of stress, the member provides strengths well in excess of the sum of the individual components, although shear failure may occur in the concrete before the load is completely transferred. The increase in failure load provided by the confinement depends on, among other things, the magnitude of strain at failure. This strength gain therefore varies inversely with length and eccentricity. All slender columns and shorter columns with high eccentricities will fail by buckling before large enough strains to transfer the load are reached. The authors suggested, then, that for practical columns which generally fall into this category, it is unnecessary to take triaxial effects into account.

Analytical Study. Two methods were used to obtain the load-deflection curve of axially loaded sections. In the analytical formulation it was assumed that complete interaction between the materials took place, each material was subjected to a uniaxial state of stress (concrete in tension was ignored), the stress-strain curves were fully reversible (i.e., immediate elastic unloading was not assumed), and no local buckling or shear failure occurred.

Axially Loaded Columns. The axially loaded CFTs were analyzed numerically using the tangent-modulus approach. The computer program the authors developed used an iterative technique to find a value of strain to equilibrate (within a set tolerance) the following equations:

$$P_{cr} = \frac{\pi^2}{L^2} \cdot \left(E_{st} \cdot I_s + E_{ct} \cdot I_c\right)$$

and

$$P_o = A_s \cdot f_y + A_c \cdot f_c^{'}$$

The first equation is the Euler buckling equation with the respective tangent moduli of the steel and concrete. The stress-strain curve for the steel was a trilinear curve with elastic, elasto-plastic, and perfectly-plastic regions. For some calculations, in order to avoid a sudden drop in stiffness at yield (intersection of the elastic and elasto-plastic lines on the curve) which will cause certain columns to buckle, the elasto-plastic region was modeled as a parabola rather than a straight line. The concrete stress-strain curve was based on a variation of Hognestad's equation.

Beam-Columns. The beam-columns were analyzed by two methods: by calculating the 'exact' deflected shape by Newmark's iterative method, and by assuming a deflected shape in the form of a partial-cosine wave. The first method involved subdividing the beam-column into a number of sections along the length and performing an incremental solution for the moments and curvatures until a preset tolerance was reached. By integrating the curvature over the length, deflections could be obtained and the load-deflection curve could be plotted. The latter method assumed that the deflected shape is part of a cosine wave which satisfies equilibrium only at mid-height. A similar iterative procedure was used to select values of deflection and neutral axis distance until convergence of the internal and external moments was achieved. Since the methods of the computational approach incremented central deformations and not axial load, they had the advantage of being able to trace the load-deflection curve into the post-buckling region.

Comparison of Results. The experimental and analytical results agreed for columns with *L/D* ratios of greater than 15, suggesting triaxial effects are negligible for such columns, since buckling occurs before the strains may reach a large enough value for confinement to occur. For columns with smaller *L/D* ratios, some strength gain was realized due to triaxial effects, rendering the calculated loads conservative. This effect diminishes, however, as the eccentricity increases. The partial-cosine wave estimate of the deflected shape was always conservative but never more than 5% below the exact shape calculation.

Knowles, R. B. and Park, R., 1970 (c; m)

Introduction. Design equations to compute the ultimate strength of CFT columns subjected to axial compression were presented in this paper. The design formulation was based on the tangent modulus approach as discussed in Knowles and Park (1969). The authors also derived a method of calculating the slenderness ratio below which some increase in concrete strength due to confinement is likely. The results of previous tests by other investigators as well as the authors' own tests were compared with the proposed design equations. The authors also discussed, at length, many of the proposed design formulations and pointed out some of the false assumptions embedded in these equations.

Theoretical Discussion. The tangent modulus theory proposed in the paper was justified by experimental evidence, namely the results of Furlong (1968) and the authors' own work from 1969. Both papers concluded that little bond exists between the concrete and steel. In the formulation of the design equations as discussed in the next section, the two materials were assumed to act independently. Therefore they will not have the same longitudinal strains. The existence of bond would instill error into this assumption.

Design Formulation. The design equations presented were based upon summing the separate tangent modulus buckling loads of the concrete core and the steel tube, which has been shown by the authors in a previous investigation to be an accurate prediction of ultimate strength. The tangent modulus of the respective materials may be obtained from the stress-strain curves determined experimentally. The authors derived elaborate formulas for the ultimate buckling strength of concrete and steel. The concrete equation was based on a stress/strain relationship expressed by the Hognestad parabola. The AISC formulation was used to determine the ultimate strength of a hollow tube. Summing the two equations resulted in the following expressions: 1) for steel tubes with a slenderness ratio, $K \cdot L/r_s < \sqrt{2 \cdot \pi^2 \cdot E_s / f_y}$:

$$P_o = 2 \cdot \left(0.85 \cdot f_c^{'}\right) \cdot A_c \cdot q \cdot \left(\sqrt{q^2+1} - q\right) + f_y \cdot A_s \cdot \left(1 - \frac{f_y \cdot (K \cdot L / r_s)^2}{4 \cdot \pi^2 \cdot E_s}\right),$$

and 2) for slender tubes governed exclusively by a buckling mode of failure, $K \cdot L/r_s > \sqrt{2 \cdot \pi^2 \cdot E_s / f_y}$:

$$P_o = 2 \cdot \left(0.85 \cdot f_c^{'}\right) \cdot A_c \cdot q \cdot \left(\sqrt{q^2+1} - q\right) + \frac{A_s \cdot \pi^2 \cdot E_s}{(K \cdot L / r_s)^2}$$

The c and s subscripts refer to concrete and steel, respectively, and

$$q = \frac{162.8 \cdot w^{1.5}}{\sqrt{f_c^{'}} \cdot (K \cdot L/r_c)^2},$$

where w is the weight of the concrete. The first term in both equations reflects the ultimate axial strength of the concrete and the second term reflects the strength of the steel. The authors also constructed an equation to determine the slenderness ratio of the concrete core above which an increase in concrete strength due to steel confinement is not likely to occur. From concrete cylinder tests they performed, the authors found that concrete showed a rapid volumetric expansion at an average longitudinal strain of 0.002 and an average stress of 0.954 of the maximum. These results agreed with those of other investigators. Based on the buckling formulas for each material, two equations were derived to express minimum slenderness ratios above which buckling will occur before the concrete becomes confined. Expressed in terms of concrete strains, an increase in the concrete strength due to confinement will not occur if:

$$\left(\frac{K \cdot L}{r_c}\right)^2 > \frac{\pi^2 \cdot \left(1 - \frac{\varepsilon_{vol}}{\varepsilon_o}\right)}{\varepsilon_{vol} \cdot \left(1 - \frac{1}{2} \cdot \frac{\varepsilon_{vol}}{\varepsilon_o}\right)},$$

where ε_{vol} is the strain when volumetric expansion occurs and ε_o is the strain at f'_c. In terms of steel stress-strain properties, an increase in the concrete strength will not occur if :

$$\left(\frac{K \cdot L}{r_s}\right)^2 > \pi^2 \cdot \left(\frac{E_{st}}{f_{su}}\right) \text{ at } \varepsilon_s = \varepsilon_{vol},$$

where f_{su} is the ultimate steel strength and E_{st} is the tangent modulus of the steel.

Comparison of Results. The design equation for the lower range of slenderness ratios was conservative for most experimental tests (17 conducted previously by the author and 100 from other investigators). The equation for the higher range was conservative as well, although it was only checked against two results. For square tubes, the equations were up to 12% unconservative suggesting the possible need of a reduction factor to account for the square shape of the concrete core. All of these results were summarized in a tabular format. A comparison of the equations proposed by others revealed that they do not give a better prediction of ultimate loads than the authors' design equations. The authors conducted a thorough discussion of these previous papers and presented all of their formulas.

Further Research. Other problems requiring further investigation were mentioned by the authors. These include examining the effects of various methods of load application at the ends of the specimen, beam-column and slab-column connections, creep at high loads, the effects of corrosion and fire, and the properties of low cost spiral welded steel pipe columns.

Chen, W. F. and Chen, C. H., 1973 (no tests)

Introduction. The elasto-plastic behavior of pin-ended, circular and square concrete-filled steel tubes was studied analytically. Three types of stress-strain relationships were investigated: 1) uniaxial state of stress in both the steel and concrete; 2) uniaxial stress state in the steel and triaxial stress state in the concrete, assuming the triaxial effect increases the concrete's ductility only; and 3) uniaxial steel stress state and triaxial concrete stress state, assuming the triaxial effect increases both ductility and strength in the concrete. Interaction curves relating axial force, end moment, and slenderness ratio were presented for the maximum load carrying capacity of a beam-column and compared to test results from other investigators. The three main objectives of the paper were to: 1) develop theoretical column curves for combined axial load and bending taking into account the effect of concrete confinement; 2) demonstrate that an axially loaded CFT column may be accurately predicted by assuming a certain amount of eccentricity in axial load application; and 3) present interaction curves for simply-supported CFT columns under unsymmetric loads.

Theoretical Discussion. The authors stated that all columns must be treated as beam-columns because imperfections always exist and loads are never applied concentrically. This implies analyzing columns as deflection problems rather than problems analyzed by the eigenvalue or tangent-modulus approach. The moment-curvature relationship for a constant axial load was developed first under the following assumptions: the concrete has no tensile strength; the steel has an elastic-perfectly plastic stress-strain relationship; plane sections remain plane in bending; and complete interaction between the steel and concrete exists. For each given axial load, three sets of moment-thrust-curvature (M-P-ϕ) curves were developed corresponding to the three aforementioned stress-strain relationships. The theoretical method used to obtain interaction curves was the Column Curvature Curve (CCC) method. Axially loaded members that were initially curved, eccentrically loaded, and contained residual stresses in the steel tube were considered. To account for these imperfections, the authors assumed an initial applied load eccentricity of $0.001 \cdot L$ for circular sections and 0.002L for square sections. The theoretical curves for axially loaded columns showed that triaxial effects were negligible for values of $L/D > 15$ for circular columns and $L/D > 20$ for square columns. Test results obtained by Knowles and Park were compared to the theoretical values and good correlation existed for ratios of $L/D > 15$, especially for large eccentricities. Columns with L/D ratios less than 15 were influenced by the effect of triaxial confinement of the concrete. An accurate prediction of the ultimate axial load for square columns using the CCC method was obtained by using the concrete stress-strain curve assuming triaxial stresses with an increase in both strength and ductility. Because the triaxial effects were much greater for the circular columns than the square columns, the theoretical estimates for the circular columns were quite conservative for low L/D ratios. The circular columns were more accurately modeled by amplifying the value of f'_c used in the initial stress-strain relationship. For columns with combined axial load and moment, the authors found that the unconfined stress-strain curve for concrete was adequate for columns with $L/D > 15$ under both symmetric and unsymmetric loading. For columns with $L/D < 15$, the effects of column instability may be neglected and the confined stress-strain curve of the concrete should be used.

Tomii, M. et al., 1973 (no tests)

Introduction. This paper presented a very thorough review of the knowledge of CFT behavior to date. Topics of discussion included axially loaded short and long columns, the behavior and design of beam-columns subjected to combined axial load, bending moment, and shearing force, and connection types and their behavior.

A number of advantages of CFTs were cited in the paper's introductory section: the load-carrying capacity of the member is increased without increasing the member size; the tube provides ideally-placed reinforcement; thinner steel tubes may be used since the concrete core forces all local buckling modes outward, delaying the onset of local buckling; the tube prevents concrete spalling which increases member ductility; in precast members, the tube protects the concrete during shipment; the tube serves as the concrete formwork in construction; and CFTs will have a higher fire resistance and require less fireproofing than hollow tubes because the concrete has a larger thermal capacity than air.

Theoretical Discussion. *Short Axially Loaded Columns*. The shape of the CFT (i.e., circular or rectangular) will affect the axial behavior of the CFT. The relationship between axial load and longitudinal strain illustrates the marked decrease in capacity in the inelastic region for rectangular tubes. The decrease is not, however, evident for circular tubes due to the greater degree of confinement. Also, the thinner the tube and the higher the concrete strength, the steeper the descending branch of the axial load-longitudinal strain curve. Most importantly, though, the CFT displays a greater ductility and strength than the sum of the individual ductilities and strengths of the constituent materials due to the interaction between the steel and the concrete.

The authors described short CFT column behavior as occurring in two phases. The first phase is the elastic phase, in which the Poisson's ratio of concrete (0.16 to 0.25) is lower than that for steel (0.3). Provided the bond between the concrete and steel does not break down, the concrete's smaller lateral expansion will pull the steel inward, inducing compressive hoop stresses in the tube and tensile lateral stresses in the concrete. The behavior in this region is similar to the sum of the individual behaviors of the constituent materials and the authors suggested that modulus of elasticity of the CFT may be dervied from uniaxial stress conditions.

In the second phase of loading, the longitudinal strain reaches a point at which the concrete undergoes volumetric expansion and its lateral expansion exceeds the lateral expansion of the steel. This occurs at a longitudinal strain of approximately 0.002. At this point in the loading, the concrete will be triaxially confined in compression and tensile hoop stresses will exist in the steel tube. The authors suggested that both the concrete's strength and ductility will increase for both circular and rectangular CFTs due to the confinement. The augmented concrete strength was calculated for circular tubes as follows:

$$f_{cc} = f_c^{'} + 4.1 \cdot \sigma_r$$

where σ_r is the lateral pressure exerted on the concrete. This value is calculated by:

$$\sigma_r = \sigma_{sc} \cdot \left(\frac{t}{D/2 - t} \right)$$

where σ_{sc} is the hoop stress in the steel tube. Based on failure theory, for an existing hoop stress in the inelastic range, the longitudinal stress, σ_{sl}, will be less than f_y. The reduction in the yield strength of the steel tube in the presence of a hoop stress is, however, offset by the increase in the strength of the concrete. The modified calculation of the ultimate axial strength accounting for these effects is:

$$P_o = f_{cc} \cdot A_c + \sigma_{sl} \cdot A_s$$

Assuming no lateral interaction occurs, the ultimate axial load may be conservatively calculated by:

$$P_o = f_c^{'} \cdot A_c + f_y \cdot A_s$$

Long Axially Loaded Columns. The authors stated that the effects of confinement do not affect the behavior of long, or slender, columns, which fail by buckling before longitudinal strains become large enough for interaction between the material sto take place. Two types of buckling may occur: slender columns will fail by elastic buckling; and intermediate length columns will fail by inelastic buckling.

The failure load for a CFT column failing by elastic buckling may be computed by the following formula:

$$P_{cr} = \frac{\pi^2 \cdot (E_s \cdot I_s + E_c \cdot I_c)}{(K \cdot L)^2}$$

The critical buckling load, P_{cr}, is calculated using a stiffnes that is simply the sum of the individual stiffnesses of the concrete and steel. The above formula may be modified for CFT columns failing by inelastic buckling by replacing the respective elastic moduli with tangent moduli. The resulting equation is:

$$P_{cr} = \frac{\pi^2 \cdot (E_{st} \cdot I_s + E_{ct} \cdot I_c)}{(K \cdot L)^2}$$

A second, similar method of computing the ultimate load for inelastic columns assumes that the concrete core and the steel tube act as independent columns. The buckling load is calculated by summing the individual tangent modulus loads of the concrete core and the steel tube. Both of these methods require that the stress-strain relationships of the materials by known. These properties are often obtained from stub column tests. The tangent modulus load of the steel tube has been typically calculated using the AISC design code and the concrete core tangent modulus load has been typically derived using Hognestad's parabolic stress-strain curve.

Beam-Columns. CFT members subjected to a combination of axial load and bending moment sustain large deformations without spalling of the concrete or local buckling of the steel. For CFTs with an *L/D* ratio larger than 15, the load-deflection relationship may be accurately calculated using the uniaxial stress-strain relationships of the steel and the concrete. The member may show increased strength and ductility due to confinement, however, as the axial load increases. The effects of confinement in these cases will be larger in circular CFTs than in rectangular CFTs.

The authors proposed the following formulas to compute the initial composite stiffness of a CFT beam-column, which is used in the analysis and design of beam-columns. For an applied axial load P, $0 \le P \le 0.5 \cdot P_o$,

$$E \cdot I = E_s \cdot I_s + E_c \cdot I_c$$

And for $0.5 \cdot P_o \le P \le P_o$, the above equation is modified as follows:

$$E \cdot I = \left(E_s \cdot I_s + E_c \cdot I_c\right) \cdot 4 \cdot \frac{P}{P_o} \cdot \left(1 - \frac{P}{P_o}\right)$$

where the modulus of elasticity of the concrete, Ec, is calculated by:

$$E_c = 33 \cdot w^{1.5} \cdot \sqrt{f_c^{'}} \text{ (psi)}$$

To analyze the overall behavior of a CFT beam-column, i.e., in a moment-curvature analysis, the effects of confinement should be considered. The ultimate bending strength of a CFT beam-column, however, may be accurately calculated using a modified stress block computation such as the ACI method for reinforced concrete members.

To account for the complicated effects of slenderness in the design of CFT beam-columns, approximate methods are often used. The authors discussed the method in the Architectural Institute of Japan (AIJ) code. The maximum slenderness ratio, λ, must not exceed 100. For λ, $50 \leq \lambda \leq 100$, the working loads are multiplied by a factor, φ_1:

$$\varphi_l = \frac{100}{150 - \lambda}$$

The slenderness ratio, λ, is calculated as the unsupported length of the beam column divided by the effective minimum radius of gyration of the section. For circular members, the radius of gyration is taken as 0.25·D; for rectangular, $0.29 \cdot D$.

Beam-Columns Subjected to Shearing Force. CFT beam-columns demonstrate excellent shear resistance. For low axial loads, the shear capacity of a CFT will increase with increasing load due to the strain hardening of the steel. Additionally, the authors stated that CFT beam-columns do not fail in shear even under high axial loads. Experiments have shown that in the presence of a moment gradient, the ductility and ultimate strength of the section increase. The authors offerred no explanation of this phenomenon. To calculate the initial composite stiffness for shearing force, the authors proposed the following equation:

$$G \cdot A = \frac{G_c \cdot A_c}{1.2} + G_s \cdot A_{sw}$$

Further Research. The authors listed several additional topics that require further investigation. These include: investigating the elasto-plastic behavior of CFTs by examining the stress-strain characteristics of confined concrete and biaxially-stressed steel; studying the effect of end conditions and load transfer on the behavior of CFT members; investigating the hysteretic behavior of CFTs subjected to alternately repeated loading; and, finally, investigating the behavior of frames subjected to lateral force.

<u>Bode, H., 1976</u> (no tests)

<u>Introduction</u>. This paper presented a brief overview of analytical methods for calculating the ultimate capacity of short and long columns, and columns subjected to combined axial load and bending moment. The author also discussed the long-term performance of concrete in CFT columns (i.e., creep and shrinkage). He enumerated a number of advantages of filling hollow steel tubes with concrete. These include: a higher critical buckling load due to the stiffening effect of the concrete; greater load-bearing capacity for the given external dimensions; higher resistance to transverse impact loads (e.g., vehicles); ideal use of relatively inexpensive concrete as a compression load-resisting element; simple connections capable of standardization; and the steel tube serving as the concrete formwork.

<u>Theoretical Discussion</u>. *Short Axially Loaded Columns*. The steel and the concrete in axially loaded short circular columns act independently until the concrete has reached a longitudinal strain of approximately 0.02. At this point, the concrete expands more than the steel tube. The steel is subjected to circumferential tensile stresses and the concrete is compressed triaxially resulting in a much larger strength than concrete loaded only in the axial direction. The increase in the capacity of the short column (the author defines a short circular column as a column with $L/D \leq 5$) was given by the following formula originally published by Sen, 1972:

$$P_o = 0.75 \cdot A_s \cdot f_y + A_c \cdot \left(f_c^{'} + \frac{3.8 \cdot t \cdot f_y}{D - 2 \cdot t} \right)$$

The carrying capacity of a short circular column is also increased if the concrete alone is loaded. To prevent the steel from buckling locally prior to the steel reaching the yield strength, the ACI Building Code 318-71 specifies a minimum ratio of the diameter to thickness of the steel tube:

$$\frac{D}{t} \leq \sqrt{\frac{8 \cdot E_s}{f_y}} .$$

For short rectangular CFT columns ($L/D \leq 10$), the ultimate axial load is equal to the sum of the individual material strengths:

$$P_o = A_s \cdot f_y + A_c \cdot f_c^{'}$$

The ratio of the tube width to the tube thickness is also limited by the ACI formula:

$$\frac{D}{t} \leq \sqrt{\frac{3 \cdot E_s}{f_y}}$$

Long Axially Loaded Columns and Beam-Columns. The author listed a number of characteristics of long, or slender, CFT columns that prohibit a direct calculation of the ultimate load: the materials do not conform to a linear stress-strain relationship; lateral buckling must be accounted for in the equilibration of forces; the steel may be plastic and elastic at different points on the cross-section; the concrete may be cracked; imperfections and residual stresses in the tube may exist; and the effects of creep and shrinkage of the concrete are uncertain. In light of this, the author very briefly described a fiber element method of analysis which has been shown to be quite accurate compared with experimental results. This method involves assigning stress-strain relations (specified by the German code DIN 1045) to fibers on the discretized cross-section. It was assumed that Bernoulli's law applies (i.e., plane sections remain plane for any amount of longitudinal strain). The ultimate combination of axial load and bending moment was obtained using this process for several load cases. Examples of the interaction between axial force and bending moment for different flexural stiffnesses and for different *L/D* ratios was illustrated for circular sections.

For purposes of design, the author restated an empirical experiment-calibrated equation proposed by Basu and Somerville to calculate the interaction between axial load and bending moment and proposed his own equation based upon the results of his computer analysis.

Biaxially Loaded Beam-Columns. For biaxially loaded beam-columns Basu and Somerville extended to rectangular CFTs the formula originally proposed by Bresler for reinforced concrete columns. The formula is a conservative calculation of the ultimate capacity of a section for a load applied with eccentricity about two axes:

$$\frac{1}{P_{xy}} = \frac{1}{P_x} + \frac{1}{P_y} - \frac{1}{P_{cr}}$$

where P_{cr} is the ultimate buckling load about the major axis, and P_x and P_y are the ultimate axial capacity of the section for eccentricities, e_x and e_y.

Long-Term Concrete Performance. The phenomena of creep and shrinkage are less severe in CFTs than in corresponding reinforced columns. Creep of the concrete may reduce the ultimate capacity of the section because it induces additional stress in the steel tube. This may be accounted for in design by adding the anticipated creep load to the dead load. Additionally, creep will reduce the overall stiffness of the CFT. This effect may be accounted for by using a reduced concrete elastic modulus. Shrinkage of concrete inside a steel tube is considerably less than the shrinkage of concrete exposed to the environment. The conditions inside the tube are more humid and shrinkage proceeds at a slower rate. However, care must be exercised if bond is assumed to exist between the steel and the concrete since shrinkage of the concrete may cause a breakdown in the steel/concrete bond.

<u>Bridge, R. Q., 1976</u> (bc; m)

<u>*Introduction*</u>. The results of a theoretical and experimental investigation of square, pin-ended, eccentrically loaded CFT columns were presented. The principal variables of the eight tests were the eccentricity of loading, the slenderness ratio, and the inclination of the loading axis (i.e., the angle of the applied load relative to the principal axis of the member). The author presented a very detailed description of the results.

<u>*Experimental Study, Discussion, and Results*</u>. *Influence of the Concrete Core*. Under axial loading, the concrete core provided about 30% of the load carrying capacity while under pure bending it only provided 7.5%. If no local buckling occurs, the moment capacity of the

hollow tube is adequate for calculating the design load for pure bending (as was proposed by Furlong, 1967). The contribution of the concrete is enhanced by axial force. For increasing axial load, the concrete provides an increasing proportion of the moment capacity. Since the steel (which has a higher elastic modulus than the core) is located on the outer perimeter of a CFT, it represents a larger portion of the member stiffness. Therefore, for low axial loads, the hollow tube undergoes only 25% more deflection due to bending than the equivalent CFT. While it is widely recognized that the triaxial effect plays a role in strengthening short circular columns, it was not evident for the tested square columns, echoing earlier investigator's conclusions.

Bond. The author noticed no evidence of slip in the tested specimens, but he drew no definitive conclusions in this regard. The analytical method assumed perfect bond. Since the results were very close to the experimental, this suggested that bond was maintained in the member. If bond did not exist, the load-carrying capacity of the section was not significantly affected.

Ductility. The author's tests reinforced earlier investigative reports of the CFT's ability to withstand large deformations. In the unloading portion of the experiment the CFTs were able to maintain a high proportion of their maximum loads even in a state of large deformation. This was largely due in part to the stabilizing role of the concrete in preventing early local buckling of the tube.

Biaxial Bending. Bending tests were performed with the moment applied at three axes of inclination with respect to the perpendicular faces of the square cross section: 0°, 30°, and 45°. The orientation of the tube's axes with respect to the bending direction had little effect on the amount of moment the member could withstand. The columns deflected in the plane of the applied moment and displayed no twisting due to their very high torsional rigidity.

Stability. For slenderness ratios above 45, the maximum axial load was reached before the member realized its full moment capacity. Columns with such large slenderness ratios do not reach their full cross-sectional strength before the member fails by overall flexural buckling.

Analytical Study. Using moment-thrust-curvature relationships, an elastic-plastic column stability analysis was performed. Simplified stress-strain relationships were used for the steel and concrete. The procedure was based on the method developed by Roderick and Rogers. Limited details about the procedure were given. The analytical predictions matched the experimental deflections exactly up to the yield point of the specimens. After yielding there was some discrepancy. The author attributed these variations to a number of possibilities: errors in determining the steel tube yield stress, variation of yield across the cross section, a different concrete strength than assumed, or a residual stress pattern that was not accurately simulated by the stress-strain curve.

Further Research. The results of the bond tests were inconclusive and the author recommended further study of this phenomenon. He also questioned the applicability of pin-ended, single column results to an entire building frame where entirely different loading conditions could exist. Experimental tests regarding the behavior of connections and the interaction between structural members was a high recommendation of the author. At the paper's writing, investigation into the latter problems was being conducted at the University of Sydney.

<u>Council on Tall Buildings and Urban Habitat, 1979</u> (no tests)

<u>Introduction</u>. A comprehensive summary of CFT behavior and corresponding design procedure was presented. Several references were made to previous papers. Much of what this article described can be found in other summaries contained in this report.

<u>CFT Behavior</u>. *Short Axially Loaded Columns*. The authors describe the relationship between the Poisson ratio of concrete and that of steel as the strains in the CFT section increase. At longitudinal strains of 0.001, the concrete begins to expand at a more rapid rate than the steel tube. Knowles and Park (1969), suggested that at a strain of 0.002 (about 0.95 of the maximum load), contact between the materials is initiated and interactive stresses between the concrete and the steel result. The concrete will be triaxially confined and the steel will be in a state of biaxial stress, and thus unable to sustain the initial uniaxial yield stress in the longitudinal direction because of the secondary hoop stress. At failure, experiments have shown that the longitudinal stress is about three-fourths of the uniaxial yield stress and the hoop stress is about one-half of the uniaxial yield stress. Triaxially confined concrete may attain strengths double the unconfined strength in circular sections, although little strength gain has been recognized in rectangular specimens.

Failure of short CFT specimens was described as a primary result of local buckling of the tube in regions of maximum strain. The tube failure is accompanied by either concrete crushing or a diagonal shear fracture in the concrete. The shear failure is initiated in the concrete, and the concrete slides along an inclined shear plane of approximately 64° (Sen, 1969).

Long Axially Loaded Columns. Concrete-filled steel tube columns exhibit much the same behavior as any structural compression member, showing a decrease in strength with an increase in length. Additionally, strength gain due to confinement decreases rapidly with an increase in slenderness. Knowles and Park (1969), computed a $K{\cdot}L/r_c$ value of 44.3 as the point beyond which no beneficial confinement action takes place. However, even when no confinement action occurs, the confined core has the advantage of guaranteeing ductile behavior for longer columns. Very long columns, which fail by elastic buckling, do not often exhibit ductile behavior. Elastic buckling occurs at a larger slenderness ratio for lower yield strength steels and for columns with a higher percentage of concrete in the cross section.

Combined Compression, Bending, and Shear. The moment-curvature behavior of a CFT section is similar to a plain thick-walled steel tube, with a ductility at least as great as the hollow tube. As the tube begins to yield, the slope of the moment-curvature curve decreases until the point of local buckling of the steel tube. The concrete at the location of buckling will be in a crushed and disintegrated state. The flexural capacity of the section corresponds to the incipient point of tensile failure in the concrete. The addition of concrete to a steel tube greatly increases the shear capacity of the section. Tomii et al. (1972) indicated that the shear strength is the sum of the individual strengths of the concrete and steel. The authors recommended using the sum of the concrete and steel shear rigidities to compute the rigidity of the concrete-filled tube:

$$\left(G \cdot A\right)_{comp} = G_c \cdot A_c + G_s \cdot A_s$$

It has been shown that CFTs possess a greater shear strength for a given section size than reinforced concrete members. CFT columns subjected to shear demonstrate the ability to sustain shear forces as transverse deflections increase, even under constant axial load.

Cyclic Compression, Bending, and Shear. CFTs under alternately repeated loading have displayed good deformation capacity and spindle-like hysteresis loops typical of stocky steel sections. Tests of specimens with a low *D/t* ratio have been conducted with no evidence of buckling or concrete crushing (probably due to confinement). Tests of thin-walled specimens with accompanying shear, however, have displayed local buckling before sufficient rotation capacity is attained. To compute the shear rigidity of the composite section under cyclic loading, the authors added a reduction factor to the concrete term of the monotonic formula:

$$(G \cdot A)_{comp} = \frac{G_c \cdot A_c}{1.2} + G_s \cdot A_s .$$

<u>SSRC, Task Group 20, 1979</u> (no tests)

<u>*Introduction*</u>. This paper proposed design equations for composite members (both concrete-filled steel tubes and concrete-encased steel shapes). Short and slender columns under pure axial load and beam-columns under combined axial load and bending were examined. Included were a number of design equations and an appendix documenting previous tests and comparing them to the allowable loads formulas proposed.

<u>*Design Formulation*</u>. *Short Axially Loaded Columns*. SSRC used a conservative approach to determine the amount of axial capacity for a given CFT section. The modified allowable stress for CFT columns was expressed as:

$$\mathrm{f_{my}} = f_y + 0.85 \cdot f_c^{'} \cdot \left(\frac{A_c}{A_s}\right)$$

with $f_y < 55$ ksi to prevent the concrete from crushing before the steel yields. Fifty-five ksi corresponds to an axial strain of 0.0018, the strain at which concrete fails in compression. Using a steel yield stress below 55 ensures that the steel will yield at a smaller strain than 0.0018, producing a desirable ductile failure. The above equation superimposes the individual strengths of the concrete and the steel tube, but it does not account for any confinement effect of the steel tube on the concrete, which is a conservative approach. Limits were placed on the *D/t* ratio such that the section will not buckle locally before the yield strength is reached. For rectangular sections this *D/t* limit was established as

$$\frac{D}{t} \le \sqrt{\frac{3 \cdot E}{f_y}}$$

and for circular sections:

$$\frac{D}{t} \le \sqrt{\frac{8 \cdot E}{f_y}}$$

The SSRC specification also requires that only concrete strengths between 3 ksi and 8 ksi be used.

Slender Axially Loaded Columns. The slenderness of a column is a function of its modulus of elasticity, *E*, its moment of inertia, I, and its length, L. While these values are easily obtained for a steel section, the task is not straightforward for CFTs. Concrete is an inhomogeneous material and its *E* varies with sustained loads. Also, tensile cracking significantly reduces the effective stiffness of the concrete, even when it is confined within a tube. To account for this, the SSRC council imposed a reduction factor of 0.4 on the initial stiffness of the concrete. This accounts for both creep and tensile cracking. They expressed the total modified modulus of elasticity of the concrete-filled steel tube as:

$$E = 29000 + 0.4 \cdot E_c \cdot \left(\frac{A_c}{A_s} \right)$$

The modified values of E may be incorporated into column curves, which reflect the relationship between thrust capacity and column slenderness.

Beams and Beam-Columns. Axial capacity in a column is greatest in the absence of any bending moment. Likewise, a member's greatest moment capacity occurs without any axial load applied. In this case, the SSRC recommends using only the moment capacity of the steel tube in calculating the member's resistance:

$$M_o = S_s \cdot f_y$$

where S_s is the section modulus of the steel tube alone. For beam-columns, the allowable loads are expressed in terms of an interaction equation which requires that the sum of the axial stress ratio squared and the stress ratios due to bending in the *x* and *y* directions be less than 1. To account for secondary moments, the stress ratios due to bending were modified by a moment magnifier as in the AISC Specification.

Comparison of Results. A number of CFT tests were compared to the design formulas for both types of loading. The average value of the ratio between test load and allowable load was an acceptable 2.26 for axially loaded CFTs and varied from 1.90 to 2.50 for the eccentrically loaded beam-columns.

Tomii, M. and Sakino, K., 1979a, 1979b (c, bc; m)

Introduction. In this two-part paper, tests were conducted to determine the moment-thrust-curvature relationships of square concrete-filled steel tubes. Under a constant axial load, moment was applied to the section in uniformly increasing amounts. In addition to the very detailed experimental study, the authors proposed analytical equations to estimate the ultimate moment of a section. In the second part of the paper, analytical moment-thrust-curvature relationships were developed and compared to the experimental results.

Experimental Study, Discussion, and Results. Thirty-six specimens, all 4 inches square and approximately 12 inches in length, were tested. Four different series of specimens were tested, with each series containing 2 pure axial load tests and 7 tests with combined axial load and moment. The tube thickness and the material properties were varied from series to series and within each series the axial load was varied. A constant axial load was applied, followed by a monotonically increasing moment, which was applied by constraining the beam-column at the third points and jacking the ends of the specimen. This introduced two types of moments in the section: one due to the bending induced by the end jacks and an additional moment due to the axial load acting on the deflected shape (P-δ effect).

The concentrically loaded columns produced results in excess of those calculated analytically (see following section). The authors suggested that the attainment of higher strengths may have occurred because the concrete was cured within the steel tube versus within a mold.

Columns subjected to both axial load and bending moment displayed behavior that was highly dependent on the *D/t* ratio and the magnitude of the axial load, especially in the inelastic range of strains. Detailed moment-curvature-thrust plots were provided showing an increase in ductility with a decrease in *D/t*. Specimens with a *D/t* ratio of 24 behaved in a ductile manner, showing no falling branch of the moment-curvature curves. However, columns with a *D/t* ratio of 44 subjected to high axial load and moment behaved in a brittle manner due to the high axial load and the decreased confinement. But even though these columns failed in a rather brittle manner, the tension side of the steel tube still yielded. It was suggested that the ductility increase as the *D/t* ratio decreased was due largely in part to the lateral containment of the concrete by the steel.

Analytical Study, Results, and Discussion. The squash load calculated by the authors was as published by several other investigators:

$$P_o = A_s \cdot f_y + A_c \cdot f_c'$$

To calculate the ultimate moment of a beam-column subjected to combined loading, the authors proposed a simple method applicable to specimens with an axial load P of less than $0.5 \cdot P_o$. Tensile and compressive stresses in the cross section are represented by uniform rectangular blocks. By equating tensile and compressive forces, the neutral axis of the section may be found. Knowing this value, the ultimate moment may be computed. Equations expressing these results were presented in the paper. Also, as mentioned above, the axial load must be limited to a value such that the tension will yield in tension before the ultimate moment is reached. Otherwise, the steel tube stress block assumption is invalidated. The analytical method overestimated the strength for experimental sections with P/P_o of 0.58, therefore the authors set an axial load ratio limit of 0.5 for their analytical studies.

Moment-thrust-curvature relationships were developed under the following assumptions: there is no slip between the steel and the concrete and plane sections remain plane in bending; each longitudinal steel and concrete fiber follows an idealized stress-strain behavior (described below); stress-strain relationships are reversible; and the tensile strength of concrete is neglected. An elastic perfectly plastic stress-strain curve was assumed for the steel in compression and linear strain hardening was added in the tensile region. Two curves were used for the concrete -- a confined and an unconfined curve. Hognestad's parabola was used for the ascending branch of both curves. A flat portion followed by a descending portion dependent on the *D/t* ratio was used for confined concrete. Beyond a strain of 0.015, the confined concrete was assumed to sustain its strength ad infinitum. No provisions were made for an increase in strength due to confinement. The unconfined curve decreased from the maximum stress at a strain of 0.002 to zero stress at a strain of 0.016. The stress-strain curve for confined concrete gave good results for all tests, and gave much better results than the unconfined curve for tests with a high axial load. The moment-curvature curves were divided into three regions based on calculated values of M and ϕ: 1) elastic region, 2) primary plastic region, and 3) secondary plastic region. The ultimate moment was determined as described above. The yield moment was determined from the ultimate moment by assuming a shape factor. The respective ultimate and yield curvatures were obtained by the equation

$$\phi = \frac{M}{E \cdot I}$$

The remaining parameters defining the curve were determined via empirical curve-fitting based on the *D/t* ratio and the aforementioned moments and curvatures.

Tomii, M. and Sakino, K., 1979c (bc, sh; m)

Introduction. The objectives of this paper are twofold--1) to clarify the elasto-plastic behavior of CFT columns subjected to combined axial and shearing forces, and 2) to present a practical analytical model to predict the behavior of such columns.

Experimental Study, Discussion, and Results. Forty tests were conducted in five series. Each series contained different material properties and tube thickness (*D/t*). The variable parameters within each series were the shear span ratio (*a/D*) and the axial load ratio (P/P_o). The tubes were annealed to remove residual stresses. The test setup simulated fixed end conditions and induced a double-curvature deformation in the specimen.

The paper includes numerous *V-R* curves, where *R* is the translation angle, measured as the midheight deflection divided by the length. Each curve included the point of observed local buckling, which typically occurred at or near the peak shear stress, but well in advance of the maximum rotation. Two types of behavior were recognized based on the *a/D* ratio. For a shear span ratio of 0.83 or 1.0, diagonal shear cracking was observed in all specimens except the one without an axial load. The column with a high *D/t* ratio behaved in a brittle manner under a high axial load. For a shear span ratio of 2.0 or 3.0, all of the columns failed in flexure.

Analytical Study. The relationship between shear force and rotation was determined following the same assumptions proposed in Tomii and Sakino (1979a). Additionally, the fracture zone was assumed to occur in a region at the member end with a length of $0.6 \cdot D$. Shearing deformation is accounted for by using an effective shear modulus of the concrete and steel. Additional moments due to the P-Δ effect (end offset) were considered using a Column Deflection Curve method, which was not discussed in any detail. Also, the descending branch of the concrete stress-strain diagram was adjusted to account for the additional ductility provided by the presence of a moment gradient. The analytical results agreed well with the experimental except for the shear span ratio of 2.0. The authors believe the discrepancy lies in the perfect bond assumption and an overestimate of the shearing rigidity of the section.

Sakino, K. and Tomii, M., 1981 (bc, sh; r)

Introduction. The experiments contained in this paper were performed to examine the hysteretic behavior of square CFT beam-columns. Fifteen specimens were subjected to a constant axial load and a cyclic shearing force. The main test parameters were the *D/t* ratio, the axial load ratio, and the shear span ratio. Detailed hysteretic plots describe the effect of varying each parameter.

Experimental Study, Results, and Discussion. Essentially the same materials and the same apparatus were used in these tests as those in the experiments described in Tomii and Sakino (1979b). The difference in these tests was the nature of the loading. Each specimen underwent 3 cycles of lateral loading at increasing increments of displacement.

A number of conclusions were drawn based on the results. An increase in the *D/t* ratio or an increase in the axial load ratio brought on a decrease in the shear resistance (moment capacity) of the section at the unloading point. The *a/D* ratio had only a negligible effect. For specimens with a high axial load ratio ($P/P_o = 0.5$), after a certain amount of decrease in the shear resistance, the hysteretic loops tended to stabilize and even showed a slight increase in shear resistance. The reason for this was the behavior of the tube at the ends of the section (critical regions). The square tube began to form a circular shape as it underwent successive local buckling at the critical regions. This transformation in shape effectively increased the amount of confinement of the concrete and resulted in the stabilization of the hysteretic loops. There was also a considerable amount of axial shortening observed for columns with a P/P_o of 0.5 due to the combination of steel local buckling and concrete crushing. Values of axial shortening ranging from 27% to 34% of the section depth were measured. Accompanying the shortening and considerable bulging near the end of the sections subjected to a high P/P_o ratio, a second wave of local buckling formed on the tube closer to the midpoint of the section. Finally, the ultimate moment was 1.0 to 1.2 times the value calculated by the method described in Tomii and Sakino (1979a, 1979b). The authors suggested that this was due to a combination of strain hardening in the steel tube and moment gradient effects in the confined concrete at the critical section.

Sakino, K. and Ishibashi, H., 1985 (bc, sh; m, r)

Introduction. This study expanded upon Sakino and Tomii (1981) by examining short beam-columns under shearing forces. Nine specimens with an *a/D* ratio of 1.0 were subjected to cyclic shearing combined with a constant axial load. Twelve specimens with an *a/D* ratio of 1.5 were tested monotonically with combined shear and axial loads. An effort was made to study the failure mechanism of such beam-columns. The paper closed with an analytical formulation to determine the ultimate strength of short beam-columns.

Experimental Study, Results, and Discussion. The procedure and test setup were nearly identical to those described in Sakino and Tomii (1981) and Tomii and Sakino (1979b). The monotonic tests with an *a/D* ratio of 1.5 displayed a crack pattern indicative of a flexure failure with plastic hinges forming at the ends. Previous monotonic tests with *a/D* ratios of 1.0 (Tomii and Sakino, 1979b) displayed the characteristic diagonal concrete cracking of a shear failure. The cyclically loaded specimens with an *a/D* ratio of 1.0 failed in shear, as opposed to the cyclically loaded specimens with *a/D* ratios of 2.0 and 3.0, which failed in flexure (Sakino and Tomii, 1981). (The authors define a "shear failure" as a failure in which the concrete fails in shear, contrary to Wakabayashi et al. (1976) and Kato et al. (1978) who consider this failure "flexural" since both flanges of the steel tube yielded at ultimate). These beam-columns showed considerable energy absorption and displayed less strength deterioration than the longer columns that failed in flexure. Like the longer specimens, the beam-columns tested here showed an initial decrease in capacity and then a slight increase as local buckling in the critical regions transformed the shape of the tube from rectangular to circular. Also, specimens with larger *D/t* ratios showed a larger decrease in shear resistance with successive cycles of loading.

Analytical Study, Discussion, and Results. The analytical method proposed in this paper to compute the ultimate strength of a CFT beam-column in shear was based on the formulations by Wakabayashi et al. (1981) and Kato et al. (1978). The axial load is computed by summing the axial load on the concrete and the steel individually. The ultimate shear force is likewise computed by a summation of the ultimate concrete strength in shear and the ultimate steel strength in shear. The details of the individual strength calculations are given in the paper. Resulting from these computations are graphs of axial load ratio (P/P_o) versus the shear strength ratio (V/V_{max}). The analytical method presented agreed well for beam-columns with *a/D* ratios of less than 1.5.

Matsui, C., 1985, and Matsui, C., 1986 (fr; r)

Introduction. In these two papers, the behavior of one-bay frame structures composed of concrete-filled and hollow tube beam-columns spanned by steel beams are studied. The first paper presents preliminary results on 4 CFT frame specimens, whereas the second paper presents a more detailed set of results for the full set of 12 specimens. The papers investigate overall frame behavior and the behavior of the connections between the steel beam and the hollow tube or CFT beam-columns; connection design equations are presented based upon the experimental results. A new value for the limiting width-thickness ratio (*D/t* ratio) of CFTs is also proposed based upon the performance of the frames and an examination of the post-buckling behavior of CFTs using a plastic limit analysis.

Connection Zones. The author suggests that the design of beam to column connections is crucial to the ductile behavior of frames which are composed of CFT columns and steel I-beams. The details for the connection of steel beams to CFTs presented are of two general types: outside stiffener and through stiffener. The outside stiffener connection consists of top and bottom continuous plates which are welded to the outside of the CFT and rigidly connect to the top and bottom flanges of the I-beams. The inside stiffeners are continuos plates (generally with holes which allow concrete to flow through the joint) which penetrate the joint. These connections require field welding of adjacent CFTs. Both types of connection allow the designer to control the strength and behavior of the panel zone.

Experimental Study, Results, and Discussion. Twelve frame specimens were tested both monotonically and cyclically. The loading frame consisted of a hydraulic press which applied axial load to the columns through a spreader beam, and a hydraulic jack which applied horizontal loading under deflection control. The horizontal deflection at various points in the column height, as well as strains, were recorded. The behavior of the connections was monitored closely as was local buckling and post-buckling behavior of the tubes.

The *D/t* ratio of the steel tubes ranged from 33 to 68. The connections were designed such that the frame specimens with beam-columns having a *D/t* equal to 33 would exhibit a connection failure and frame specimens with a *D/t* ratio of 68 (weaker beam-columns) would fail at the base of the beam-columns. The frames composed of CFT columns demonstrated a higher strength (in comparison with predicted plastic limit analysis--see below), and, more importantly, much better post-local buckling behavior than the frames composed of HT columns. The CFTs were able to withstand a larger portion of their ultimate strength after local buckling of the steel tube.

The author cited two main reasons for the improved CFT behavior. First, the infilled concrete results in an outward buckling of the steel tube, rather than an inward buckling, as in hollow tubes. The outward buckling collapse mechanism differs from the inward mechanism, requiring more energy to result in a failure (Matsui, 1985). The second reason for the CFTs' exemplary behavior occurs in the post-buckling region. When the steel buckles, the axial load is transferred to the concrete and the section can maintain a significant portion of its pre-buckling capacity. The cyclically-loaded frame specimens maintained their strength for several cycles showing strength deterioration only after cracking and fracturing of the welded portion of the stiffeners in the connection-failing specimens and extensive local buckling in the column-failing structures.

Analytical Study, Results, and Discussion. The frames were analyzed using a plastic limit analysis for comparison to experimental results. For a given value of axial load and lateral load, the moment capacity of the frame was determined by calculating the ultimate moment of the beam-columns. Formulas to compute this moment were given based upon the full plastic moment of the tube under pure bending and the strength of the concrete in compression. As mentioned above, the frames with the CFT beam-columns exceeded the plastic mechanism line, while the frames with the hollow tube beam-columns were unable to reach the mechanism line due to local buckling of the steel. Based upon the results of the experiments, the author proposed a value for the limiting *D/t* ratio of CFTs equal to 1.5 times that of hollow tubes. The author concluded that frames composed of CFT beam-columns and steel wide-flange beams with connection designed as demonstrated in the paper produce ductile structures having excellent seismic resistance.

Kitada, T. et al., 1987 (c; m)

Introduction. The elastoplastic behavior of short circular CFT columns was studied experimentally and compared to the values obtained from analytical methods proposed by other authors. The effect of the method of loading (i.e., load both materials, load the steel tube only, or load the concrete core only) was examined in detail as well as the effect of bond on the ductility and strength of the section. The study focused on a detailed examination of the triaxial stresses in the concrete and the biaxial stresses in the steel for the different loading methods and for the bonded and unbonded specimens.

Experimental Study, Discussion, and Results. The test specimens were very short columns with variable parameters including the steel yield strength, the concrete compression strength, and the relative ratio of the two strengths. The specimens were loaded by three methods: 1) load both the concrete and the steel simulataneously such that the longitudinal strains in both materials are equal, 2) load the concrete core alone without applying axial compression on the tube, and 3) load the steel tube alone. The stresses in each specimen were analyzed using the Mises yield criterion which is defined by the following equation:

$$\left(\frac{\sigma_{sl}}{f_y}\right)^2 - \left(\frac{\sigma_{sl}}{f_y}\right)\cdot\left(\frac{\sigma_{sc}}{f_y}\right) + \left(\frac{\sigma_{sc}}{f_y}\right)^2 = 1.0$$

where σ_{sl} and σ_{sc} are the logitudinal and hoop stress, respectively, in the steel tube. For the different loading conditions, the authors plotted the stress path of the steel, i.e., the hoop stress versus the longitudinal stress, at different locations on the tube. Because the Poisson's ratio of the concrete is less than the corresponding ratio for steel in the elastic range, the concrete and steel do not interact and act independently of one another. At the crushing strain of the concrete, the lateral expansion of the concrete exceeds that of the steel and hoop stresses are induced in the steel tube, and corresponding triaxial stresses are induced in the concrete core. A failure curve proposed by Cai that was restated in this paper accurately described the relationship between the confining stress on the concrete and the axial compressive stress. This empirical equation is given by:

$$\sigma_c = f_c^{'} \cdot \left(1 + 1.5 \cdot \sqrt{\frac{\sigma_r}{f_c^{'}} + 2 \cdot \frac{\sigma_r}{f_c^{'}}}\right)$$

where σ_c is the ultimate axial compressive stress of encased concrete subjected to a uniform, radial compressive stress, σ_r. The failure of the section under this loading condition occurred at the ends of the specimen. Soon after yielding, the steel buckled locally and the concrete in the region of local buckling crushed.

When the concrete alone was loaded, little or no compressive stress was transferred to the steel tube at the ends of the specimen and the hoop stress predominantly occurs in these regions. At the midpoint of the tube, the longitudinal and hoop stresses increase proportionately until the steel yields. The axial compression at the middle of the tube was due to the friction between the two materials. For similar tests with a debonded interface, there was much less axial stress at the middle of the tube. Failure of the CFT columns for this loading case was initiated by crushing of the concrete at the midpoint of the section. The adjacent steel bulged outward, completing the failure.

The final loading method resulted in a column that performed in an manner similar to a hollow steel tube since there was little interaction between the two materials.

The load-deformation behavior of the tubes revealed that the axial capacity as computed by summing the individual strengths of the component materials largely underestimated the actual capacity of the section. The relationship between load and deformation is linear until the steel yields, at which point the concrete begins to undergo confinement and the stiffnes of the member decreases substantially. For axial displacement beyond the yield point, the capacity of the section increases gradually, exhibiting good ductility. The method of loading did not affect the ultimate capacity of the section, except in the case of loading the steel alone in which case the concrete did not contribute any strength. Loading both materials simulataneously produced a slightly larger stiffness. The authors suggested that the calculation for the ultimate load of a short column proposed by Cai matched their results with sufficient accuracy. The equation is as follows:

$$P_u = A_c \cdot f_c' \cdot \left(1 + 2 \cdot \frac{f_y \cdot A_s}{f_c' \cdot A_c}\right)$$

<u>Matsui, C. and Tsuda, K., 1987</u> (bc; m, r)

<u>Introduction</u>. The authors presented an experimental study of CFT beam-columns subjected to vertical axial load and lateral load. The main test parameters were the presence of concrete in the tube, the *D/t* ratio (concetrating on CFTs with larger *D/t* ratios), and the method of loading--monotonic or cyclic. The authors proposed an even larger value for the limiting width-thickness ratio of the tube to 2.0 times that of a hollow steel tube. The first author had earlier proposed a value of 1.5 (Matsui, 1986).

<u>Experimental Study, Results, and Discussion</u>. The test program consisted of twenty-six cantilever specimens ranging in *D/t* from 47 to 94--twelve hollow specimens and fourteen CFT specimens. All of the hollow tubes were tested monotonically; six CFTs were tested cyclically and the remainder monotonically. Each cantilever specimen was oriented vertically to simulate a column in a frame structure. A constant axial load ranging from $0.1{\cdot}P_o$ to $0.4{\cdot}P_o$ was applied vertically to the top of the column and a lateral load was applied to the top of the column at a distance of 29.5 inches from the base, which approximates the location of the inflection point in a frame structure.

The experimental results were compared to the capacity predicted by a plastic collapse mechanism line, which assumes a plastic hinge forms at the base of the beam-column. Similar to the results of Matsui (1986), the hollow tube was unable to reach the mechanism line due to local flange buckling and then local web buckling of the tube at the beam-column base. The CFT specimens, on the other hand, invariably reached the mechanism line, even specimens having a *D/t* ratio of 2.0 times the recommended Japanese design specification to prevent local buckling. The cyclic hysteresis curves for the CFTs exhibit large ductility and large energy-absorption capacity. As the *D/t* ratio or the axial load ratio of the beam-column increases, the effect of the concrete becomes much more apparent. Hollow tubes exhibit a noticable loss of capacity and ductility as these quantities increase, while CFTs do not.

The maximum strength was compared to the predicted capacity of the section which was calculated by using a fully plastic stress distribution--all of the steel and the concrete in compression contribute moment resistance. The ratio of the maximum experimental moment to the calculated moment was conservative for most cases, ranging from 0.91 to 1.11. The authors propose this method of calculation over the Japanese Standard, which uses $0.85{\cdot}f'_c$ and is more conservative. The ratio of experimental to the author's calculated moment for the steel tubes greatly underestimated the capacity, ranging from 0.27 to 0.72.

<u>Cai, S.-H., 1988</u> (c, bc, pb; m)

<u>Introduction</u>. This article highlighted the results of seven phases of tests conducted and documented in earlier papers written in Chinese. Therefore, this paper gave only limited details regarding the nature of the experiments. However, a significant number of tests were performed, lending credence to the conclusions the author draws. He also presented detailed descriptions of failure mechanisms for short and long CFT columns and derives formulas for the ultimate strength of these columns. A method to convert the strength of what the author referred to as a "nonstandard" column (unequal end eccentricities) to a "standard" column (equal end eccentricities causing single curvature), and formulas to predict the equivalent length of standard columns, are also proposed. This latter procedure is described below under Cai (1991).

<u>Experimental Study, Discussion, and Results</u>. The seven phases of tests conducted encompassed a wide range of loading techniques including the following: concentrically loaded short and long columns, pure bending, eccentrically loaded columns bent in single curvature and double curvature, and cantilever columns. The three parameters of greatest interest in the author's tests were the slenderness ratio, the end eccentricity ratio (for the beam-columns), and the ratio of the end moments.

Short Column Behavior. The behavior of the short columns depended on the *D/t* ratio. For thin-walled tubes ($D/t \geq 19$), the load-strain curve was linear until the first yield lines were observed in the steel. The ultimate load was reached when the slope of the load-strain curve became zero, at which point the entire steel cross-section had yielded and the concrete reached its ultimate compressive strength. For very short, thick-walled tubes ($D/t = 10$), the steel confined the concrete as described in previous work by many authors. As the concrete expanded outward against the tube wall, hoop stresses were induced in the steel, effecting a decrease in the amount of longitudinal capacity of the tube and a transfer of axial load to the concrete. The confinement of the concrete allowed the section to realize a net gain in strength. Both ranges of *D/t* underwent failure when the resultant compressive force carried by the two materials reached ultimate.

Long and Intermediate Column Behavior. Axially loaded columns with a large slenderness ratio (long columns) fail by elastic buckling. A failure of this type may be described accurately using Euler's formula. However, columns between the range of short and long columns (intermediate columns) buckle inelastically. A tangent modulus may be used in place of the elastic modulus in the Euler formula to model the behavior of intermediate columns, but this requires a detailed knowledge of the CFT's stress-strain behavior. Also, columns of long and intermediate length are significantly affected by initial out-of-straightness, eccentric loading, and other imperfections, which further complicates the column's behavior.

<u>Theoretical Discussion.</u> *Short Columns*. The author derived a formula for predicting the ultimate strength of short columns. The basic assumptions were that at the limit stage, the concrete was in a state of triaxial compression and the steel was in a biaxial state of stress, the steel conformed to the Von Mises yield criterion, and the concrete failure criterion was an empirical formula developed by the author to fit experimental data. After some algebra, the ultimate load may be expressed by the following formula:

$$P_o = A_c \cdot f_c' \cdot (1 + \sqrt{\phi} + 1.1 \cdot \phi)$$

where the confinement ratio

$$\phi = \frac{\mathrm{A_s} \cdot f_y}{A_c \cdot f_c^{'}}.$$

Long and Intermediate Columns. The author proposed the following formula to determine the ultimate load of a column that fails by overall inelastic or elastic buckling:

$$P_u = \varphi_l \cdot \varphi_e \cdot P_o$$

P_{o} is the ultimate strength of a axially loaded stub column, φ_l is an empirical strength reduction factor due to the slenderness ratio, and φ_e is an empirical strength reduction factor due to the eccentricity ratio (the ratio of load eccentricity to the radius of the concrete core). For combined axial load and bending, the author presented an interaction equation to based on the above parameters:

$$\varphi_l = \frac{P_u}{P_o} + 0.74 \cdot \frac{M_u}{M_o}$$

for an eccentricity ratio less than or equal to 1.55 and

$$\varphi_l = \frac{M_u}{M_o}$$

for eccentricity ratios greater than 1.55.

Comparison of Results. *Short Columns*. The experimental and theoretical results showed good agreement. The ratio of the tested to computed ultimate axial load for the 44 specimens varied between 0.704 and 1.283 with an arithmetic mean of 0.985 and a standard deviation of 0.139.

Long and Intermediate Columns. The ratio of the tested to computed strength reduction factor for the 48 tests ranged from 0.877 to 1.352 with an arithmetic mean of 1.049 and a standard deviation of 0.107.

Zhong, S.-T. and Miao, R.-Y., 1988 (c; m)

Introduction. A standard test for short CFTs was developed to provide a basis to correlate results from different sources and to provide stress-strain input for design formulations. In lieu of experimental results, the authors also developed an analytical method to determine key points on the stress-strain curve which could be used for in computing the ultimate design strength of the CFTs.

Experimental Study, Discussion, and Results. The main goal of the standard test was to accurately obtain the longitudinal stress-strain relationships in the steel and the concrete for use in a design method. This required a test on short columns that would not fail by buckling. Columns having an *L/D* ratio ranging from 2.0 to 5.0 and a ratio of steel to concrete area, α, of 0.115 and 0.211 were selected for study. From their tests, the authors concluded that a standard test should use an *L/D* of 3.0 to 3.5. Tubes with a ratio of 3.5 showed no unloading, remained essentially straight throughout the test, and had constant strain through the cross-section. This was not the case, however, for specimens with *L/D* greater than 4.0. The lower limit of 3.0 was to avoid significant end effects. They also recommended using plate hinges for end supports (the type of support condition did not markedly affect the results and the plate hinges were the simplest to construct).

Theoretical Discussion. The analysis of the stress-strain relationship showed three stages: elastic, elasto-plastic, and strain hardening (for $\alpha \geq 0.06$) or perfectly-plastic stage ($\alpha = 0.05$). Unloading occurred if α fell below 0.04. Formulas are presented to calculate these points and the elastic modulus of the combined section. The deformation behavior of the CFT for $\alpha \geq 0.04$ closely resembles that of a plain steel tube with good ductility and energy absorbing capacity.

Design Formulation. Two semi-empirical approaches were suggested for determining the ultimate load capacity of a CFT. The first method used the theory of plasticity. The yield strength was taken as the sum of the individual steel and concrete strengths. The yield strength of the steel tube was determined using the theory of plasticity, as given by the following formula:

$$P_{so} = A_s \cdot f_y \cdot \left(\frac{\mu'+2}{\sqrt{3 \cdot (\mu'^2 + \mu' + 1)}} \right)$$

The yield strength of concrete was given by:

$$P_{co} = A_c \cdot (0.8 \cdot f_{cu} + 4 \cdot \sigma_r)$$

where σ_r is the confining pressure:

$$\sigma_r = -\frac{\alpha}{2} \cdot f_y \cdot \left(\frac{2 \cdot \mu'+1}{\sqrt{3 \cdot (\mu'^2 + \mu' + 1)}} \right)$$

The μ' value was determined empirically based on experiments by:

$$\mu' = -\frac{1}{2} - \frac{1}{2 \cdot \left(\dfrac{\alpha \cdot f_y}{0.8 \cdot f_{cu}} + 1 \right)}$$

Summing the two values of P resulted in the yield strength. The second method used the theory of elasticity. This method incorporated a similar summation of the strengths of the steel and concrete. It assumed that both materials were elastic-perfectly plastic, a valid assumption since confined concrete will behave in a manner much like steel. The steel was governed by the von Mises' yield criterion with plastic flow occurring when the steel met the yield surface. At this point on the surface, the longitudinal stress will decrease and the hoop stress will increase, due to the expanding concrete. Before the yielding of the steel, the strength of the CFT was found by simply summing the Mohr's strength of each material. By a similar formulation used above, stresses could be found. After yield, the confinement pressure was obtained assuming compatibility of the longitudinal and circumferential strains of the steel and concrete.

Liu, Z. and Goel, S. C., 1988 (br; r)

Introduction. The cyclic load behavior of rectangular CFT bracing members was investigated. The main parameters of the study were: the presence of concrete, strength of concrete, effective slenderness ratio, and width-thickness ratio. A simplified method of computing the first cycle buckling load was presented and compared to the tangent modulus approach given by Knowles and others. Both of these methods were then compared to the experimental results. The paper gave detailed descriptions of the buckling failure modes in hollow and concrete-filled bracing members.

Experimental Study, Discussion, and Results. Each tested specimen was placed diagonally inside a four-hinge testing frame. The tubes were welded to gusset plates which were in turn welded to the frame. An actuator connected to a concrete reaction wall loaded the frame, simulating the movement of a building under cyclic loads. The movement of the frames alternately stressed the brace in tension and compression. Failure occurred in all the specimens by overall buckling in the in-plane direction (bending about the minor axis). The presence of concrete may increase the number of cycles to failure and dissipate more energy, provided the tube's width-thickness ratio was not too small. Concrete may change the local buckling mode, reduce its severity, and delay the occurrence of cracking in the steel which is very beneficial in cyclic applications. The concrete forces the buckling outward which has two advantages. First, the distance between the top and bottom flanges of the tube increases rather than decreases in the case of hollow tubes. This effectively prevents the section modulus from decreasing significantly. Also, the cracking at local buckling is spread over a larger area than the hollow counterpart, alleviating severe strain concentrations, which extends the cyclic life of the specimen. The strength of concrete (4 to 8 ksi in this study) did not show a significant influence on the behavior of the CFTs since the failure was governed primarily by the buckling failure of the steel tubes. On the other hand, severe local buckling in the plastic hinge zones rapidly deteriorates the peak forces in the member. In summary, the concrete in the braces played a different role than the concrete in short columns. Rather than provide added strength, it served to augment the steel tube's buckling strength.

Theoretical Discussion. In previous papers (Knowles, 1969 et al.), the tangent modulus formula was presented to calculate buckling loads. This approach uses the tangent moduli of concrete and steel along with the Euler buckling equation to give the following form:

$$P_{cr} = \frac{\pi^2 \cdot (E_{st} \cdot I_s + E_{ct} \cdot I_c)}{(K \cdot L)^2}$$

Assuming strain compatibility between the steel and concrete combined with the equilibrium condition

$$P_o = A_s \cdot f_y + A_c \cdot f_c^{'}$$

the buckling load may be obtained. Although this procedure has shown very accurate results, it requires the use of a computer to perform the calculations, namely to obtain the tangent moduli from the complicated stress-strain functions. Therefore, the authors wanted to develop a simpler process, hence the approximate method presented in this paper. The method relied on the following assumptions: the steel tube and concrete acted independently, thus allowing the AISC formula to be used for the tube; the stress-strain relationship was linear up to the buckling load; and the longitudinal strain in the concrete was the same as the steel. The first step was to calculate the buckling load of the steel without the presence of concrete using the AISC formula. Second, the strain and the stress in the concrete were computed using the latter two assumptions stated above. Knowing the stresses in both materials led to a simple calculation of the first cycle buckling load:

$$P_o = A_s \cdot f_y + A_c \cdot f_c^{'}$$

This approach resulted in values in good agreement with those obtained by the more complex tangent modulus method and the experimental results were also quite close to these values.

<u>Kawano, A. and Matsui, C., 1988</u> (br; r)

<u>Introduction</u>. In this paper, an experimental study to investigate the behavior of circular HTs and CFTs under repeated axial loading was presented. In addition, the response of the frames having tubular bracing members was examined analytically.

<u>Experimental Study</u>. The specimens were simply supported at the ends. Two loading schemes were selected. In the first one, the axial load was applied with repeated large amplitude axial deformation and a repeated axial load with gradually increasing amplitude was applied in the second loading scheme. The specimens had a *D/t* ratio of 28.9. The measured compressive and tensile yield strengths of the steel tubes were 45.67 and 48.51 ksi, respectively. The measured concrete compressive strength was between 4.75 and 5.08 ksi. The range of the *L/D* ratio was 6.81 to 40.88.

Failure mostly occurred when the second loading scheme was applied. The failure location was at the mid-length and the failure pattern was cracking of the steel tube. For the CFT specimens, cracking occurred at the crest of the local buckling bulges. The hollow tubes cracked at earlier stages of loading, with cracking occurring in the corners of the tube. For the CFT specimens having an *L/D* ratio less than 40.88, the yield strength was larger than the buckling strength. The CFT specimens exhibited better response than the hollow ones under both of the loading schemes. The concrete infill delayed local buckling and cracking of the steel tube. The energy dissipation of the CFT members was up to 3.2 times that of the hollow tube specimens.

Analytical Study. The behavior K-braced portal frames made up of tubular braces was investigated theoretically. Two types of frames were analyzed. The first type had a beam hinged at the middle where the braces were connected. For this type of frames, the CFT and HT braces underwent buckling and the frames exhibited rapid deterioration in strength. However, the CFT braces improved the behavior under repeated horizontal loading. In the second type of frames, the beam was hinged at mid-length. In this case, CFT braces achieved a more stable behavior compared to HT braces and no deterioration in strength was obtained.

Matsui, C. and Kawano, A., 1988 (br; r)

Introduction. The behavior of circular CFTs when they were used as truss members was investigated. The authors presented the results of the experiments conducted on trusses. Two kinds of parallel chord plane trusses were tested in two test series. The first type was constructed from concrete-filled tubular chords and the second type was constructed only of hollow steel tubes.

Experimental Study, Results, and Discussions. In the first test series, the specimens were loaded monotically with uniform bending moment. The trusses were simply supported at their ends for both in-plane and out-of-plane deformations. The *D/t* ratio for the chord tubes and web tubes were 26.3 and 14.32, respectively. The column slenderness ratio, L/r_s, of the chords was 60 and the height between the parallel chords was 75.86 in.. The measured yield strength of the steel was 48.52 ksi and the measured compressive strength of the concrete was 4.98 ksi. The concrete-filled tubular trusses exhibited more ductile response compared to the specimens with hollow tubes. No lateral instability was observed in the concrete-filled trusses, which resulted in less reduction in strength after the peak load. However, rapid reduction in the post peak strength occurred in the hollow steel tube trusses due to lateral instability. From the compression and tension tests conducted on individual concrete-filled chord members (Kawano and Matsui, 1988), the buckling load was determined to be higher than the tensile yield strength by up to 25%. Consequently, it was concluded that the concrete in the steel tube made the buckling strength higher than the yield strength, thus preventing lateral instability.

The second test series included testing of cantilever truss beam-column specimens under constant axial load and cyclic horizontal load. The specimens were fabricated from the same materials as in the previous tests. They were braced against lateral deformations at the middle and at the ends. The applied axial load was equal to 20% of the yield strength found from the compression tests of similarly sized stub columns that were conducted in an earlier phase of this research. The concrete-filled trusses showed large and stable hysteresis loops, but the behavior of the hollow steel tube trusses was governed by lateral instability, and they showed much smaller hysteresis loops.

The authors reported that stable behavior of the trusses could be ensured if the buckling strength was greater than the yield strength. Consequently, the column slenderness ratio should be limited to increase the CFT buckling capacity. When selecting the limiting value of the column slenderness ratio, both the strain-hardening and Bauschinger effect should be taken into account because strain-hardening increases the strength of the steel, while the Baushinger effect tends to lead to a lower buckling strength. These effects both reduce the limiting column slenderness ratio.

<u>Prion, H. G. L. and Boehme, J., 1989</u> (c, bc, pb; m, r)

<u>Introduction</u>. The authors presented results from tests of thin-walled steel tubes filled with high strength concrete. The program focused on the behavior up to the ultimate load and beyond, with an emphasis placed on ascertaining the amount of ductility that could be achieved with these members. Loading combinations included pure axial compression, pure bending, and various combinations of the two. Three cyclic load tests were performed along with the monotonic tests. The results were compared to design codes based on either strain compatibility or superposition (Japanese code). The description of the tests and the presentation of the results was extremely thorough and detailed and included a number of very illustrative graphs and figures.

<u>Experimental Study, Results, and Discussion</u>. Each of the tested specimens for all types of loading had the same cross-sectional properties, allowing a direct comparison of the results. In general, all of the specimens behaved in a ductile manner. The high strength concrete, however, behaved in a brittle manner.

Short Columns. To observe the effect of confinement, the authors tested columns with a slenderness ratio (L/D) of less than 15. Along with the slenderness ratio, the compressive capacity of the section depends on the D/t ratio and the method of load application. To test the effect of load application, two types of tests were performed -- one loading the concrete alone, and one loading both the concrete and the steel equally. Preferably, only the concrete should be loaded to maximize the effectiveness of the steel tube (i.e., have it only subjected to tensile hoop stresses from the expanding concrete). However, in reality, friction and chemical bond between the two materials will cause axial stress in the steel due to a load transfer from the concrete to the steel. The short columns showed two stages of failure. The first stage involved a shear failure in the concrete causing an abrupt loss of capacity. The shear failure of the concrete caused the two wedges to slide past one another, effectively alleviating the axial compression in the steel and inducing additional circumferential stress in the steel tube. The failure mode at the secondary load level was of the desirable ductile type, although the tube was able to sustain only about 60% of the ultimate load. There was no noticeable difference in the results of the different methods of load application; the specimens with the concrete loaded only and both materials loaded uniformly performed quite similarly.

Beams. Load was applied to the beam at two points such that there was a region of constant moment at the center of the beam. The length of this region was varied to study the effect of shear on the moment capacity of the section. No definite trend was recognized. A cyclic test was performed on one beam at a ductility level of two to three times the deformation at first yield until failure occurred. The beam specimens failed in a ductile manner at the points of load application. Shear fracture of the concrete initiated failure. This was followed by tensile yielding and subsequent rupture of the steel tube. Local buckling also occurred in the compression region of the beam near ultimate. Significant slippage between the materials was observed at the ends of the specimens. Although this did not seem to affect the moment capacity, it would have a substantial effect on the stiffness of the member. The specimen under cyclic loading showed a slight decrease in strength with each cycle but dissipated a large amount of energy.

Beam-Columns. These tests were similar in fashion to the beam tests. The axial load was applied first and then transverse load was applied at two points creating a constant moment region. Two cyclic tests were performed, again with the load cycled at a ductility of two to three times the deformation at first yield. Failure of the beam-columns occurred at the center of the specimens. After cracking of the concrete and buckling of the steel tube in the compression region, the member failed due to a rupture in the tensile zone. These members showed a significant increase in moment capacity due to the presence of an axial load. The axial load causes a greater utilization of the concrete in the compression zone than occurred in the beam tests. The post-failure behavior mimicked the behaved observed in the beam tests; the moment capacity of the beam-column beyond ultimate decreased to a nearly equivalent level as the beam. The cyclic tests exhibited good ductile behavior. Failure occurred after 4 1/2 cycles due to tensile fracture. Slight pinching of the hysteresis loops was observed which was attributed to the opening and closing of concrete cracks combined with the yielding and buckling of the thin steel tube.

Eccentrically Loaded Columns. The eccentric loading tests were performed using a high axial load. Unlike the previous tests, the eccentrically loaded columns failed in a non-ductile manner. The authors attributed this to the brittle nature of the high strength concrete and the lack of proper confinement. The authors warn that this behavior should be of concern since these members are often used in earthquake regions.

Comparison of Results. It was found that, with the exception of the beam-columns subjected to a high axial load, the CFTs could be modeled accurately with a strain-compatibility model. Behavior of CFTs beyond ultimate should be described by a superposition model treating the two materials as individual elements. The Japanese superposition model was generally unconservative. Due to the lack of confinement from the thin steel tube, the concrete in the tests was unable to reach its full moment capacity. Also, local buckling of the tube prevented the full steel plastic moment capacity to be achieved.

Shakir-Khalil, H. and Zeghiche, J., 1989 (c, bc; m)

Introduction. Tests were conducted on 7 full-scale rectangular CFT columns, 1 axial, 4 in uniaxial bending (2 about the major axis, 2 about the minor axis), and 2 in biaxial bending. The results were compared with the predictions of a finite element analysis and the BS5400 predicted failure loads. Studies were also conducted to study the amount of bond in CFTs. A number of graphs and tables illustrated the results of the tests in detail.

Experimental Study, Discussion, and Results. Loading was applied in increments of 10% of the maximum load until some relatively large non-linearity was observed. Failure was considered to have occurred when the columns shed off any additional load increment.

Bond. Bond tests were performed using a push-out test (i.e., the concrete was loaded by a steel ram and could move vertically when the bond failed). The relationship between load and slip was linear up to about 90% of failure, after which the load-slip curve became nonlinear as the slip showed a continuous increase with the addition of load. The bond strength of the tested specimens varied from only 38 to 54 psi. These values are relatively low compared to reinforcement bars and even circular CFT sections. The combination of shrinkage and the relative flexibility of the rectangular tube walls versus a circular tube has an adverse effect on the amount of bond strength developed.

Long Column Tests. The failure mode of all the columns was overall flexural buckling. No local buckling was observed. The results show that the moment due to the eccentricity played a significant role in columns subjected to small eccentricities. For larger eccentricities, the secondary moment due to P-δ effects could become larger than the primary moment caused by the eccentric loading. The need to incorporate initial out-of-straightness was recognized, although it was not considered in the paper. Strains across the cross section showed a linear variation up to 96% of the failure loads for all of the specimens. The yield strain was reached in the compression zones at loads varying from 80% to 90% of ultimate. Beyond this point, the strains increased rapidly with very small increases in the load. The strains in the tensile zones reached yield after failure as the column underwent large lateral displacements and were subjected to high bending moments.

Theoretical Discussion. *Axially Loaded CFTs*. The behavior of axially loaded CFT columns depends mainly on the column's effective buckling length, the minimum dimension of its cross section, and the mechanical properties of the steel and concrete. These properties may be combined to define the column as 'short' (stocky), 'medium' length, or 'long' (slender). Short columns fail by steel yielding and concrete crushing. Medium length columns exhibit inelastic behavior and fail by overall flexural buckling. The failure mechanism is characterized by partial steel yielding, crushing of the concrete on the compression face, and concrete cracking on the tensile face, which are caused by the induced bending moment in the column as it deforms laterally. Long columns behave elastically and fail by flexural buckling. Their behavior may be accurately predicted using the Euler approach.

Eccentrically Loaded CFTs. Short columns under eccentric loading exhibit a linear load-moment relationship up to failure (i.e., lateral displacement of the section is insignificant compared to the applied moment). As the column length increases, the secondary moment due to the mid-height lateral deflection becomes larger than the moment caused by the applied eccentricity. This causes the column to fail by buckling or bending rather than by compression.

Analytical Study. The first step in the numerical analysis was to develop moment-thrust-curvature diagrams. Following that, the analysis to determine the ultimate load was carried out by a finite element method and also by an approximate method using a sine curve to model the deflection with equilibrium maintained at mid-height. The moment-thrust-curvature curves were developed under the following assumptions: complete interaction between the steel and concrete, plane sections remain plane, the steel stress-strain relationship was elastic-perfectly plastic, and strain hardening of steel and tension in the concrete were ignored. The curves were constructed by establishing the neutral axis in the cross section for a given set of axial load and curvature. The moment to maintain equilibrium was established for each variation in curvature, then the process was repeated for a different axial load. The sine-curve deflected shape procedure first assumed an initial axial load and corresponding mid-height displacement. Using the moment-thrust-curvature relationship, external and internal bending moments were computed and their difference compared to a preset tolerance. This iterative process continued until a specified maximum number of iterations was exceeded. The load at this point was taken as the failure load. In the finite element analysis, the column was discretized into 10 beam elements with equilibrium and compatibility satisfied at each of the 11 nodes. The relationship between the external moment and the bending moment at each node was given as follows:

$$E \cdot I \cdot \frac{\partial^4 \delta}{\partial x^4} - P \cdot \frac{\partial^2 \delta}{\partial x^2} = 0$$

where E·I is the flexural rigidity of the section (not defined by the authors) and P is the axial load. The authors began the analysis by setting the end force and lateral mid-height displacement. The moment-thrust-curvature relationship was generated and then the stiffness matrix was formulated from which the curvature vectors could be obtained. Like the sine-curve approach, internal and external moments were computed based on curvatures and compared in an iterative process until a solution was found.

Further Research. The authors emphasized in conclusion that many more tests must be performed to validate the results of their investigation. They suggested that initial out-of-straightness should be measured and incorporated into the analytical formulation. Finally, more tests regarding the validity of the BS5400 predictions should be performed, especially concerning minor axis bending.

Cederwall, K. et al., 1990 (c; m)

Introduction. This paper presented results of experimental tests of rectangular CFT columns using high-strength concrete. Eighteen slender columns with warying degrees of eccentricity were tested to evaluate the advantages of high strength concrete in CFTs, the confining effects of composite sections, and the shear transfer at the steel/concrete interface. The main variable parameters in the tests were the concrete and steel strengths, the thickness of the steel tube, and the load eccentricity. Additionally, CFTs with interior reinforcement and CFTs with a debonded interface were examined.

Experimental Study, Discussion, and Results. The rectangular CFT columns in the test were simply supported and subjected to an increasing axial load until failure of the member occurred. Each long column test was accompanied by a corresponding stub column test having the same material properties and cross-sectional geometry to analyze the squash load. It was shown that the axial load capacity of the CFT section was about 6% higher than the capacity calculated by summing the individual strengths of the steel and concrete. The 6% increase in total capacity corresponds to a 15% increase in the capacity of the concrete core. The authors stated that this increase in strength has not, as a rule, been shown for rectangular CFTs. The experiments showed two distinct points of stiffness degradation as the member was loaded. The first occurred at the onset of compression steel yielding, and the second at the onset of tensile steel yielding. Therefore, the authors suggested that the maximum load bearing capacity of the section was determined by the strength of the steel tube. Furthermore, as the ratio of the nominal concrete load bearing capacity to the nominal capacity of the steel, P_{co}/P_{so}, increased, the ratio of the load bearing capacity of the long column to the capacity of the short column, P_u/P_o, decreased.

Several parameters were varied in the course of the experimental study. The strength of the concrete in the CFT column had a greater effect on the ductility of the secdtion than on the strength of the section. As the concrete strength was increased, the column exhibited an increase in the amount of ductility, but a somewhat less significant increase in strength. The ductility at deflections beyond the section's ultimate strength was attributed to the reserve capacity of the concrete, which was underutilized at the ultimate load (the tests showed an average concrete load of $0.3 \cdot P_{co}$ at P_u). This underutilization of the concrete in slender columns decreases its contribution and relative importance to the axial load capacity of the section. An increase in the thickness of the steel tube resulted in an increase in the load bearing capacity of the section, as did an increase in the yield strength of the steel. Increasing the yield strength, however, resulted in a loss of ductility in the section. The effect of increasing load eccentricity was a decrease in the load bearing capacity and stiffness of the column, and an increase in the ductility. The high-strength concrete increased the load bearing capacity of CFT columns with little or no eccentricity. However, the contribution to the ductility of the section was realized only in the long CFT columns subjected to large eccentricities.

The ductile behavior of the CFT columns depended integrally on the existence of bond between the steel and the concrete. In tests where the materials were debonded, the concrete core did not contribute to the column behavior when the steel alone was loaded and the column behaved as a hollow steel tube. When the concrete alone was loaded, the strength of the section increased because the steel confined the concrete to some extent.

Konno, K. et al., 1990 (bc; r)

Introduction. Experimental tests were conducted on square concrete-filled steel beam-columns to determine the ultimate strength of the member, focusing on the effects of local buckling of the steel tube and confinement of the concrete. The authors recognized a marked increase in both the strength and the ductility of the concrete due to the effect of confinement in the square tubes. They proposed a design formulation which accounts for confinement as well as local buckling of the steel tube.

Experimental Study, Results, and Discussion. The tested specimens were subjected to combined axial load and bending, the latter of which was induced by an alterating transverse load at the midpoint of the section. The axial load on the specimen was kept constant throughout each test. A total of 19 specimens were tested. The variable parameters included the steel and concrete strength, the D/t ratio, and the amount of applied axial load.

The authors observed that the ultimate strength of the specimen was closely related to the onset of local buckling of the tube, which usually occured prior to strain hardening of the steel. They also showed that square CFT members have a large capacity for energy-absorption, even members with a large D/t ratio or memebers subjected to high axial loads.

Design Formulation. The analytical strength of a CFT was computed by superposing the individual strengths of the steel and the concrete. The steel strength was determined by computing an effective width, which was computed using an empirical relationship based on the D/t ratio and the yield strength of the steel tube:

$$D_{effective} = D \cdot \left(2.5 \cdot \frac{\sqrt{E_s/f_y}}{D/t} - 1.56 \cdot \left(\frac{\sqrt{E_s/f_y}}{D/t} \right)^2 \right)$$

The concrete strength was increased based on an empirical experiment-based formula to account for the effect of confinement:

$$f_{cc} = f_c' \cdot \left(1 + 48.8 \cdot \frac{f_y / f_c'}{(D/t)^2}\right)$$

The strength calculation using the modified effective width and concrete strength showed better agreement with the experimental values.

<u>Shakir-Khalil, H. and Mouli, M., 1990</u> (c, bc; m)

<u>Introduction</u>. Following up on the study reported in 1989, the authors conducted nine more tests on full-scale rectangular CFT columns, incorporating two column sizes. The majority of the columns were subjected to biaxial bending. Also tested were a series of short CFT specimens in axial compression to establish the squash load of stub columns having this type of section. The results were discussed and compared with the British standard BS5400. The main parameters of study were the concrete strength, the tube strength, and the effective length.

<u>Experimental Study, Discussion, and Results</u>. *Short Column Tests*. Tests were performed to determine the effect of different parameters on the carrying capacity of various sections. The capacity increased with an increase in the size of the steel section and the with the use of higher strength concrete. However, the capacity decreased with the use of higher strength steel and/or an increase in the column length. In both of the latter cases, the steel carries more of the load, decreasing the concrete's contribution. The benefits of the concrete's compressive strength begin to diminish and the CFT behaves more like a hollow tube.

Long Columns Tests. The columns were oriented in a horizontal position with the larger cross-sectional dimension in the vertical direction to minimize the effect of the column self-weight. The loading process was very similar to that described in Shakir-Khalil and Zegiche (1989). The failure of the columns was typified by yielding in the compression corner of the tube at midheight at about 90% of the failure load, followed by tension yielding on the opposite corner, and leading to an overall buckling failure. No local buckling was noticed in any of the sections. As expected, filling the tubes with concrete produced a considerable gain in strength -- 12% to as much as 65%, depending on the column dimensions and material properties. The percent gain was augmented by an increase in concrete strength and an increase in the tube size. Increasing the yield strength of the steel and increasing the length had detrimental effects.

Bond. Bond tests paralleling those conducted in Shakir-Khalil and Zegiche (1989) were repeated here with similar results. It was noted, however, that the higher strength concrete produced the higher value of bond strength.

<u>Analytical Study</u>. A computer program developed by the authors was briefly described. They used the program to generate several interaction diagrams, plotting P/P_u versus M/M_u. The values of both quantities causing full plasticity of the cross section were obtained by the program. The calculation of the ultimate moment M_u was based on the use of rectangular stress blocks assuming strengths of $0.6f_{cu}$ for concrete in compression and no strength in tension, and f_y for steel in both tension and compression. The curve was obtained by varying the position of the

neutral axis, and considering the equilibrium between the resultant stress blocks and the applied eccentric load. These could be generated for various sizes and properties of the columns.

Cai, S.-H., 1991 (bc; m)

Introduction. Twenty-seven circular CFT columns with non-uniform moment distribution diagrams were tested and the results were briefly described. The main parameters were the slenderness ratio, eccentricity ratio, and the ratio of the smaller to larger end moments. A method was described to convert a column with a non-uniform moment distribution to an equivalent column with uniform moment distribution. This formulation was the main thrust of the paper.

Experimental Study. Of the 27 tests performed, 18 were columns bent in single curvature with eccentricity only at one end; the remainder were bent in double curvature with two opposite end eccentricities. These experiments served as test data for the theoretical study, namely to obtain values for the parameters that will subsequently be defined.

Theoretical Discussion. The ultimate strength of a circular CFT column was proposed as follows:

$$P_{uo} = \varphi_l \cdot \varphi_e \cdot P_o$$

P_o is the ultimate strength of a axially loaded stub column, φ_l is a strength reduction factor due to the slenderness ratio, and φ_e is a strength reduction factor due to the eccentricity ratio (the ratio of eccentricity to the radius of the concrete core). Five types of moment distribution were described and quantified using a parameter β equal to the ratio of the smaller end moment to the larger end moment. Standard columns were defined as columns with equal end moments bent in single (β = 1) or double curvature (β = -1). The remaining three types had unequal end moments --one type was bent in single curvature (0 < β < 1), one in double curvature (β < 0), and the third was the special case of zero moment at one end (β = 0). To convert an unequal moment distribution to an equal one, the authors applied an equivalent-length factor K (less than 1) and replaced the unequal moments with uniform ones equal to the larger moment. Values of *K* were determined experimentally for β = -1 and β = 0 (k must equal 1 for β = 1) and then interpolated for intermediate values. Using the value of *K*, the eccentricity ratio in the above equation was modified. As shown in a previous paper by the author, the load-moment interaction equation based on the above parameters may be expressed as:

$$\varphi_l = \frac{P_u}{P_o} + 0.74\frac{M_u}{M_o}$$

for an eccentricity ratio less than or equal to 1.55 and

$$\varphi_l = \frac{M_u}{M_o}$$

for eccentricity ratios greater than 1.55. The paper illustrated an interaction curve based on these formulas and limits on the strength reduction factors.

Comparison of Results. For ultimate load, the results of the experimental method and the computation using the above approach showed good correlation: the ratio of P_u/P_o varied from 0.935 to 1.161.

Huang, S. et al., 1991 (bc; m, r)

Introduction. The test results of 46 CFT members under combined compressive, flexural, and shear loading were presented. The objectives of the experiment were to plot hysteresis loops for cyclic loading, determine the ductility ratios and absorbed energy ratios of the tested members, observe the failure mode and the effect of different parameters (i.e., slenderness ratio, steel ratio, and the ratio of applied axial load to the section's squash load) on member behavior, and to analyze the difference between the behavior of CFT and reinforced concrete members.

Experimental Study, Discussion, and Results. The test members were oriented vertically, fixed at the base, and attached rigidly to a frame through which the lateral load was applied. An axial load P was applied first and then a lateral load H was applied until the member reached its maximum capacity. The plots of the hysteresis loops for the CFT member were very full and there was no deterioration of stiffness as the loads were cycled, except in specimens with small axial load ratio (P/P_o). The failure of the specimens was due to a combination of compression and bending, manifested by a cracking of the concrete and local buckling around the entire perimeter of the tube at the top of the column where the loads were applied. Failure was preceded by four distinct stages of loading. Up to $0.4 \cdot H_{max}$, the member remained elastic. At this point, confinement began as Poisson's ratio for concrete exceeded that of the tube. The ratio of circumferential to longitudinal strain in the tube at the point of maximum compression at the top of the member was approximately 0.4. The portion of the load-deformation curve at this stage was non-linear. At $0.75 \cdot H_{max}$, the tube began to deform plastically. The concrete continued to expand laterally at a greater rate than the steel and the confinement increased. The ratio of circumferential to longitudinal strain at this stage was more than 0.8. Strain hardening occurred here for some specimens while others began to show the formation of local buckling. The behavior in the final stage (H_{max} and the subsequent unloading beyond this point) was governed by the P/P_o ratio. For very small P/P_o, no unloading occurred. The authors' discussion of the relation between H and P for higher values of P at failure was very nebulous.

Most of the specimens were able to absorb over 90% of the energy during cyclic loading. The P/P_o ratio had the greatest effect of the variable parameters. Increasing P produced an increase in both the ductility ratio ($\varepsilon_u/\varepsilon_y$) and the amount of energy absorbed (area within the hysteretic loop). Increasing the steel ratio (A_s/A_c) increased ductility, energy absorption, and P_u; increasing the slenderness ratio had the opposite effect. Comparing the CFT member to the reinforced concrete member with the same slenderness ratio, steel ratio, and axial load ratio showed a value of P_u for the CFT of 2.174 times that of the reinforced concrete member. This gave a corresponding hysteretic loop with much greater area signifying better seismic behavior. Despite this, the ductility ratio and the energy absorption ratio of the CFT and the reinforced concrete members were very similar, indicating that these parameters were not good measures of seismic performance. Therefore, the authors recommended calculating the absorbed energy per unit volume of the member. Doing this, they found that the energy absorption capacity of the CFT was 2.9 times the capacity of the reinforced concrete member.

Ichinohe, Y. et al., 1991 (bc; m, r)

Introduction. The elasto-plastic behavior of CFT circular specimens constructed of high-strength steel tubes and high-strength concrete was examined by performing 11 tests to determine moment-curvature relations and 9 tests to determine the shear in specimens subjected to bending. The tests were conducted both monotonically and cyclically and included an axial load. The authors defined a limiting D/t ratio below which specimens were free from moment degradation due to local buckling. The authors also defined a limiting axial force ratio above which excessive axial deformation under a cyclic load would occur, leading to a loss of building frame stability. The paper also proposed an analytical method to explain the elasto-plastic behavior of the columns.

Experimental Study, Discussion and Results. The following parameters were used in the tests: D/t, P/P_o, λ, annealing or no annealing, and loading pattern. The first group of tests, the moment-curvature tests, were performed by loading a horizontally-oriented, simply-supported specimen axially and transversely at approximately the third points along the specimen's length. In the monotonic moment-curvature tests, decreasing the D/t or P/P_o ratio delayed local buckling and steepened the slope of the increasing moment in the plastic zone. All specimens with D/t less than or equal to 53 showed an increasing moment after local buckling with no degradation. However, a specimen with D/t equal 71 did show degradation. Therefore, a critical bifurcation point exists between these two values. Under cyclic loads, all specimens exhibited good energy absorption and little or no moment degradation. The second group of tests, the shear bending tests, again consisted of a horizontally oriented, simply-supported member loaded axially and transversely. This time the transverse load was applied via a beam framing into the member at mid-length. The connection of the beam and the tested specimen formed a rigid joint at the load point. Therefore, each half of the beam was treated as an axially loaded cantilever; the rigid joint at the midpoint of the specimen served as the fixed end of each half, and the simply-supported ends simulated the free ends of a cantilever. For these shear-bending tests, if the average axial force ratio P/P_o was less than or equal to 0.5, the deformation due to P had little influence on stability, even though P was temporarily increased to $0.7 \cdot P_o$ by the overturning moment.

Analytical Study. The proposed analytical method was a mechanics-based analytical method which tried to model the behavior of concrete-filled steel tubes. The model assumed biaxial stress in the steel and triaxial stress in the concrete, and hence confinement of the concrete. The ratio of hoop stress to longitudinal stress in the steel was found experimentally to be 0.5 to 0.6 on the tension face and between 0 and 0.5 on the compression face. Using these experimental values and the von Mises yield criterion to relate the longitudinal and hoop stresses, the authors obtained the stress-strain relationship in the plastic region. The concrete stress-strain relationships were adopted from Park and Priestly. The moment-curvature relationship was obtained by numerical integration based on Navier's hypothesis. The moment-rotation was obtained by integrating the calculated moment-curvature along the axis of the member. The load-deformation relationship of the column was traced favorably by this method over the elasto-plastic region (the region between first yield and the full plasticity of the steel tube). The shear bending test was mimicked well showing that both the moment-curvature and shear bending tests may be evaluated with the same moment-curvature relationship.

Xu, J. et al., 1991 (ct; m, r)

Introduction. The experimental behavior of CFT members under combined compression and torsional loads was discussed. The authors examined the effect of parameters including the ratio of compressive to torsional load, the ratio of steel to concrete, the strength of the constituent materials, the loading path, and oiled versus non-oiled inner steel surfaces. The experimental results were rather abbreviated. Forty-seven specimens were tested under nonproportional monotonic axial force plus torsion loading and six were tested under axial force plus nonproportional cyclic torsion loading.

Experimental Study, Results, and Discussion. The tested specimens showed good torsional resistance and exhibited axial bearing capacity even at the member's ultimate torsional strength. This indicated that as the tube bulges and creases, the concrete rapidly fills the voids created between the materials, keeping both materials in an active role in resisting the axial load. Failure of the specimens began with a cracking of the concrete. Torsional resistance at this point was still influenced by the interlock of the aggregate particles as well as the friction caused by the axial load. Therefore, the authors found that up to a point (it varied with section properties -- usually about one half of the ultimate axial load) an increase in axial load produced a corresponding increase in the torsional moment the section was able to resist. This increase in torsional moment was not observed in tests of plain concrete or hollow tubes.

Luksha, L. K. and Nesterovich, A. P., 1991 (c; m)

Introduction. The behavior of large diameter CFT members under axial compression was investigated and the authors noted some of the failure peculiarities of the specimens. The discussion of the large diameter failures was quite detailed.

Experimental Study, Results, and Discussion. Ten sets of specimens were tested, 3 CFT and 1 HT in each set. The main parameter of interest was the diameter of the tube. Each set had a different diameter varying from 6.25 in. to 40.2 in. The steel tubes were welded and filled with heavy concrete of unspecified weight and the *L/D* ratio was 3. Two types of failure were recognized in the study. Small diameter tubes were characterized by local buckling around the end of the specimen accompanied by crushing of the concrete in this zone. The large diameter specimens failed in shear. The failure began at 90% of the ultimate load by the formation of buckles along the cylinder's diagonal. The failure lines on the tube shifted before failure, and an oblique crack formed in the concrete at about a 25 to 35 degree angle to the vertical. Just before the shift occurred, the radial compressive stresses in the concrete reached their maximum value. The concrete shifted and the load between this point and failure increased very little as the steel buckled. The authors alluded to analytical results from an earlier paper and compared their experimental results to these values.

<u>Masuo, K. et al., 1991</u> (c; m)

<u>Introduction</u>. The buckling behavior of CFT columns was studied both experimentally and analytically, using both lightweight and normal weight concrete subjected to concentric axial load. Ultimate loads were discussed with regard to three parameters: concrete weight, size of the steel tube, and effective column length.

<u>Experimental Study, Discussion, and Results</u>. Twenty-six columns were tested, 18 lightweight and 8 normal weight. Initial out-of-straightness was accounted for by assuming a mid-height deflection of L/2000 for all columns except those with a slenderness factor

$$\bar{\lambda} = \frac{L_{eff}}{\pi \cdot \sqrt{(E_s \cdot I_s + E_c \cdot I_c)/P_o}}$$

of 0.8-1.0, where L_{eff} is the effective column length and P_o is the squash load, given by

$$P_o = A_s \cdot f_y + 0.85 \cdot f_c^{'} \cdot A_c$$

The initial deflection at mid-height for columns in this range of slenderness ratios was computed as the deflection of the column before the test divided by the effective length. In the tests, an initial deflection of L/4000 or L/8000 was used, the higher number for more slender columns. The authors found that both weights of concrete with slenderness factors around 0.3 were definitely affected by confinement, the normal weight concrete showing a somewhat larger effect. The load-deflection relations were also significantly affected by the confining effect in this range of slenderness factors. Several detailed graphs elucidate this point. Varying the *D/t* ratio from 30-40 and holding the other test parameters constant did not seem to affect the squash load. Finally, the ultimate loads of the CFTs for both weights at a slenderness factor of 0.6 were somewhat larger than the European column curve, to which they were compared to.

<u>Analytical Study</u>. Load-deflection curves were developed using a mechanics-based analytically method incorporating the following assumptions. A sinusoidal shaped deflection curve with an initial out-of-straightness was used with equilibrium taken at mid-height on the basis of assuming plane sections remain plane. The stress-strain relations for the steel were assumed to follow the Ramberg-Osgood function. Two different constitutive equations were used for the concrete based on Popovics's formula, one assuming no confinement and the second assuming confinement. Load-deflection relations that ignored confinement were less than the experimental results, especially when slenderness factors are small, suggesting significant strength augmentation due to confinement. The analytical results made accurate predictions of the load deflection curves when confinement was assumed in the model.

<u>Nakai, H. et al., 1991</u> (c; m)

<u>Introduction</u>. The authors performed an experimental study on creep and drying shrinkage in circular CFTs. They attempted to predict the time-dependent behavior of these

structures by means of a viscoelastic model, whose parameters were evaluated experimentally. Creep is an important consideration in the design for serviceability and has a significant role in the amount of stress and deformation during the construction of a structure. To the point of the article's writing, little study of creep had been performed.

Experimental Study. Three test specimens were used to investigate each type of behavior. One of each group of three was a plain concrete specimen with no tube. The tests for all specimens were performed simultaneously in the same environment. The specimens measuring creep were loaded and the specimens measuring drying shrinkage remained unloaded. The duration of the test was 6 months, at which point the amount of creep had stabilized.

Analytical Study. The authors used a viscoelastic model -- the relatively simple Kelvin model. The element consisted of a spring and dashpot connected in parallel. The applied stress was assumed to be a function of the initial stress and experimentally-derived parameters obtained from tests of long-term axial loading of CFT columns.

Comparison of Results. The results of the experimental and theoretical methods showed good agreement except in the initial part of the strain versus time curve. The simplicity of the viscoelastic model may have accounted for this discrepancy. The authors hesitated in recommending a more complicated element because they felt it would lead to excessive computation time were this element to be employed in a finite element evaluation. The amount of creep as measured by the creep coefficient (ratio between the final strain and the elastic strain at time 0) varied from 1.44 to 1.61 for the CFTs and from 2.72 to 2.84 for the plain concrete column. Drying shrinkage of the concrete in the CFTs was very small in comparison to the plain concrete and may be neglected in practice.

Pan, Y. and Zhong, S., 1991 (no tests)

Introduction. The authors presented a definition of CFT strength under axial loading, based on a load-deformation curve they derived in earlier papers. They strove to present a definition of ultimate strength which would allow a direct comparison of results obtained by investigators using disparate experimental methods. To accomplish this, the authors recommended using load-deflection curves.

Theoretical Discussion. The authors defined two types of behavior for axially loaded CFT members based on the ratio of the column length to its diameter (L/D). Load-deformation curves for $L/D \geq$ 4-5 showed an unloading portion of the curve as the deformation increases beyond the maximum load. Specimens of this type failed by overall buckling. As L/D increased beyond $L/D = 5$, the curve showed a sharper drop-off indicating a more sudden failure and the unloading occurred at lower and lower loads.. For tubes with an L/D ratio of between 3 and 3.5, the curve ascended to failure. This indicated a specimen dominated by shear failure in the concrete and local outward buckling of the tube. The authors used these latter specimens to study the fundamental behavior of axially loaded CFTs. For such specimens, they showed that the steel ratio may also affect the shape of the load-deformation curve. A steel ratio below 6% or 7% produced results much like the more slender columns, showing a descending curve after the maximum axial load was reached. In practice, though, the steel ratio will generally be higher than 7%.

Axially Loaded CFTs. A number of different definitions of bearing capacity for axially loaded CFTs have been given in the literature. The authors briefly touched on several of these, ranging from a low value of axial load given as the yield strength of the tube plus the load on the concrete at the time of steel yielding (Furlong, 1967) to a high value obtained by assuming the steel yields entirely in hoop tension with no longitudinal strain and the concrete supports the full load under triaxial compression (Bondale and Clark, 1966). In fact, this steel stress state will never occur even if only the concrete is loaded because of interaction between the steel and concrete. The authors noted, however, that the values obtained by this formulation may not be far off since the steel goes into strain hardening and the concrete is able to reach the triaxial stresses in the aforementioned definition. The definitions were compared by plotting the ultimate load points on a load-deformation curve which allowed a legitimate comparison between the methods to be made.

Further Research. In conclusion, the authors stressed the variation existing in different methods and cautioned against direct comparison between tests. They lauded the load-deformation relationship as one of the better ways to obtain knowledge of CFT behavior.

Sakino, K. and Hayashi, H., 1991 (c; m)

Introduction. The axial load-longitudinal strain behavior of CFT stub columns with circular cross-sections was analyzed and compared to experimental tests. The effect of different *D/t* ratios and different concrete strengths was investigated. Studies were also conducted regarding the ratio of hoop strains to longitudinal strains in the steel tube. The main objective was to estimate the effects of both strain hardening of the steel tube and the triaxial confinement of the concrete core. The introduction to the paper presented a concise summary of the nature of stresses in the components of a CFT as the load increases.

Experimental Study. Annealed steel was used such that no residual stresses existed in the tube specimens. Hoop and axial strains were carefully measured to study the volumetric expansion, or dilatancy, of the concrete at higher strains.

Analytical Study. The plastic theory developed in the paper was based on the following assumptions to calculate the relationship between axial force and longitudinal strain in the column. The concrete was isotropic, had an elastic-perfectly plastic stress-strain curve, and adhered to a nonlinear constitutive law in the elastic range; the steel stress-strain curve was represented by a five-line curve simulating test results; the steel tube was in a biaxial state of stress and the concrete was in a triaxial state where the two principal stresses, hoop stress in the steel and radial stress in the concrete, were equal; octahedral stress components were used for the concrete and the von Mises yield criterion defined the elastic limit for the steel; the associated flow rule was adopted for both the steel and the concrete; and isotropic hardening of the steel was assumed.

Comparison of Results. The analytical and experimental results agreed quite well except for the specimens containing 'high-strength' concrete (6.5 ksi) and having a large *D/t* ratio. To predict the behavior of high-strength concrete in large deformation regions, the effect of work softening must be considered. The observed maximum axial load capacities were 1.12-1.25 times the analytical capacity, with the effect of strain hardening ignored. The theoretical and experimental results showed that the ratio of the hoop strain to the longitudinal strain became greater than 0.5 under large strains, indicating the concrete dilates in the plastic region. The hoop strain to longitudinal strain ratio increased with an increase in the *D/t* ratio and increased slightly with an increase in the concrete strength. However, the theoretical values were less than the experimental, prompting the authors to suggest the need for an alternative to the assumed associated flow rule.

Sato, T. et al., 1991 (bc; r)

Introduction. A new method for determining the interaction of the materials comprising CFTs was presented in order to analyze the interactive effects of the concrete and the steel and to elucidate the issue of higher cumulative strength and ductility of the combined materials. A very detailed series of figures was included in the paper, giving clear and concise results.

Experimental Study, Discussion, and Results. The tested circular CFTs were oriented vertically and pinned at both ends. The three specimens were subjected to a transverse reversed cyclic shear force at midheight with a constant axial load. Loads were applied to a rotation angle R equal to 6.0% over 13 cycles. After this, load was applied monotonically until R equaled 10%, the limit of the testing system. In spite of local buckling, the maximum strength during loading exceeded the cumulative strength calculated using the individual material strength in the fully plastic state.

Analytical Study. Based on the results of the experiments (i.e., axial and shear forces, and deflections), the local buckling in the steel was analyzed first. By equilibrating forces at the site of local buckling and estimating the buckled wave form of the steel tube as a cosine function, the authors were able to calculate the critical strain in the steel at buckling. The material properties of the tube were obtained from tests. Axial and circumferential stresses were obtained such that the equivalent biaxial stress obeyed the von Mises yield criterion. With these values, the interactive calculation along the critical buckled section was carried out using a fiber element model, assuming linear strain distribution. To obtain the forces taken by the concrete, the results of the steel analysis were subtracted from the experimental results. Both shear forces and axial forces in the concrete could be obtained in this manner. The authors then discussed a procedure to determine the relationship between the two using a fiber element model. Also, they formulated the relationship between stress and strain in the concrete at the critical section. From the forces, the eccentric distance of the axial force (the centroid of the compression block) in the critical section may be determined.

The calculated hysteresis shape of the steel tube was similar to that of the experimental results of the CFT, indicating that the CFTs mechanical behavior is very much like that of the steel tube. After a deflection angle R = 2.5%, the axial and shear force taken by the steel gradually decreases and local buckling occurs.

A series of detailed graphs provided the results of the analytical study and compared them to the experimental values.

<u>Tomii, M., 1991</u> (no tests)

<u>Introduction</u>. A qualitative discussion was presented including the topics of bond in CFTs, the behavior of CFTs as columns in frames, and the behavior of CFT columns under various combinations of axial compression, bending, and shear. Both monotonic and cyclic loading was studied and emphasis was placed on the deformation capacity of the tubes. The discussions centered around previous tests conducted by the author and others.

<u>Experimental and Theoretical Discussion</u>. *Axially Loaded Columns*. The author included a scatter-plot of the axial load ratio to the slenderness ratio of a number of previous tests. The plot showed that the ultimate load of circular CFTs is considerably larger than the nominal load, which is the sum of the two component strengths. This is due to strain hardening of the steel and confinement of the concrete. Although the confinement effect diminishes with increasing column length and is generally neglected for columns of practical length, it ensures that the column behaves in a ductile manner, a distinct advantage in seismic applications. Previous tests by the author and others resulted in a classification based on test parameters including cross-section shape, *D/t* ratio, and concrete strength. Three categories emerged: strain hardening type, elastic-perfectly plastic type, and degrading type, which was characterized by a loss of load carrying capacity after the yield point was reached. All of the square columns that were tested exhibited a degrading type of axial load versus strain curve. The circular tubes all produced either strain hardening or elastic-perfectly plastic type curves. The author proposed using the yield load rather than the ultimate load as the capacity of the section, because the ultimate load was much more sensitive to the effects of varying the slenderness ratio and the *D/t* ratio.

Beam-Columns. Tomii and Sakino (1979a) conducted tests on short beam-columns to obtain moment-thrust-curvature relationships for square CFTs with an increasing moment under constant axial load. Despite the occurrence of local buckling in the steel tube, the CFT was more ductile than a comparable reinforced concrete column. The paper discussed an earlier method proposed by the author and Sakino of determining the axial load-moment interaction diagram. This "simple method" used for square columns is based on simple rectangular stress blocks for the tube and the concrete core without a limiting concrete strain since the concrete stress-strain curve used was very ductile (Furlong proposed a more complicated method involving a limiting crushing strain of 0.003). Limited details were shown regarding this method, but it was stated that it could also be used conservatively for circular cross-sections.

Columns under Compression, Bending, and Shear. Members under this combination of loading are very important in seismic applications. Although designs follow the weak beam-strong column philosophy, many instances arise where the base of columns are subjected to high cyclic shear. Eighty tests of short (columns in which shear cannot be neglected) and medium length square columns bent in double curvature were studied, the details of which were omitted in the paper's discussion. For monotonic tests, two failure mechanisms occurred. The short columns (with a shear span to depth ratio of 1.0 or smaller, where the shear span is *L/2*) showed a shear failure, while medium columns (shear span to depth ratio of 1.5 or larger) produced a flexural failure. Therefore a critical value existed somewhere between 1.0 and 1.5. For the cyclic load tests of square tubes, which were conducted under a high axial load ratio (0.5), the specimens exhibited a small amount of strength degradation, but the hysteretic loops were very stable and a large amount of energy was dissipated. The strength degradation was due to the local buckling of the tube walls at the points of maximum shear stress (the top and bottom of the column) and a subsequent crushing of the concrete at those locations. The tube becomes circular at the buckling/crushing points, which stabilizes the behavior of the column. The short columns, which failed by shear, showed less strength deterioration that the medium columns, which failed by flexure. An additional problem with the local buckling of the steel and the crushing of the concrete was a considerable axial shortening (up to 30% of the column depth). Therefore, the magnitude of the axial load should be limited to mitigate this effect.

Columns in Frames. The author discussed frame tests conducted by Matsui (1985), which compared square concrete-filled steel tubes and square hollow steel tubes. The frames were devised such that the plastic hinges would form in the columns, thus making the CFT or HT columns the critical members. A considerable improvement in both strength and ductility was shown by filling the hollow tubes with concrete. Local buckling of the steel tube, which produced severe strength degradation in the hollow tube tests, did not affect the behavior of the CFTs. The post-local buckling behavior was strongly influenced not only by the difference in the local buckling mechanism between the CFT and HT, but more so by the transfer of axial load from the steel to the concrete in the CFT. This greatly enhanced the energy absorption capacity of the CFT.

Bond. In practice, only limited bond strength is realized. Therefore the full composite action cannot be counted on. The author has proposed design criteria to transfer axial force from the steel tube to the concrete. For long-term loading, a portion of the column should have a zero bond stress resultant such that the concrete core can carry as much long-term axial compression force as possible. For seismic loading, the concrete core should carry the maximum compressive force at the critical sections at the top and bottom of the column such that the CFT can be designed for a moment corresponding to this axial force. Bond tests performed by Morishita et al. (1979a, b) showed that the concrete strength had an insignificant effect on the bond strength and the bond strength was considerably less than that observed between reinforcement bars and concrete. Their tests resulted in the following recommendations for the bond strength between the steel and concrete: for long term monotonic loading -- 21.8 psi for circular sections and 14.5 psi for rectangular; for cyclic loading -- 32.0 psi for circular sections and 21.8 psi for rectangular. They proposed using either a checkered internal tube surface or an expansive concrete to improve bond, the latter of which was deemed more effective.

<u>Tsuji, B. et al., 1991</u> (c; m)

<u>Introduction</u>. The interaction between the steel and concrete in CFTs subjected to axial compressive loads was examined. Considering a triaxial state of stress for concrete and a biaxial state of stress for steel, the authors derived separate constitutive equations for the two materials in an attempt to mimic the behavior of CFTs. Using these constitutive relationships, the authors plotted axial load versus axial strain. Finally, axial compression tests were carried out on short CFTs and compared to the theoretical results derived from the constitutive equations.

<u>Experimental Study, Discussion, and Results</u>. The limited number of experiments performed resulted in ultimate strengths beyond those predicted by summing the individual strengths of the CFT components. This indicated the presence of interaction between the steel and concrete. Up to a strain of 0.001, the apparent Poisson's ratio (measured as the relationship

between the strains in the longitudinal and circumferential direction) remained constant at about 0.3. As strains increased to 0.003, the apparent Poisson's ratio increased from 0.3 to 0.8. Beyond this point, Poisson's ratio remained constant at 0.8 for additional strain. These values of Poisson's ratio were used in the concrete stress formulation described below. It was also found that significant strength gains were achieved due to the interaction of the steel and the concrete. The strength corresponding to the 0.2% offset strain and the strength at 2.0% strain were 16% and 33% larger, respectively than the strengths obtained by simply summing the individual strength contributions of the two materials.

Analytical Study. Using constitutive equations, the authors attempted to model the interactive behavior of the concrete and steel in CFTs. The Drucker-Prager yield condition in strain space was employed for analysis of the concrete. The associated flow rule was adopted whereby plastic strain vectors are normal to the yield surface specified by the Drucker-Prager equation. It was also assumed that only plastic flow occurs in the plastic range, and that there is no fracture. The stress versus strain relationship for the steel tube followed the von Mises yield criterion and the Prandtl-Reuss isotropic hardening rule. In analyzing the behavior of axially loaded CFTs, the authors looked at the experimental behavior to gauge the amount of interaction. For strains less than 0.001, where Poisson's ratio began to increase, no contact existed between the materials and simple superposition of stresses was made. For additional straining, the concrete became inelastic and contacted the steel, initiating interaction between the materials. Beyond the point of interaction, the longitudinal stresses in the concrete and the steel remained equal, as did the circumferential and radial strains in the steel and concrete, respectively. The authors imposed an equation of equilibrium for this condition to relate the stress of the steel in the circumferential direction (σ_{sc}) and the radial stress in the concrete (σ_c):

$$2 \cdot t \cdot \sigma_{sc} + (D - 2 \cdot t) \cdot \sigma_c = 0$$

where D is the tube diameter and t is the thickness of the tube.

Comparison of Results. The experimental and analytical results both showed smooth axial force versus axial strain curves, indicating stability in the section. The effectiveness of the analytical procedure was demonstrated by the general agreement with the experimental results.

Xu, J. et al., 1991 (c, ct; m)

Introduction. Experimental research on medium and long columns subjected to compressive loading, torsional loading, and a combination of the two was reported. The slenderness ratio and the steel to concrete area were varied and the results examined. The authors also presented a theoretical analysis based on the theory of mechanics, which was, for the most part, referenced to previous papers and given little coverage in this paper.

Experimental Study, Discussion, and Results. *Axially Loaded Columns*. The short column (L/D = 7) loaded under pure axial compression failed beginning with yield lines at a 45 degree angle on the tube, followed by a slight bending of the tube and an outward bulging of the steel. The medium length column (L/D = 13) failed in a similar manner with no bulging of the steel and a much smaller ultimate strain. The long column (L/D = 20) lost its bearing capacity immediately and seemed to buckle in the elastic range.

Torsionally Loaded Specimens. Failure in the torsion specimens was not abrupt and did not occur at a well-defined point; it was therefore assumed to occur when the rotation angle increased by a large amount for relatively little increase in load. The rotation angle beyond which little or no increase in torsional moment was observed was called the critical rotation angle. For the short, medium, and long columns the critical rotation angles were 5°, 9°, and 14° respectively. The ultimate torsional moment resistance decreased with an increase in the axial load ratio. The greatest torsional moment was attained in the pure torsion case. The characteristic failure mechanism was a cracking of the concrete followed by a propagation of the cracks along the length of the tube in a spiral pattern. The cracked concrete sustains the axial load and actually produces additional axial load while the steel picks up the torsional moment through tensile resistance. This "spiral effect", in which the concrete produces axial force under the influence of a torsional moment, results in a specimen with high ductility.

Kitada, T. and Nakai, H., 1991, and Kitada, T., 1992 (no tests)

Introduction. A review of the ultimate strength and ductility of circular and rectangular concrete-filled steel tubes under a number of different types of loading was presented. Included in the review were discussions on columns, beam-columns, beams, members subjected to shear, and members subjected to torsion. Both monotonic and cyclic loading were examined. The authors cited a number of advantages of using CFTs: the buckling of the outer steel tube may be prevented by the concrete core; the concrete core's strength is augmented by the confining effect of the steel; the combined effect of the latter two advantages affords an increase in the ultimate load carrying capacity of the section. The CFT also has a greater ductility than steel or reinforced concrete.

Theoretical Discussion. *Short Axially Loaded Columns*. The authors compared three cases: a composite section with uniform loading of the steel and concrete, a composite section with only the concrete loaded, and a hollow steel tube alone. They showed that the compressive load of the two composite sections gradually increases (to the limit of their test) with increasing axial displacement, even in the region of large displacements, while the steel section shows a decrease after its smaller ultimate load is achieved. The composite section was shown to have 20% more strength than the sum of the individual steel and concrete components. Rectangular sections will not exhibit the increased strength of the circular sections, but will, however, still exhibit a substantial ductility.

Beams. CFT sections, both circular and rectangular, show substantial increases in strength and especially ductility over comparable steel sections, both with slip between the concrete and steel restricted and unrestricted. With the slip at the ends of the beam restricted, a significant gain in ductility can be achieved. Curves examining cyclic behavior showed that the decrease in strength with subsequent cycles in hollow square tubes could be alleviated by filling the tube with concrete. The CFT hysteresis loops were much more stable and showed little strength deterioration.

Short Beam-Column. The authors illustrated the advantage of using a circular CFT cross-section. The circular section has the ability to sustain a larger moment combined with an axial load. Also, the rectangular section loses its ductility under combined axial load and bending.

Shear. Graphs were presented to illustrate a short CFT's superior ductility over a comparable reinforced concrete section under the effect of a cyclic shear load. Again, the circular section exhibited a somewhat greater ductility than the rectangular section.

Torsion. The ultimate torsional moment of a CFT section is about 1.2 times the sum of the individual torsional resistance of the steel and concrete. The ultimate torsional moment can, however, be accurately predicted by assuming that the ultimate shearing stress of the concrete is equal to $f'_c/2$. Both steel and composite specimens with rectangular cross-sections show good ductility. The circular CFT section, however, shows a superior ductility over the hollow steel tube and a much larger ultimate torsional moment. The behavior of the circular section is decidedly different from the rectangular section. In the circular section, the components behave independently. Therefore the torsional rigidity will rival that of the steel tube until yield, at which point the two materials behave compositely.

Rangan, B. V. and Joyce, M., 1992 (bc; m)

Introduction. The results of tests performed on nine eccentrically loaded, slender, circular CFTs filled with high-strength concrete were reported. The two main test parameters were the slenderness ratio and the eccentricity of axial thrust. A simple method to calculate the strength of columns was presented and it showed good correlation with the test results reported in the paper and those reported by Neogi, Sen, and Chapman (1969).

Experimental Study, Discussion, and Results. All of the specimens failed at midheight due to the crushing of concrete in compression. The extreme tensile fibers in the steel did not reach yield for the low values of eccentricity, but did in the other cases. Extreme fiber compressive strains ranged from 0.002 to 0.004.

Analytical Study. The method for calculating the axial load capacity of eccentrically loaded slender CFT columns was based upon the assumption that the failure load was reached when the maximum moment at midheight of the column was equal to the ultimate bending strength of the cross section at that location. This value was determined by iteratively computing the internal and external moments until equilibrium was established. The computation of the external moment included the effects of creep, initial eccentricity, and initial imperfections. The sum of the midheight deflections due to these effects and the deflection due to the load was multiplied by the axial load to get the external moment. The internal moment was computed by idealizing the cross-section. Linear strains over the cross-section were assumed with a maximum compressive strain of 0.003 in the concrete. Stress resultants acting at the centroids of the tensile (steel) and compressive (steel and concrete) regions were computed based on the distance to the neutral axis and the constitutive relation (Hognestad's parabola). The internal forces were computed based on these stress values multiplied by their respective areas. To relate the internal and external moments, the authors assumed a deflected shape in the form of a sine curve. Curvature was calculated by dividing the extreme fiber strain by the neutral axis distance. From this, deflection was calculated using the sine curve assumption and the midheight relationship between curvature and deflection. The neutral axis was adjusted until moment equilibrium was achieved.

Comparison of Results. The results from the formula were conservative compared to the authors' tests and tests from other investigators. For columns with small eccentricities, the results were very conservative, probably due to assuming that the concrete crushed at a strain of 0.003. In reality, the more concentrically loaded columns will attain higher concrete strains. Neglecting the low eccentricity tests, the mean value of test/calculated for these 21 specimens was 1.08 with a coefficient of variation of 5% (1.17 and 16% for all 27 compared tests).

<u>Sugano, S. et al., 1992</u> (bc; r)

<u>Introduction</u>. This paper presented a brief overview of several tests of square and circular CFT beam-columns subjected to cyclic lateral loads. The strength and ductility of the beam-columns were examined with respect to the predicted capacities presented in the Japanese standard [Architectural Institute of Japan (AIJ)] which does not account for the confinement of the concrete.

<u>Experimental Study, Discussion, and Results</u>. Thirty-eight tests were performed; the test parameters were the type of tube (circular or rectangular), the amount of axial load, the width-to-thickness or diameter-to-thickness ratio (*D/t* ratio), and the strength of the materials. The cyclically loaded circular columns exhibited large hysteresis curves. The effect of the *D/t* ratio on the ultimate load was minor. For *D/t* ratios greater than 39, local buckling governed the ultimate capacity of the section, but the load did not decrease substatially after buckling had occurred. For smaller *D/t* ratios, the load continued to increase after local buckling. In the square tubes, local buckling determined the ultimate capacity for *D/t* ratios greater than 33. The square columns exhibited larger hysteresis curves for smaller *D/t* ratios, concrete strengths, and axial load ratios. The authors also observed rupturing of the steel tube at unspecified loads beyond ultimate.

Both the square and the circular sections provided a much larger amount of strength than that predicted by AIJ which using a simple superposistion of the strengths of the concrete and the steel. Ultimate strengths of 1.2-4.5 and 1.2-3.5 times the predicted value were realized in the tests of the circular and square CFTs, respectively. The strength enhancement due to the confinement of the concrete was larger in the circular sections.

The ductility of the CFT sections was gauged by the amount of displacement the beam-column could undergo while maintaining at least 95% of the ultimate load. The circular beam-columns showed a larger ductility than the corresponding square beam-columns (a rotation of 3.0-6.0% versus 2.2-3.3% for an axial load ratio of 0.3). Additionally, the ductility of the square sections decreased at a more rapid rate as the axial load ratio was increased. The higher strength materials produced marked decreases in ductility for both section types, as did an increase in the *D/t* ratio.

In conclusion, the authors reiterated their finding that both circular and square beam-columns show a significantly enhanced strength due to confinement, and that this fact must be accounted for in the design procedure. They also concluded that circular beam-columns are quite ductile even when subjected to high axial loads and square beam-columns are ductile for low values of axial load.

Bridge, R. Q. and Webb, J., 1993 (c; m)

Introduction. The experimental study contained in this paper was prompted by the desire to use thin-walled steel tubes filled with concrete for the columns in a 43-story high-rise in Melbourne, Australia. The use of CFTs proved economically superior to equivalent steel or steel-reinforced columns. Not only were the material costs less, but there was no need for concrete formwork and the construction of the steel framework could proceed several stories above the filling of the steel tubes. The steel tubes were filled by pumping the concrete into the bottom and then up the steel tube. As many as six stories were filled at a time. The most economical CFT would be a column in which only enough steel to support the framework during construction was required. The authors conducted the tests based on this efficiency of thin-walled tubes.

Experimental Study, Discussion, and Results. The tubes used in the construction of the aforementioned structure had *D/t* ratios on the order of 120, in excess of the limits specified by the Australian Standard AS4100-1900 and Eurocode 4 [see Roik and Bergman (1993) for the Eurocode 4 *D/t* limits]. Therefore, the authors proposed an effective diameter, D_e:

$$D_e = \sqrt{\frac{82}{\frac{D}{t} \cdot \frac{f_y}{250}}} \cdot D$$

to account for local buckling and conducted experimental tests to verify the accuracy of using an effective diameter.

Two CFT tests and two HT tests were conducted using high-strength concrete with suitable pumping characteristics. The CFTs were loaded into the post-ultimate stage. Local buckling began prior to the point of ultimate load, although at initially small magnitudes of deformation. Beyond the ultimate load, the entire steel tube buckled locally near midheight and axial shortening was concentrated in this region.

The axial load capacity using the effective diameter for the steel tube was expressed by:

$$P_o = A_e \cdot f_y + A_c \cdot f_c^{'}$$

where A_e is the effective area of the steel tube using the effective diameter, D_e. The results of this calculation provided a conservative estimate of the strength and the authors confirmed the appropriateness of their equation. The corresponding tests on the HT sections showed that the steel was only contributing 10-12% of the compressive strength of the column. Aside from supporting the construction loads prior to being filled with concrete, the tube was primarily used to contain the concrete.

Bridge, R. Q. and Yeung S., 1993 (no tests)

Introduction. The authors presented a design formula to determine the limiting slenderness (*L/r*) of a 'short' composite column. Columns defined as 'short' are columns which can be designed for their full cross-section strength without considering the strength-reducing effects of stability. In their formulation, the authors considered the effects of cross-section type (CFTs and SRCs of different sizes), slenderness ratio, initial imperfections, loading eccentricities, and the ratio of applied end moments. An inelastic non-linear column analysis was used to verify the proposed equation.

Theoretical Discussion. The proposed equation is as follows:

$$\frac{L_s}{r} = 74 \cdot (\beta + 1) \cdot \left(1 - \frac{P}{P_o}\right)$$

where L_s/r is the limiting slenderness ratio for a composite column to be considered 'short'. P is the applied load, P_o is the cross-section strength, and β is the ratio of end moments. This equation obviates the need for a column curve (which relates the slenderness ratio to the axial load ratio). First, a short column slenderness λ_s was defined as:

$$\lambda_s = \gamma \cdot \left(\frac{L_s}{r}\right)$$

where

$$\gamma = \frac{1}{\pi} \cdot \sqrt{\frac{P_o}{E_s \cdot A_s + E_c \cdot A_c}}$$

For practical steel ratios (A_s/A_c) and concrete strengths between 3 and 7 ksi, the parameter γ only varies from 0.0087 to 0.0113, and was taken as an average, 0.01. The authors further assumed a conservative approximation to the column curve as

$$\frac{P}{P_o} = 1 - \frac{\lambda}{2}$$

Then, by substituting these values into an equation proposed by Rotter (1982):

$$\lambda_s = 0.37 \cdot (\beta + 1) \cdot \lambda$$

the above equation was obtained.

The parametric study verified the validity of this expression. For columns bent in double curvature, the formula proved accurate. For single curvature, it was conservative. Columns bent in symmetric single curvature by equal end moments (β = -1) are very sensitive to initial imperfections and the column can never be considered short because the moment at midheight is always magnified. The effect of initial-out-of-straightness diminishes, though, as the end moment ratio increases and the column approaches double curvature. Changing the parameter β may have a significant effect on the slenderness ratio. A change in the load P will also induce a large change in the slenderness ratio. The effect of concrete strength in both CFTs and SRCs was negligible.

Kawaguchi, J. et al., 1993 (bc; r)

Introduction. The results of 26 square cantilever beam-column tests were presented. Twelve HT tests and 14 CFT tests were conducted. Each specimen was loaded axially and subjected to an alternately repeated transverse load at the top of the beam-column. The effect of filling the steel tubes with concrete on the energy dissipation capacity and the number of load repetitions the member could withstand was discussed. The strength deterioration of the specimens due to local buckling of the steel tube was also investigated. A number of detailed hysteretic curves including an indication of the point of local buckling were presented as well. The parameters that were varied in the tests included the D/t ratio, the axial load ratio, and the amount of lateral displacement the beam-column was subjected to, expressed as the ratio Δ_{max}/Δ_{cr}.

Experimental Study, Discussion, and Results. Each specimen was loaded transversely until the specified displacement, Δ_{max}, was reached. This value was varied from $0.6 \cdot \Delta_{cr}$ to $1.3 \cdot \Delta_{cr}$ for different tests. After the specified displacement was reached in one direction, the load was reversed and the beam-column was displaced an equal amount in the opposite direction. Ten cycles of alternately repeated load were applied at this constant displacement amplitude. The displacement at the occurrence of local buckling decreased with an increase in axial load. The magnitude of the displacement at which local buckling of the steel tube occurs was 1.4 times greater for the concrete-filled tube than for the hollow tube for a D/t ratio of 22.1, and 1.9 times greater for D/t = 31.3. The hollow tubes could not, in some cases, sustain sufficient axial strength to undergo 10 cycles without failure. The concrete-filled tubes exhibited less strength deterioration and more energy dissipation capacity than comparable hollow tubes. Plots of the maximum strength versus load cycle showed how the CFT was able to sustain its strength, reaching a steady state after local buckling, while the hollow tube's maximum strength dropped, sometimes significantly, with each cycle due to strength deterioration. Although the concrete did not contribute much strength in bending, it increased the energy dissipation mainly by delaying the onset of local buckling of the tube. The value of the axial strain at the occurrence of the local buckling was not greatly affected by the axial load ratio.

Analytical Study. Stress-strain curves and corresponding formulas were presented to model the behavior of the individual components of the CFT. The formulas for the steel traced linear elastic and linear strain-hardening curves with a curvilinear elasto-plastic portion between the two linear curves. Also presented was a function tracing the degradation following local buckling. The concrete curve followed Popovics's relation and it was assumed that the maximum strength was maintained due to confinement. The beam-column was modeled by dividing the cross-section into fiber elements to determine load-deflection and moment-curvature diagrams analytically. The model of the beam-column cross-section consisted of two types of fibers: a series of fibers in the tube and a series of fibers, each of which is a transverse layer, to model the concrete. It was assumed that the stress in each concrete layer was uniformly distributed and plane sections remained plane after bending. The true value of the centroidal strain (calculated from a given curvature) satisfying equilibrium with the applied axial force was obtained by a trial and error method. From the strains, the moment in the section was calculated, giving the moment-curvature relationship. The transverse load-displacement relation was calculated assuming that the deformable portion of the beam-column was located over a width D at the base of the specimen and the remainder of the beam-column was rigid. Once a moment-curvature relation was established, the applied transverse load could be calculated without trial and error from the deflection. The deflection was obtained from a formula expressing it in terms of the deformable length D and the amount of curvature at the base:

$$\Delta = \phi \cdot D \cdot \left(L - \frac{D}{2} \right)$$

where *L* is the member length and ϕ is the base curvature. The calculation of the transverse load H followed:

$$H = \frac{\left(M(\phi) - P \cdot \Delta \right)}{L}$$

where *M(ϕ)* is the moment as a function of curvature and *P* is the applied axial load. The analytical method presented predicted the experimental behavior well. However, it did not model the Bauschinger effect, i.e., the gradual softening of the stiffness beyond the elastic region.

Morino, S. et al., 1993 (fr; r)

Introduction. This paper presented a study of three-dimensional cruciform subassemblies composed of four steel girders framing into a CFT beam-column. Axial load was applied to the top of the CFT beam-column, constant loads were applied to the ends of the two girders framing into the minor axis of the beam-column, and anti-symmetric, cyclic loads were applied to the girders in the major axis direction of the beam-column. The CFT/steel connection was designed for two modes of failure: shear failure of the connection and flexural failure of the beam-column. In addition to presenting the test results, the paper discussed the hysteretic behavior, the maximum strength, the energy dissipation, and the failure configuration of each subassembly.

Experimental Study, Discussion, and Results. In all, ten specimens were tested, five failing by connection shear, and five by flexural failure of the CFT column. Each subset of tests consisted of a planar specimen (no out-of-plane girders), a specimen with no loads on the out-of-plane girders, one specimen with equal constant applied loads on the out-of-plane girders, and two final specimens with unsymmetric loads (biaxial bending of the CFT beam-column). The load applied to the in-plane girders was anti-symmetric (i.e., one beam was loaded in an upward direction while the opposite beam was loaded in a downward direction, then the loads were reversed). The specimens were cycled by increasing increments of rotation until failure or the limit of the testing apparatus was reached.

The specimens loaded uniaxially (either no out-of-plane loads or symmetric loads) exhibited a larger deformation capacity. Nonetheless, the biaxially-loaded specimens achieved rotation angles of $R = \pm 4/100$. The subassemblies failing at the connection showed slight pinching behavior, but no significant strength deterioration. Deformation of these specimens consisted of an S-shaped pattern in the plane of cyclic loading with yielding above and below the connection. The connection displaced laterally in one direction, but the column was able to sustain the axial load until the limits of the appartus were reached.

The column-failing specimens exhibited unstable deformations under three-dimensional loading when the rotation approached ±3/100 to ±4/100. These specimens also produced a larger strength degradation than the connection-failing specimens. Two types of failure mechanism were observed. The first, which occurred in the symmetric cases, consisted of local buckling of the CFT beam-column above and below the connection. The connection then displaced laterally leading to column instability. The second type of failure was similar to the first except that the beam-column underwent large plastic deformations below the connection in the out-of-plane direction. The result was again a failure due to lateral displacement of the connection area, leading to an instability failure.

The connection fainling specimens show less strength degradation, a larger deformation capacity, and more stable hysteresis loops than the comparable column-failing specimens. Biaxial bending of the CFT beam-column (when asymmetric loads are applied to the out-of-plane beams), results in a noticeably smaller deformation capacity in both types of structures because the bending axis rotates with repeated loading which causes earlier local buckling of the tube.

Roik, K. and Bergmann, R., 1993 (no tests)

Introduction. Two methods for the analysis and design of composite columns were presented in some detail. The first method covered a simplified method of analyzing and designing composite columns under compression, bending, and shear, as specified by Eurocode 4. Following the commentary on this code were a number of examples for both steel-reinforced concrete columns and concrete-filled steel tubes. The second method presented was an incremental method of analysis structured for computer calculation. In conclusion, experimental results compiled from different sources were compared graphically to the presented methods and showed accurate correlation.

Analytical and Design Formulation. The authors cite a number of formulas from the Eurocode. Limiting ratios of section depth to thickness were specified to prevent local buckling of the steel shell. For rectangular sections:

$$\frac{D}{t} \leq 52 \cdot \sqrt{\frac{235}{f_y}}$$

and for circular sections:

$$\frac{D}{t} \leq 90 \cdot \left(\sqrt{\frac{235}{f_y}}\right)^2$$

The value of the yield strength, f_y, in these equation is in metric, with units of MPa. The resistance to axial force is specified by:

$$P_o = \frac{A_s \cdot f_y}{1.1} + \frac{A_c \cdot f_{cu}}{1.5}$$

where the 1.1 and 1.5 are safety reduction factors. The factor of 0.85 usually applied to the concrete strength is omitted because the steel encasement provides confinement effects and the protects the concrete from environmental effects. A presentation of combined moment-axial load calculations based on plastic stress blocks and the computation of the neutral axis location is detailed for composite sections (both CFT and SRC).

Matsui, C. et al., 1993 (c, bc; m)

Introduction. This paper, along with El Din et al. (1993), reports the results of a series of 30 tests on long CFT and HT beam-columns under eccentric and concentric axial load. While the first paper gives a summary of the results and important conclusions, the second paper presents a more detailed description of the test parameters, materials, and experimental results. The design formulas from the Japanese code (AIJ) are compared to the experimental results, and the effect of column slenderness is discussed.

Experimental Study, Discussion, and Results. Thirty column and beam-column tests were performed on CFT and HT sections. The column tests were performed with simple support conditions, and the beam column tests were on pinned end columns with applied eccentric loading. The main test parameter was the slenderness ratio *Lk/D* (= 4, 8, 12, 18, 24, and 30). The secondary parameter was the eccentricity of the axial load, with four cases tested for each column group (e = 0, k, $3k$, and $5k$, where k is the eccentricity required to induce tension at the extreme fiber of the cross-section). One column test from each group was performed on a HT under concentric loading. The tubes were all $150 \times 150 \times 4.5$ mm. square sections filled with 4.6-5.0 ksi concrete.

The specimens were instrumented to record strains and deflections at the midhieght of the tube. The tests were terminated when a stable post-ultimate load was achieved or when the deflection limit of the test rig was reached. The results are presented as plots of load verses midhieght deformation, with the results of a material nonlinear analysis plotted for comparison. The HT test and analytical results varied the most due to local buckling and other geometric nonlinear effects. The CFT tests, however, showed very good correlation between analysis and experiment. The largest difference was in the post-ultimate region when the analysis tended to give results which were below the experimentally measured values. This was due in part to the material models chosen for the analysis, as the concrete model did not allow for a plateau of the stress-strain diagram for large strains [such as in Tomii and Sakino (1979b)].

The authors note that the limiting slenderness ratio for negligible geometric nonlinear effects is approximately 12. The HT limiting slenderness ratio is also observed to be around 12. The authors describe three modes of failure which depend on the eccentricity and the slenderness of the section. The first is a squashing failure, which is observed for short columns (*Lk/D* < 12) with low to moderate eccentricities. This mode of failure is dominated by local buckling of the steel section at the ultimate load of the CFT. For short sections with large eccentricities, the critical load is also reached as local buckling is initiated. The second mode of failure is observed in longer columns which have a moderate to high eccentricity. The moment in the section at ultimate load is near the capacity of the section, and the author attributes failure to a three-hinge beam mechanism. For long columns (*Lk/D* > 18) under concentric loading, the dominant failure mode is flexural buckling. Finally, the authors compared the Japanese code recommendations for ultimate capacity of CFT and HT sections with the experimental results. They found that the code was very conservative when applied to CFTs, but less conservative for HTs.

Lu, Y. Q. and Kennedy, D. J. L., 1994 (pb; m)

Introduction. Pure bending tests were performed on rectangular CFTs to determine the effect of different width to depth ratios and different shear span to depth ratios. The authors were interested in the effect of different ratios of concrete to steel in the compression zones, and what affect (if any) this would have on the stiffness and ultimate strength of the members. Through investigation of results, it was found that the concrete in the compression zone was able to reach its full cylinder strength rather than 85% of f'_c (the common assumption made in ultimate strength design of RC members) The transfer of shearing forces from the steel to the concrete through bond was investigated by loading configurations which induced a variety of shear span to depth ratios. The beams were instrumented to measure bond slip between the two materials during loading at various points in the beam depth. It was found that bond degradation did not occur until the onset of concrete crushing in the compression zones, and did not effect the ultimate load carrying capacity of the CFT beams.

Experimental Study, Discussion, and Results. *Column Tests*. Five HT stub columns were tested to determine the average steel σ-ε curve. The σ-ε curve for concrete was obtained from cylinder tests. Five CFT stub columns were tested and the σ-ε curve obtained was plotted against the linear superposition of the HT and concrete curves. The results varied, depending upon the *D/t* of the cross section. CFTs with low *D/t* ratios carried a higher load than the superposition of steel and concrete, indicating some increase in concrete strength due to confinement. CFTs with high *D/t* ratios actually carried less load than the superposition of steel and concrete, perhaps due to the biaxial state of stress in the steel coupled with local buckling effects.

Beam Tests. 5 HT tests and 12 CFT tests were performed under simple loading conditions. The *D/t* ratio, *L/D* ratio, and shear span-to-depth ratio (z) were the main parameters. Concrete strength ranged from 5.9 to 6.8 ksi. Results of the HT beam tests compared well with the predicted ultimate capacity of the sections. Results for the CFT sections ere presented as moment-curvature plots. These plots showed a region of linear behavior followed by gradual yielding which approached a limiting ultimate moment value. When compared to the HT tests, the CFT tests showed a moderate increase in strength, but a dramatic increase in ductility due the prevention of local buckling by the concrete core.

Slip. The slip recorded at the steel-concrete interface was observed to be very small up to the point of sudden concrete crushing which accompanied the local buckling of the steel compression flange. For small z values, the author suggests shear transfer through a tied-arch model, whereby a compression strut develops in the concrete to transfer the applied load to the support. For larger z values, shear transfer occurs through bond between the steel and concrete. In no case did the z value adversely affect the ultimate moment capacity of a CFT section.

Sakino, K., 1995 (bc; m)

Introduction. Circular CFTs were tested to investigate their flexural behavior. The moment capacity of the specimens was examined and the test results were compared with the AIJ (1987) design code provisions.

Experimental Study Results and Discussions. Twenty-eight circular CFT specimens were tested under constant axial load and monotically increasing uniform bending moment. Low, medium, and high strength materials were used to manufacture the specimens. The steel yield strength and concrete compressive strength ranged from 41.2 to 121.1 ksi and from 3.55 to 11.26 ksi, respectively. Different *D/t* ratios were selected for each steel grade, varying from 26.9 to 152. The axial load ratios (P/P_o) were chosen between 0.15 and 0.8. The *L/D* ratio was constant and equal to 3. The ends of the specimens were welded to 1.57 in. thick end plates. The end plates were attached to cylindrical bearings to simulate pinned end conditions.

It was observed that the *D/t* ratio and axial load ratio were the dominant factors affecting the flexural behavior. The specimens having *D/t* ratios larger than 100 showed a rapid decrease in strength. However, the response was ductile when the *D/t* ratio was lower than 50. The specimens having high strength steel, high strength concrete, and a low *D/t* ratio also achieved ductile response, even under high axial loads. Excluding the specimens with small *D/t* values, local buckling was observed for most of the specimens, and it generally took place either at the peak point or in the post-peak region of the load-deformation responses.

Analytical Study. Interaction curves for the specimens were obtained analytically using the stress block approach presented in the AIJ (1987) design code provisions. Two moment-thrust analyses were performed for the specimens. In the first one, no reduction factor for concrete strength was introduced. However, in the second one, a 0.85 reduction factor was applied to account for the concrete stress block. It was found that the analysis with full concrete strength gave conservative strength estimates for low *D/t* ratios but unconservative strength estimates for high *D/t* ratios. The enhancement in capacity observed in the case of low *D/t* ratios was attributed to the confinement of the concrete and strain hardening of the steel. On the other hand, the analysis that included a reduction in the concrete strength predicted the capacity of the specimens conservatively. However, the capacity predictions for the low *D/t* specimens were found to be too conservative.

Fujimoto, T. et al., 1996 (bc, sh; r)

Introduction. This paper presents tests results of CFT columns subjected to combined axial load and cyclic shear. The objectives of the tests were to study the cyclic load deformation behavior and determine ultimate strength and deformation capacity of CFTs.

Experimental Study, Results, and Discussion. The main test parameters were tube shape (square and circular), *D/t* ratio, strength of steel and concrete, axial load ratio P/P_o, and loading angle for biaxial bending. Thirteen circular and twenty square CFTs were subjected to axial load and cyclic shear load. The *D/t* ratio of the specimens varied between 17.8 and 53.3. The ranges for the measured yield strength of steel and measured compressive strength of concrete was 58.0-

113.1 ksi and 5.80-13.05 ksi, respectively. The *L/D* ratio was 6 for all of the specimens. Axial load was varied from $0.7 \times P_o$ to negative $0.3 \times P_o$ for four circular and four square specimens, and was held at $0.4 \times P_o$ for the rest of them. The test setup simulated fixed end conditions and induced double curvature in the specimens. Cyclic shear loading was displacement controlled. Square steel tubes were built of two channel sections by welding, while channel sections were cold formed from flat plate. No annealing of steel tube was reported; thus the steel tube was likely to have residual stresses due to both cold forming and welding. Moment-chord rotation and rotation-axial strain diagrams were shown in the paper. The rotation capacity of square CFTs was observed to be larger for higher steel strength, smaller *D/t* ratio, and smaller concrete strength, with loading angle effect being unclear. No clear effect of these factors on the circular CFT rotation capacity was observed.

Axial strain of the square CFTs having a constant axial load was higher for higher concrete strength, lower steel strength, and larger *D/t* ratio. Effects of steel strength and *D/t* ratio on square specimens with variable axial load were unclear. Axial strain in biaxial bending was lower than in uniaxial bending. For circular CFTs having a constant axial load, the value of axial strain was higher for lower steel strength, lower concrete strength, and larger *D/t* ratio, while the influence of steel strength and *D/t* ratio was unclear for circular specimens with variable axial load.

Analytical Study. Maximum strength of all specimens plotted against the interaction diagrams calculated for ideal full plastic state of stress was shown in the article. Experimental strength exceeded the interaction diagram values, except for several specimens with a variable axial load.

Several conclusions were drawn by the authors. The rotation capacity and axial strength of circular specimens exceeded those of square specimens. Axial strain was larger for square specimens. Rotation capacity and maximum strength were smaller for the variable axial load specimens, and were not affected by the loading angle.

Lundberg, J. E., Galambos, T. V. 1996 (no tests)

Introduction. The objective of this work (Lundberg, 1993; Lundberg and Galambos, 1996) was to determine the reliability index for composite columns and beam-columns, i.e., determine the probability that failure of a member will not occur. A large number of test results were analyzed to provide a statistical base for the reliability study. Results of steel-reinforced concrete shapes and rectangular and circular concrete-filled steel tubes were examined, the latter two of which will be highlighted in this summary.

Discussion of Results. The experimental results were compared to the AISC LRFD Specification (1993) with the flexural and compressive resistance factors set to 1.0 to allow a direct comparison between the predicted and actual strengths. The data from 243 tests of circular CFTs (161 columns and 82 beam-columns) and 68 rectangular CFTs (16 columns and 52 beam-columns) was collected, tabulated, and analyzed. The analysis of the columns resulted in a mean ratio of test load to predicted load (P_u/P_o) of approximately 1.3 with a standard deviation of 0.23, reflecting the large scatter of the results. These values decrease if the results of the stub columns are omitted. Circular columns had a mean ratio of 1.5 with a standard deviation of 0.21 and rectangular columns had a mean ratio of 1.1 with a standard deviation of 0.12. A number of the rectangular sections had ultimate strength values below the AISC Specification. The beam-column tests exhibited a much larger scatter, producing a mean ratio of 1.5 with a large standard deviation of 0.36. The corresponding mean value and standard deviation values for the circular and rectangular beam-columns were approximately 1.6, 0.39 and 1.3, 0.21 respectively. The author concludes that the AISC Specifications are inconsistent and inadequate for CFT columns and beam-columns. The distributions of the test results were not normal and displayed a large amount of scatter, contributing to the difficulty in predicting CFT strength. The AISC approach for stub columns, which ignores the effect of confinement, was shown to be very conservative.

Hajjar, J. F., Gourley, B., 1996 (no tests)

Introduction. This paper presents an analytical study to derive a polynomial equation to calculate the 3D cross-section strength of square and rectangular CFTs. For this purpose, a fiber based analysis method from the literature was adopted to analyze zero length CFT beam-columns. After ensuring the accuracy of this method with the experimental results, a wide range of CFT cross-sections were analyzed. A polynomial equation was then proposed and verified using the experimental findings

Analytical Study. In the cross-section analysis program, each CFT cross-section was divided into individual steel and concrete fibers. The stress-strain response of each fiber was monitored throughout the analysis. A constant axial load level was selected and the curvature of CFT cross-section was increased incrementally. The location of the neutral axis was tracked until the equilibrium was satisfied. After reaching the equilibrium condition, the moment capacity was calculated. The whole process was repeated for different axial load levels. The stress-strain curves used in the analysis were taken from the available literature and the effects of concrete confinement and a biaxial stress condition in the steel were accounted for implicitly. This method of analysis was applied for short CFT column specimens tested in past experimental studies from the literature, and the experimental results were predicted accurately. In the case of tensile loading, the concrete was neglected and the strength of the cross-section was calculated as the steel tube yield strength.

The analysis method presented above was executed for a wide range of square cross-section types to provide data for verification of a polynomial cross section strength equation. The *D/t* ratio was varyied from 24 to 96. The yield strength of the steel was taken as 46 ksi, while the range of the concrete strength was varied from 3.5 to 15.1 ksi. A total of sixteen square cross-sections were analyzed and 100 load points were obtained to form their 3D strength surfaces. In addition, rectangular cross-sections with an aspect ratio of up to 1 to 2 were also analyzed. The yield strength of the steel tube for the rectangular cross sections was also ranged up to 70 ksi. A three-dimensional polynomial cross-section strength equation was then determined in terms of normalized axial load and normalized biaxial bending moments. As the axial load-moment interaction curve of the CFT cross-sections was approximately symmetric about the moment axes, the origin of the surfaces was shifted such that the shifted moment axes corresponded approximately to the peak moment achieved in the cross section in the presence of axial compression). The axial load was then redefined and normalized with respect to the new origin to make use of the approximate symmetry of the interaction diagrams. By performing a least squares analysis of the analysis results for the sixteen cross-section types, a high-order polynomial formula was proposed in terms of four coefficients that vary with D/t and f'_c/f_y. For the rectangular cross sections, the four coefficients are calculated for both major axis and minor axis D/t ratios and then averaged for use in the equation. Moreover, nominal axial strength and nominal moment strength values were also needed in the cross-section strength equation to normalize the axial load and moment values. The nominal axial load was calculated by adding the full strengths of concrete and steel components. A rectangular stress block was utilized to calculate the nominal moment strength, with the tensile strength of the concrete also being taken into account. These nominal strengths were compared with experimental results to insure accuracy. Finally, the cross-section strength formula was compared with both fiber-based cross section analysis results and experimental results for a wide range of square and rectangular CFTs cross sections. In the both cases, strong correlation was achieved.

Inai, E. and Sakino, K., 1996 (no tests)

Introduction. In this paper, a computational simulation study was described to predict the behavior of square CFTs subjected to constant axial load, bending, and shear. The computational formulation was applied to test specimens from the literature, and the findings were compared with the experimental results.

Analytical Study. Stress-strain relations for the steel and concrete were proposed to be used in the numerical simulation of the behavior of square CFTs. It was assumed that the behavior of the concrete was the same as plain concrete up to the peak load. Confinement was assumed to occur in post-peak response. It was accounted for in the stress-strain relation by introducing a factor that depended on the confinement pressure, steel strength, and concrete strength. For the stress-strain relation of steel, local buckling was the key issue to take into consideration. If the compressive strength of the hollow steel tube decreased due to local buckling, no reduction in yield strength was assumed for the steel tube with concrete infill. This was because the concrete infill improved resistance to local buckling. After the peak load, the stress-strain curve of steel descended linearly down to a strain level and then the stress remained constant. The corresponding strain level depended on the steel strength and D/t ratio. The proposed stress-strain relations for steel and concrete were first used to estimate the behavior of the axially loaded square CFT specimens. Good correlation was obtained between experimental and analytical behavior.

An analysis procedure for square CFT columns subjected to constant axial load, bending moment and shear force was also developed. The purpose of the analysis was to calculate the moment and axial strain at the critical section, which was located at the middle of the column for the specimens considered in this research. A linear curvature distribution was assumed and an incremental relation between drift angle and curvature was derived in terms of flexural stiffness, shear stiffness, length of the column and length of the plastic hinge. The moment at the critical section was calculated incrementally with step size increments in curvature and drift angle. The axial strain was also calculated using an assumed strain distribution along the length of the column. The concrete and steel stress-strain relationships were adjusted for hysteretic loading and the moment-drift angle and axial strain-drift angle relations were compared to specimens tested by other researchers. The axial shortening and the behavior after the ultimate load were predicted well. However, the ultimate moments were found to be a little smaller than the experimental values.

Toshiyuki, F. et al., 1996 (no tests)

Introduction. In this paper, equations were proposed to calculate the rotation capacity of circular and square CFT beam-columns. For this purpose, regression analysis was performed based upon test results from the Japanese literature.

Analytical Study. The test data included 165 rectangular and 47 circular beam-columns. For the rectangular sections, the ranges for the yield strength of the steel and the compressive strength of the concrete were 28 to 120 ksi and 2.61 to 14.8 ksi, respectively. The *D/t* ratio varied between 14 and 95 and the range of axial load ratio was 0 to 0.9. For circular sections, the steel strength ranged from 41 ksi to 119 ksi and the concrete strength ranged from 2.32 to 17.7 ksi. The *D/t* ratio varied between 18 and 67. The lower and upper limits of axial load ratios were 0.1 and 0.7, respectively.

The rotation capacity was determined based upon the limit rotation when the post-peak strength of the beam-column decreased to 95% of its peak value. The rotation capacity was found to decrease with an increase in the *D/t* ratio due to the reduction of confinement. A sharper decrease in strength was also observed after peak strength when the axial load ratio got larger. This was attributed to the occurrence of local buckling.

By fitting a line to the test data, the following equation was proposed for the limit rotation of rectangular beam-columns:

$$R(\%) = 4.24 - 1.68(P/P_o) - 0.105(P/P_o)D/t$$

An average value of 1.17 was obtained for the ratio between experimental and analytical rotation capacities.

The same technique was used to derive the equation for the rotation capacity of circular beam-columns and the following equation was proposed:

$$R(\%) = 8.0 - 7.0(P/P_o) - 0.03D/t$$

The average value of the ratio between the experimental and analytical results was found to be 0.99.

Tsuda, K. et al., 1996 (c, bm; m, r)

Introduction. An experimental study conducted on circular and square CFT beam-columns was presented in two companion papers (see also Matsui et al., 1995). The behavior of CFT specimens was examined under axial loading and combined axial and flexural loading. Columns having a wide range of *L/D* ratios were tested. The experimental results were compared with AIJ (1987, 1990) and CIDECT (1994) design code provisions.

Experimental Study Results and Discussions. There were two series of experiments. In the first series, forty-eight CFT and twelve HT columns were tested. The CFT specimens were subjected to monotonic concentrically and eccentrically applied axial loads. The main parameters were *L/D* ratio and magnitude of eccentricity. The *L/D* ratios ranged between 4 and 30. The maximum value of eccentricity for square and circular specimens were 4.9 in. and 4.1 in., respectively. The *D/t* ratio was selected as 33 for the square specimens and 37 for the circular specimens. The nominal yield strength of steel was 58.0 ksi and the nominal compressive strength of concrete was 4.3 ksi. Pinned-end conditions were used in the test setup.

For the first series of experiments, it was observed that the specimens having a higher magnitude of eccentricity exhibited lower axial strength and larger mid-height deflection. The effect of eccentricity decreased for high *L/D* ratios. The columns with *L/D* ratios less than 18 achieved the plastic moment capacity. The circular specimens in this range even exhibited larger capacities due to the confinement effect. For square specimens, the confinement effect was not observed. The capacities of the columns having *L/D* ratios above 18 could not attain the plastic capacity due instability effects.

In the second series of experiments, twenty CFT columns were tested under constant axial and cyclic horizontal loading. The main parameters were *L/D* ratio and the axial load ratio. The *L/D* ratio was varied between 6 and 24. The axial load ratio (P/P_0) ranged from 0.2 to 0.7. The *D/t* ratio and material properties were kept the same as in first series of experiments. The cantilever columns were fixed at the bottom and free at the top. Displacement-controlled loading was applied at the free end of the columns. For axial load ratios greater than 0.5, square columns showed rapid deterioration in strength. Circular columns showed stable and ductile hysteresis loops even for high slenderness and high axial load ratios.

Analytical Study. A modified AIJ method was developed to calculate the capacity of slender CFT columns. It was different from the AIJ (1987, 1990) method of calculating the capacity of the concrete portion of a CFT, and a new concrete column interaction equation was proposed. It was based on an elasto-plastic analysis that was conducted using a Newmark iteration method in which a sine curve is assumed as the deflected shape of the beam-column. The modified AIJ method agreed with experimental results of the first series of experiments when the *L/D* ratio was between 8 and 30. However, the test results were underestimated when the *L/D* was smaller than 4. This trend was most notable for circular columns and it was attributed to the effects of confinement and strain hardening, which in the proposed analytical method were not taken into account explicitly. On the other hand, the AIJ (1987, 1990) method was generally conservative. The CIDECT (1994) design provisions gave almost the same capacities as the modified AIJ method for *L/D* ratios between 12 and 30. This method had more accurate results for short circular columns, as concrete confinement was accounted for. In the second series of experiments, the columns with *L/D* ratios smaller than 9 achieved their cross-section strength. Both the AIJ (1987, 1990) and modified AIJ methods predicted most of the experimental results conservatively. It was pointed out that the design equations for CFT beam-columns having no restraint for joint translation should be reviewed.

Aho, M. and Leon, R. T., 1997 (no tests)

Introduction. In this report, the design procedures for composite columns and beam-columns in the AISC LRFD (1993) and EC4 (1993) Specifications were assessed and compared. The development of a database for composite column and beam-column tests was then presented. Using the tests results documented in the database, two new design methods for composite members were derived.

Analytical Study. In the AISC LRFD (1993) approach, modified material properties are provided for composite members, which are then treated and designed as all-steel sections. However, a stress-based design approach such as this was deemed to be inappropriate given the more complex behavior of composite members. The AISC LRFD (1993) procedure is also not satisfactory for coverage of issues such as confinement and slenderness effects. In addition, the interaction equations often underestimate the capacity of composite beam-columns.

The EC4 (1993) method utilizes individual material properties of the composite section in the design calculations. The confinement effect is taken into account for CFT members depending on the amount of eccentricity and slenderness. A plastic strength approach is applied in the beam-column design method. The interaction diagrams are also simplified with straight lines to avoid any iterative procedures.

The database contained material properties and capacities of specimens in column and beam-column tests from the literature. Steel-reinforced concrete specimens as well as circular and rectangular CFT specimens were included. However, cyclic tests and the tests with low *a/D* ratios were not covered in the database.

As a first step in developing new design methods for composite members, equations were proposed for the nominal axial strength and nominal moment strength based on calculation of plastic strengths. It was then intended to calculate the axial load capacity of columns by multiplying the nominal axial strength with a reduction factor. Two methods were utilized to calculate the reduction factor. In the first one, the slenderness equation of EC4 was taken as the reduction factor. Alternatively, the column curves in AISC LRFD (1993) were put into a form similar to the one in EC4. The predicted axial load capacities using the reduction factor equations were compared with the experimental values in the database. If good correlation was obtained, the applicability of the equations was checked against the beam-column test data. In the case of unsatisfactory correlation, either the equations were modified until they matched with both the column and beam-column test data or new equations were derived according to the beam-column data and checked for agreement with all of the tests.

Based on the analysis of the test data, seven different modified methods were proposed to replace the AISC LRFD (1993) design procedure. The predicted results were compared with the database and the method having the best correlation was determined. This method was then simplified and two new design methods, which employed the AISC LRFD (1993) column curve for the reduction factor of axial resistance, were derived. The first method was named the "Modified AISC Method" and it involved the replacement of the c_1, c_2, and c_3 factors in AISC LRFD (1993) method for axial strength with the values 1.0, 0.85, and 0.3, respectively. For circular CFTs, factors increasing the capacity still further were used. In the case of beam-columns, a plastic strength approach was utilized using the simplified interaction equation with straight lines. The second method was called the "Plastic Method" and the plastic strength calculations were used for both columns and beam-columns. The axial load capacity for the columns was reduced depending on the slenderness, and confinement was accounted for in circular CFTs. For beam-columns, a simplified interaction diagram was used and the axial load capacity was reduced depending on the amount of eccentricity.

El-Remaily, A. et al., 1997 (bc; r)

Introduction. In this paper, the seismic behavior of high strength concrete-filled tube columns was studied. Four simply supported circular specimens were tested under constant axial and cyclic lateral loads. At the end of the paper, a finite element model for CFT members was presented as part of ongoing research.

Experimental Study Results and Discussions. The main parameters of the tests were axial load level and *D/t* ratio. High strength concrete mixes with nominal compressive strengths of 10 ksi or 15 ksi were utilized for the specimens. The nominal yield strength of the steel tubes was 54 ksi and the *D/t* ratio was either 32 or 48.

The columns were tested in a horizontal position and displacement-controlled loading was applied in the lateral direction at the mid-height of the CFT. In addition, a rigid stub was located at mid-height to simulate the confining effect of floor system. The applied axial load ranged from $0.2P_o$ to $0.4P_o$, where P_o was the nominal axial strength of the specimen. At each ductility level from 1 to 10, the specimen was cycled two times. The ductility level for the specimens was defined as the ratio of the lateral displacement (Δ) to the ratio of the first yield displacement (Δ_y).

The specimens did not show a decrease in their moment capacities up to a ductility level of 10. They exhibited large hysteresis loops and nearly perfectly plastic behavior after the elastic range. Local buckling was observed near the midpoint of the specimens at a ductility level of 4 and this caused the axial deformations to increase. The range of axial shortening was 1.69 to 3.07 in.. The failure of the specimens was due to tensile cracking in the steel tube.

Analytical Study. The plastic moment capacity and AISC LRFD (1993) moment capacity were calculated for each specimen. The plastic moments closely estimated the test results with an error of 10% to 20% on the conservative side. The error was mainly due to lack of accuracy in predicting the effects of confinement. The AISC LRFD (1993) moments underestimated the test results by factors ranging from 2.57 to 4.54. For all of the specimens, the ratio of the measured stiffness to the transformed stiffness, which was calculated as the sum of the individual stiffnesses of the steel and concrete, was observed to decrease as the ductility level increased. The resulting stiffness degradations at the ductility level of 10 were between approximately 25% and 80%.

The finite element model, presented as ongoing research, consisted of two parallel beam-column elements and two spring elements lumped at the nodes. The two beam-column elements represented the steel and concrete portions of the member, respectively. The spring elements simulated the bond between the steel and concrete. This model is intended to be used for seismic analysis of composite structures.

Hajjar, J. F. et al., 1997a, b (no tests)

Introduction. In these two companion papers, the authors presented the development and verification of a 3D concentrated plasticity finite element formulation for CFT beam-column members. The beam-column finite element was adopted into a 3D fully nonlinear general purpose frame analysis program for unbraced frames having wide flange steel girders framing into CFT beam-columns. The finite element formulation was calibrated and verified using worldwide experimental results with a wide range of material and cross section properties.

Analytical Study. The macro model developed in this work employed a concentrated plasticity approach to account for material nonlinearity. The frame analysis program was based on the direct stiffness approach and an incremental updated Lagrangian formulation. Perfect bond between the steel and concrete was assumed. The stiffness matrix consisted of three components, including an elastic stiffness matrix, a geometric stiffness matrix, and a plastic reduction stiffness matrix. The CFT fiber element model neglects shear deformation due to bending (i.e., Euler-Bernoulli beam theory is used), nonlinearity due to torsion, and lateral-torsional and flexural-torsional buckling, as these modes of failure rarely occur, even in slender CFT beam-columns.

The elastic flexural and axial rigidity terms were taken as the superposition of the concrete core and steel tube strengths. The contribution of the concrete core to the elastic torsional rigidity was neglected. The geometric stiffness matrix was taken from the literature and was formulated assuming small strains, moderate displacements, and moderate rotations.

The cyclic stress-strain behavior of the CFT beam-column cross-section was simulated with a bounding surface model in three-dimensional stress-resultant space. The bounding surface formulation consisted of two limit surfaces at each element end. The inner loading surface represented the initiation of inelasticity, while the outer bounding surface represented the condition when the limiting stiffness of the CFT beam-column was reached. Both surfaces had the capability of expanding, contracting, and translating to capture typical characteristics of short CFT members under cyclic loading. For both the loading surface and bounding surface, the cross-section strength formula derived by Hajjar and Gourley (1996) was utilized. The CFT cross-section was assumed to be a work-hardening type material and thus both the loading and bounding surfaces were convex.

In this formula, the location of the centroids of the loading surface and bounding surface were traced with the use of a back-force vector that defined the location of the surface centroids relative to the stress-resultant-space origin. Once the load point reached the loading surface, it was assumed to remain on that surface, and the corresponding plastic deformations were assumed to be perpendicular to the loading surface, as an associated flow rule was used. The loading and bounding surfaces were assumed to harden isotropically and then kinematically. Isotropic hardening was based on the amount of accumulated plastic work. Both surfaces kinematically hardened in the same direction, although their rates of hardening were different. The rate of hardening became equal when the two surfaces contacted to each other. The kinematic hardening was assumed to be in the direction of the vector from the point on the bounding surface intersected by the incremental force vector, and its conjugate point on the loading surface.

Three cyclic tests and eight monotonic tests were chosen for the calibration of the bounding surface plasticity model. The tests included both proportional and non-proportional loading. The test setups used for calibration purposed included monotonically loaded beam-columns subjected to eccentric load, monotonically loaded beam-columns in single curvature bending with constant axial force, and cyclically loaded beam-columns in reverse curvature bending with constant axial force. In normalized force space, the behavior of CFT sections is sensitive to cross-section sizes and material strengths. Thus the parameters related to the material model were intended to be kept constant or varied with simple functions in terms of nominal strength of the concrete core normalized with respect to the nominal strength of the CFT cross section (P_{co}/P_o). This ratio accounted implicitly for the changes in both the *D/t* ratio and f'_c/f_y ratios.

For the verification of the analytical work, additional tests including monotonic and cyclic loading were selected. The analysis program was then executed for these tests while adhering to the calibrated parameters. It was found that the maximum error in the load-deflection response was less than 10% in most of the cases.

Kawano, A. and Matsui, C., 1997 (br; r)

Introduction. An experimental study was conducted to examine the axial behavior of circular CFTs. The specimens were also analyzed using finite element analysis. From the experimental and computational results, equations were derived to estimate the energy dissipation capacity of CFTs and the number of cycles up to fracture of the steel tube.

Experimental Study Results and Discussions. Both HTs and CFTs were tested. The specimens were subjected to cyclic axial loading until fracture. Pin-ended support conditions were provided. Values for the *D/t* ratio, *L/D* ratio, and loading pattern were the main parameters of the experimental program. The specimens were specified as short, medium, and long according to having *L/D* ratios of 5, 10, and 20, respectively. Values for the *D/t* ratios were 20, 30, and 50. The specimens were manufactured from steel tubes having diameters of 2.38 in. or 4 in. The ranges for the measured tensile yield strength of steel and measured compressive strength of concrete were 44.8 to 61.2 ksi and 4.37 to 6.69 ksi, respectively. Displacement-controlled loading was applied. For short columns, the loading pattern was controlled by measuring axial displacement at the mid-length. In the case of medium and long columns, the loading pattern was controlled by measuring the axial displacement between the two ends and the axial displacement at the mid-length was still recorded although it was not used as a control parameter. The HT specimens exhibited fewer cycles prior to fracture, and they showed a rapid decrease in strength after local buckling. The concrete infill improved the cyclic behavior, and the CFT specimens experienced stable hysteresis loops and a larger number cycles until fracture. An increase in the *D/t* ratio and an increase in the loading amplitude both reduced the number of cycles prior to fracture. A more dramatic decrease in strength was observed for the specimens with high *L/D* ratios. The diameter of the steel tubes did not affect the response of the columns significantly. The number of cycles until fracture was consistently larger than the number of cycles until local buckling. Consequently, it was concluded that the capacity of the CFTs should be based on the energy absorption capacity until fracture rather than the energy absorption capacity until local buckling. The energy absorption capacities of the specimens were affected by the *D/t* ratio and the amplitude of the axial displacement. It was found to decrease with an increase in *D/t* ratio and an increase in axial displacement amplitude.

Analytical Study. The tested columns were analyzed using the finite element method. A fiber-based cantilever beam-column model was employed in the analysis and axial loads were applied with an eccentricity of 5% of the tube diameter to account for imperfections. The stress-strain relationships for steel and concrete were adopted from the available literature. Local buckling and slip along the concrete-steel interfacebetween steel and concrete were not taken into account. Good correlation was achieved between the experimental and computational peak axial loads. However, the hyteresis loops were not estimated with high accuracy. This was attributed to local buckling, which was not considered in the analysis. An equation was proposed using the experimental results in order to estimate the number cycles until fracture. It was dependent on the *D/t* ratio and amplitude of the longitudinal strain at the mid-length of the specimens. The specimens having medium *L/D* ratio were shown to have the smallest number of cycles until fracture because both compression and bending were effective to increase the amplitude of longitudinal strain. In the case of long or short columns, either bending or compression governed the behavior alone. Consequently, their amplitude of longitudinal strain was calculated to be smaller and they had a higher number of cycles until fracture. Multiplying the number of cycles until fracture with the energy absorption at each cycle, both of which were obtained analytically, the energy absorption until fracture was calculated. Good correlation was achieved with the experimental results and the trend for the effect of the *L/D* ratio on the CFT energy absorption was similar to the one observed while calculating the number of cycles until fracture.

Kilpatrick, A. and Rangan B. V., 1997 (bc; m)

Introduction. Tests were conducted on circular high strength CFTs under eccentrically applied axial load. In addition, the authors described a deformation control method of analysis to estimate the strength and load-deformation response of the test specimens. The analytical and experimental results were compared.

Experimental Study Results and Discussions. The main parameter of the experimental study was the amount of eccentricity in the applied axial load. The measured yield strength of the steel tubes was 59.47 ksi. The specimens filled with high strength concrete having a measured compressive strength of 13.92 ksi. The *L/D* and *D/t* ratios were 21.43 and 42.29, respectively.

The columns were tested both in double curvature and single curvature. The eccentricity at the bottom and top of the specimens were different in most of the cases. The ends of the columns were clamped to hardened knife-edge assemblages. The required eccentricity was provided by moving the column ends laterally from the loading axis.

Analytical Study. The experimental results were compared to analytical results using a classic Newmark iteration scheme based on use of moment-curvature-thrust relations. In general, the calculated strength and force-deflection response of the specimens matched with the experimental results quite accurately. However, the response of the specimens under single curvature showed better correlation than the specimens tested under double curvature. Analytically generated strength envelopes for equal eccentricity at the top and bottom showed a greater rate of increase and a higher concentric axial load capacity in the case of double curvature.

The authors also investigated the effect of variation in eccentricity on the column strength. For this purpose, columns in double curvature having equal magnitude of eccentricities at the bottom and at the top were analyzed with ±0.0394 in. error. In the first case, the eccentricity at the top and bottom was increased by 0.0394 in.. In the second case, the eccentricity at the top decreased by 0.0394 in. while the eccentricity at the bottom was increased by 0.0394 in.. It was found that the strength decreased most severely in the latter condition, with the amount of reduction being up to 25% . This was attributed to the change in the deformed shape as it was not perfectly asymmetrical any more.

O'Shea, M. D. and Bridge, R. Q., 1997 (c, bc; m)

Introduction. In a series of papers and reports (O'Shea and Bridge, 1994, 1997a, 1997b, 1997c, 1997d, 1997e), the authors presented a series of experiments on the behavior of circular and square thin-walled concrete-filled steel tubes. In addition, the authors compared the experimental results to the design methods presented in the ACI 318 (1989), AISC LRFD (1993), AISI LRFD (1991), AS4100 (1990), EC4 (1992), and AS 3600 (1988) specifications for concrete-filled steel tube members.

Experimental Study, Results and Discussions. The experimental study consisted of four series of tests. For the first two series, the authors investigated local buckling of the steel tube wall. The main purpose of the third and fourth series was to investigate the confinement of the concrete. Eight different loading types were applied to the specimens. They were labeled as BS, BSU, BSC, CS, CL, CF, E1 or E2. Tests labeled BS included axial loading of the bare steel tube alone. Experiments BSU and BSC had axial loading of the steel tube alone while the steel tube was filled with unbonded concrete. Experiments labeled CS and CF included tests in which the CFTs were loaded axially through both the steel and concrete together. Experiments labeled CL included axial loading of the unbonded concrete only. Tests E1 and E2 had eccentric axial loading of both the concrete and steel simultaneously. For type E1 loading, the eccentricity was $D/10$, while for type E2 loading, the eccentricity was $D/20$, where D was the outer nominal diameter of the circular steel tube or the nominal depth of the rectangular steel tube. The eccentricity was provided through thick endplates connected to offset hemispherical bearings.

In the first series of the experimental study, the local buckling behavior of short thin-walled circular tube members was investigated. Both unfilled and concrete-filled specimens were tested. Type BS, BSC, E1 and E2 loadings were utilized. Type E1 and E2 loadings were applied to the bare steel tubes. Specimens with nominal D/t ratios ranging from 55 to 200 were prepared. The yield stresses for the steel tubes ranged from 26.83 ksi to 52.69 ksi according to the coupon tests. The out-of-plane imperfections of the steel tube walls were determined using a fixed measuring ring and were found to increase near the weld. The membrane residual stresses were also high near the weld. The bending residual stresses for the specimens were generally small. In these tests, the concrete was serving as a lateral restraint with an average strength of 6.89 ksi. For each axially loaded specimen, the end conditions were fixed and displacement controlled loading was applied. The BS and BSC specimens showed similar responses and it was found that the ductility improved as the D/t ratio decreased. The failure of each of the axially loaded specimen was due to outward local buckling at one end (with the exception of specimen S16BSC, which had local buckling at its mid-height). This showed that internal concrete as a lateral restraint did not improved local buckling strength for circular CFTs. Pin-pin support conditions were provided for the eccentrically loaded specimens and displacement controlled loading was applied. The peak capacities of the eccentrically loaded specimens were less than those of the axially loaded specimens and the strength of the tubes decreased with an increase in eccentricity. The failure for all of the eccentrically loaded specimens, except two of them, was due to outward local buckling at one end. For specimens S30BSE2 and S30BSE1, local buckling formed at the mid-height. The capacity of S16BSE2 specimen was higher than expected and that of S30BSE1 specimen was less than expected. This was attributed to the differences in the eccentricities. The authors presented the AISC LRFD (1993), AISI LRFD (1991), AS4100 (1990), and Grimault and Janss (1977) procedures to calculate the local buckling capacity of axially loaded specimens. These methods were applied to the axially loaded specimens and the calculated capacities were compared with the experimental results. The moment capacities according to the aforementioned standards were also computed and compared with the experimental results.

In series 2, two groups of thin-walled square box specimens were tested. For the first group, an *L/D* ratio of 3.45 was used and six different *D/t* ratios ranging from 37.4 to 130.7 were selected. In the second group, a *D/t* ratio of 130.7 was used and six different *L/D* ratios ranging from 0.77 to 3.45 were selected. The specimens in both groups consisted of bare steel tubes and steel tubes filled with either unbonded or bonded concrete. The first group of specimens was tested by loading types BS, BSU and CS. The second group was tested by loading types BS and BSU. The average yield stress for the steel was obtained as 40.90 ksi from the coupon tests. The concrete used for the specimens had a nominal capacity of 2.90 ksi. The out-of-plane geometric imperfections of the steel tube walls were investigated for each specimen using an automatic level with a micrometer. The residual stresses on representative tubes for each *D/t* ratio were also examined. Displacement controlled loading was applied and the ends of the specimens were fixed. In addition to conducting the experiments, the authors introduced a finite strip method to calculate the elastic buckling stress of a plate and then presented modified von Karman and modified Winter formulations for determining the plate strength. The test results were also compared with the predictions according to the AISC LRFD (1993), AISI LRFD (1991), AS4100 (1990) procedures. The collective results showed that the local buckling strengths of the bare steel tube specimens were found to increase by up to 50% when they were filled with unbonded concrete. This was attributed to the outward buckling mode. The results of the bare steel tube specimens and the unbonded concrete filled specimens with a *D/t* ratio of 130.7 and an *L/D* ratio greater than 1.15 indicated that the strength remained almost constant for increasing *L/D* ratios. The authors presented some modifications to the existing Winter formula and the new formula matched with the experimental results better.

The third series of experiments were performed to investigate the effect of confinement on the cross section strength. Short thin-walled steel tubes filled with medium and high strength concrete were tested by type CL, CS, E1 and E2 loadings. Five different *D/t* ratios of the specimens were selected ranging from 63 to 190. According to the coupon test results, the yield strengths of the steel tubes were found to range from 26.93 to 52.69 ksi. The membrane residual stresses in the tubes were high near the welds. The bending residual stresses were generally small except for the S30 type specimens. The steel tubes were filled with two concrete mixes having nominal strengths of 7.25 ksi and 11.60 ksi. The geometric imperfections of the steel tube walls were obtained for the tubes tested by type CS, E1 and E2 loadings. It was again found that the imperfections were highest near the seam welds. The type CS and CL tests of the specimens filled with 7.25 ksi concrete were carried out by displacement controlled loading. For all the other tests, force controlled loading was utilized. In the type CS test, two different failure patterns were observed. In the first one, the specimens maintained the bond between steel and concrete until failure. For the other specimens, the bond between steel and concrete was not maintained and local buckling took place. The vertical principal strains on the steel tube were observed to decrease when local buckling occurred. The principal strains increased rapidly and diverged from each other after concrete crushing. In the type CS tests, only one specimen experienced local buckling. This confirmed that the bond between steel and concrete prevented local buckling. For the type CS and CL loadings of the specimens with 7.25 ksi concrete, the ductility and strength of the thicker specimens were higher, as the thicker specimens provided better confinement. When concentric axial load was applied to the 11.60 ksi specimens, the ductility, strength and confinement were also found to improve with an increase in tube thickness. The principle strains for the thickest 11.60 ksi specimens increased in a linear fashion up to 0.006. This showed that the lateral deformation of the high strength concrete was less compared to moderate strength concrete. In the eccentric axial load tests, the 7.25 ksi specimens showed higher strength and ductility with increasing thickness. The divergence in the vertical strains was observed in the post peak region. This was attributed either to local buckling or concrete crushing. Increasing the eccentricity caused the ultimate strength to decrease but it improved the ductility. The eccentrically loaded 11.60 ksi specimens showed the same trend as the eccentrically loaded 7.25 ksi specimens in terms of strength and ductility. For 11.60 ksi specimens, the principle strains started to increase rapidly after the peak load and this was again the effect of concrete crushing. The authors calculated the confining pressures of the specimens in the CL tests and found that the confining pressure decreased for the specimens having high strength concrete and greater *D/t* ratio. Using the maximum energy distortion theory, they concluded that the steel in all the CS specimens yielded. The moment capacities of the eccentrically loaded specimens were calculated with a fiber analysis method by Wheeler and Bridge (1993). This method gave conservative results for specimens loaded with an eccentricity of D/20 if a confined concrete model was used. However, an unconfined concrete model gave conservative results for the specimens with D/10 eccentricity.

The fourth experimental study was again performed to investigate the effect of confinement on the cross section strength. Short thin-walled very high strength concrete- filled steel tubes were tested by type CF, CL, CS, E1 and E2 loadings. Force controlled loading was utilized in all the cases. The specimen sizes and steel properties were the same as in the third experimental study. The steel tubes were filled with three different concrete mixes having nominal strengths of 11.60, 14.50 and 17.41 ksi. The lateral imperfections of the steel tube walls were obtained for the specimens tested by CS, E1 and E2 type loadings and similar trends were observed with the previous experimental study. The concentrically loaded specimens showed brittle post peak behavior. They failed suddenly and their post peak responses could not be obtained completely. In type CS tests, two different failure patterns were observed. In the first one, the specimens maintained the bond between steel and concrete until failure. For the other specimens, the bond between steel and concrete was not maintained and local buckling took place. The principal strain values of the steel tubes were found to diverge from each other after local concrete crushing. The formation of local buckling was observed with a reduction in the vertical strains. From the available data for CS and CL tests, it was found that increasing the wall thickness also increased the ductility. According to the results of the CL tests, strength enhancement of the concrete due to confinement was also greater for the thick walled tubes. In addition, this enhancement was greater for the concrete batches having smaller strength. For the specimens tested by CS type loading, strength enhancement of concrete did not occur. In the E1 and E2 type tests, the eccentricity was provided using very thick and stiff endplates with an offset half-round. According to the results, the increase in eccentricity caused a decrease in capacity. Some specimens behaved in a more ductile manner when loaded at greater eccentricity. The capacity and ductility of thicker specimens were higher. Local concrete crushing was observed with deviations in vertical strains measured at equal rotational distances from the axis of bending. The authors calculated the capacity of the eccentrically loaded specimens using a fiber analysis method by Wheeler and Bridge (1993). The results were compared with experiments and they concluded that the increase in concrete strength due to confinement was small and did not occur except in the S30 type specimens.

Analytical Study. In the last report (O'Shea and Bridge, 1997e), the authors developed design methodologies for thin-walled concrete filled steel tubes. The capacity of a short, thick-walled circular steel tube filled with medium strength concrete was determined using ACI 318 (1989), EC4 (1992) and modified AS 3600 (1990) procedures. It was also calculated with the fiber method by Wheeler and Bridge (1993). The outputs were compared and EC4 (1992) gave the closest capacity to the one obtained by the fiber method. From the past experiments of several researchers, the specimens, which satisfied the provisions of EC4 (1992) were selected for comparison purposes. The capacity of each specimen was computed according to the three aforementioned codes. When the results were compared, the capacities of EC4 (1992) showed lower scatter than the others. This was attributed to the fact that the confinement effect was taken into account in EC4 (1992). The results for the other two procedures were very conservative because the confinement was neglected. The column curves were utilized for slenderness effect in EC4 (1992). However, minimum eccentricity approach was used both in ACI 318 (1989) and modified AS 3600. It was concluded that for slenderness, EC4 (1992) approach was more rational compared to the others. When local buckling was involved, the authors proposed a modified Winter Formula to calculate the local buckling strength of square steel tubes using the results of the experimental study (O'Shea and Bridge, 1997b). From tests of thin-walled circular steel tubes (O'Shea and Bridge, 1997a), the authors decided that the current methods in the codes could be used to calculate local buckling strength. To investigate confinement of concrete, the concrete models by Martinez, Nilson, and Slate (1984), Mandler, Priestly, and Park (1988) and Attard and Settunge (1996) were presented. The models by Mander, Priestly, Park (1988) and Attard andSettunge (1996) were calibrated according to the results of the type CL experiments conducted by the authors (O'Shea and Bridge, 1997c, 1997d). These modified models showed good correlation with experimental results and the authors concluded that they could be used to estimate the response of confined concrete in circular steel tubes. In the same test series, the experimental results were used to propose design methods for type CS loading of circular concrete-filled steel tubes. The EC4 (1992) provisions predicted accurately the response of the type CS specimens, which had 11.60 ksi concrete and experienced no local buckling. For concrete strengths greater than 11.60 ksi, EC4 (1992) predictions were close to the experimental values when confinement, steel reduction and local buckling were neglected. The formation of local buckling might result higher cross section strengths for concrete strengths up to 11.60 ksi. However, there were not enough tests for this effect. The authors concluded that design without local buckling was a conservative approach for type CS loadings. The authors also examined the experimental results from type E1 and E2 loadings (O'Shea and Bridge, 1997c, 1997d) to propose design procedures. The capacities of the eccentrically loaded specimens were calculated by the EC4 (1992) provisions and no local buckling was assumed. To evaluate the EC4 (1992) procedures, the fiber method by Wheeler and Bridge (1993) was also applied to the specimens. Using the fiber model, two analyses were performed. For the first analysis, an unconfined concrete model was used and the second one was performed by a confined concrete model. The results were compared with the experimental values. For thin-walled circular steel tubes filled with concrete up to a strength of 7.25 ksi, EC4 (1992) gave conservative results and it was recommended to be used. However, for concrete strength greater than 7.25 ksi, the authors decided that a rigorous fiber analysis was required.

Shakir-Khalil, H. and Al-Rawdan, A., 1997 (c, bc; m)

Introduction. This paper concluded a series of four papers by the first author (see also Shakir-Khalil and Zeghiche, 1989; Shakir-Khalil and Mouli, 1990; Shakir-Khalil, 1991; Shakir-Khalil, 1994) documenting tests of full-scale rectangular CFT columns. This segment contained the test and analysis results of 11 members subjected to uniaxial bending about the major and minor axes. The experimental failure loads were compared to the predictions of the British Standard, BS5400 (1979), as well as, to the predictions of three-dimensional finite element studies.

Experimental Study, Results, and Discussions. Stub columns of rectangular CFTs were first tested to determine the squash load of CFT members. Column CFT specimens were then tested monotically in a horizontal position. Pin-ended support conditions were simulated by the test setup. The *L/D* ratios were ranging from 21 to 49. For major and minor axis bending, the *D/t* ratio of the specimens was 30 and 20, respectively. The applied eccentricities did not exceed one half the diameter of the column. The yield strength of steel was varying from 47.0 ksi to 53.3 ksi. The compressive strength of concrete ranged between 5.3 ksi and 6.0 ksi. The stub columns exhibited 16 to 30% higher strength than their nominal axial load capacity calculated according to BS5400 (1979). Their strength was observed to decrease with an increase in length due to local buckling. The local buckling generally took place at the longer side of the tubes. The concrete was investigated after testing. It was crushed but kept its integrity, thus facilitating the achievement of the large strengths in the stub columns. In addition, the CFT columns were found to have an increase in strength of 25-37% over similar hollow tubes. Except for one case, the failure load decreased with an increase in end eccentricity. This was because that specimen experienced pure bending response about the major axis. The authors noted that the behavior of columns subjected to small eccentricities about the major axis was especially sensitive to any imperfections, most notably, out-of-straightness.

The results showed that the BS5400 (1979) design code conservatively estimated failure due to major axis bending by 20-66% (although the author indicated that this may not be the case for longer columns). The BS5400 (1979) specification, however, overestimated the strength in minor axis bending, and the author deemed it imperative that a change in the code be implemented.

Analytical Study. Using a standard stress block approach, the interaction diagram for the stub column sections was generated. The same graph was also derived in the finite element program ABAQUS. The CFT specimen was modeled by using brick elements for the concrete and shell elements for the steel. The brick elements close to the supports were rigid to simulate the effect of loading plates. Perfect bond between the steel and concrete were assumed. The interaction diagrams obtained from the two methods correlated well. In the three-dimensional finite element analysis, the tensile strength of concrete was accounted for, which resulted in somewhat higher strength values than the stress block approach. However, the difference for the maximum moment values from the two methods was found to be approximately 5%.

The column specimens were modeled in ABAQUS in the same way as the stub column specimens. The columns under major axis bending were analyzed with zero and 0.118 in. minor axis eccentricity, separately, where 0.118 in. was the minimum eccentricity about the minor axis required by BS5400 (1979) for the tested column sizes. The finite element results showed that the effect of the 0.118 in. eccentricity was important in the case of long columns subjected to a small amount eccentricity about their major axis. The load capacities for these specimens were found to be lower and their mode of failure was governed by out-of-plane displacements when minor axis eccentricity was introduced. The computational results obtained with minor axis eccentricity were conservative and showed better agreement with the experimental results than the BS5400 (1979) predictions. Another observation from the finite element analysis was that the in-plane mid-height deflections were found to get larger with an increase in column length and major axis eccentricity.

When the columns that were tested under minor axis bending were analyzed, the load-displacement relationships matched well with the computational results from ABAQUS. Although. the computational results were found to overestimate the experimental failure loads by 6 to 12%, they were still better than the BS5400 (1979) predictions.

Zhang, W. and Shahrooz, B. M., 1997 (bc; r)

Introduction. In this report, analytical models for cross-section strength of square CFTs and the corresponding concrete and steel stress-strain relationships were presented. The analytical models were implemented for the specimens of past and current experimental studies and the results were compared with the measured values. Member behavior was also investigated. It was captured by numerical techniques using reliable cross-section responses and compared with the experimental findings. In addition, an experimental study for two square CFT beam-columns was also presented.

Experimental Study. Two square CFTs were subjected to combined axial force and flexure. The axial load was the main parameter of the experiment. One of the specimens was subjected to an axial load of $0.18P_o$. The second one was not subjected to any axial load. The *D/t* ratio was 32 and the *L/D* ratio was 4.8 for both of the specimens. The average measured steel yield strength and compressive concrete core strength were 53.67 ksi and 6.05 ksi, respectively. The columns were tested in a horizontal position and the supports were pin-ended. After maintaining the constant axial load, transverse loads were applied at two points along the specimen length. The specimens were loaded and unloaded to a zero transverse load to examine the stiffness degradation throughout the loading history. The strain distribution along the mid-span, vertical deflections, and support rotations were measured.

Both specimens failed by inelastic flexural buckling. The column tested without axial load exhibited higher strength and more stable post-peak behavior. In both tests, the strain distribution along the mid-span showed that most of the steel was yielded close to the top and bottom fibers.

Analytical Study. The cross section types used in the experimental studies by Furlong (1967, 1968), Tomii and Sakino (1979b) and Fujimoto et al. (1995) were analyzed using six different analytical models. In the first two models, the ACI (1995) standard stress block approach developed for reinforced concrete members was implemented. A fiber approach was chosen for the remaining ones and concrete and steel stress-strain curves varied for each analytical model. The effects of cold working and residual stresses on cross- section strength were examined separately.

The cross-section strengths calculated by the standard ACI method matched closely with the measured values of Furlong (1967, 1968) and Tomii and Sakino (1979b). However, they did not estimate the Fujimoto et al. (1995) results well due to the high strength steel utilized in those specimens. The fiber models improved the analytical results for the tests of Furlong (1967, 1968) and Tomii and Sakino (1979b) only marginally. However, they estimated the results by Fujimoto et al. (1995) better than the standard ACI (1995) method. Moment-curvature-thrust analysis was also made with the fiber models for the tests of Tomii and Sakino (1979b) and good correlation was obtained with the experimentally obtained curves, even in the post-peak region. Among the fiber models, the concrete and steel stress-strain relationships varied to study the effects like concrete confinement and local buckling of steel tube. To account for confinement effect, the concrete model of Tomii and Sakino (1979b) and the concrete model of Inai and Sakino (1996) were implemented in the third and fourth analytical models, respectively. In the former concrete model, the confinement effect depended on D/t ratio alone while in the latter one, confinement was effected by both D/t ratio and yield strength of steel. The analysis results showed better correlation with the experiments for the concrete model of Tomii and Sakino (1979b) and it was recommended to be used. In the fifth and sixth analysis types, different compressive steel properties were used to account for local buckling. However, the results showed that the effect of local buckling was not so significant due to the low D/t ratio of the cross sections studied.

The increase in strength at the corner regions because of cold working was modeled in several of the analyses by selecting different material properties in these portions of the cross section. Two sets of moment curvature-thrust analyses were performed for a sample cross section. In the first analysis, cold working was neglected, while in the second analysis, cold working was accounted for. It was determined that the moment strength was affected by approximately 5% at an axial load of $0.60P_0$, and the difference was even more negligible when high strength materials were utilized. For residual stresses, two types of residual stress distribution was assumed on a sample cross section and moment-curvature-thrust analysis were performed. It was found that the residual stresses caused reduction in cross-sectional stiffness. The effect of residual stresses was greater when the axial load was large. The authors proposed a conservative residual stress distribution and recommended it to be used if the actual distribution was not available.

Using the strain distributions at the peak points along the backbone of the load-deflection curves of the two tested CFT beam-columns, moment-curvature-thrust diagrams were generated at the mid-span of the CFTs. The flexural stiffness of the specimens was computed from the linear portion of these diagrams. A fiber model employing the concrete model of Tomii and Sakino (1979b) and measured stress-strain relations for the steel was then implemented to obtain the analytical moment-curvature-thrust relations of the specimens. Good correlation was obtained between the experimental and analytical moment-curvature-thrust diagrams. The initial flexural stiffness calculated from the fiber analysis matched with the measured stiffness of the column tested under axial load. However, the measured initial stiffness was underestimated for the column having no axial load. For the column tested under axial load, the loss of stiffness observed after a moment of 1505 kip-inches was not noticeable in the analytical results. This was attributed to the softening of the steel as a result of the subsequent loading and unloading process applied to the specimens. The moment-curvature-thrust relationships from the fiber analysis were then integrated numerically and load-displacement curves were obtained. For the specimens having axial load, second order effects were accounted for. The analytical load-displacement curves showed good correlation with the experimental results. However, the analytical results indicated a loss in initial stiffness at a higher load as compared to the experimental stiffness of the specimens. This was again attributed to the softening of the steel.

Hajjar, J. F. et al., 1998a, b (no tests)

Introduction. In these two papers, the authors presented the development and verification of a 3D distributed plasticity finite element formulation for CFT beam-column members. The beam-column finite element was implemented into a 3D fully nonlinear general purpose frame analysis program for unbraced frames having wide flange steel girders framing into CFT beam-columns. The finite element formulation was calibrated and verified using worldwide experimental results with a wide range of material and cross section properties.

Analytical Study. The fiber model developed in this work discretized each end cross section of the element into a grid of steel and concrete fibers. Numerical integration through the cross section was used to compute cross section rigidities and stress-resultants, and integration along the length allowed for modeling of distributed plasticity throughout the element. Geometric nonlinearity was also included through the use of a co-rotational formulation assuming small strains, moderate displacements, and moderate rotations. The CFT fiber element model neglects shear deformation due to bending (i.e., Euler-Bernoulli beam theory is used), nonlinearity due to torsion, and lateral-torsional and flexural-torsional buckling, as these modes of failure rarely occur, even in slender CFT beam-columns.

The CFT formulation adds three translational DOFs at each element end, resulting in an 18 DOF finite element, to permit the concrete core to translate axially relative to the steel tube. The element is thus able to track slip in an arbitrarily-oriented CFT. A set of nonlinear springs permits transfer of force between the steel and concrete, and the formulation is able to capture behavior ranging from perfect bond to immediate slip. Calibration and verification of the bond strength and nonlinear slip stiffness (yielding a cyclic bilinear load-slip relation with little stiffness after loss of bond) were performed based on results of a number of tests of steel members framing into CFTs with simple shear tabs, and of CFTs in flexure in which the concrete core was allowed to slip through the ends of the member (all tests had no shear studs inside the tube). With this model, the concentration of plasticity and/or nonlinear slip in a CFT connection in a braced or unbraced frame may be tracked accurately.

The steel and concrete constitutive formulations of the fiber model were each based on stress-space bounding surface plasticity formulations. The steel formulation models the rounded shape of the stress-strain curve found in cold-formed tube steel, cyclic hardening, and ratcheting behavior. The concrete formulation captures strength and stiffness degradation by means of a cumulative damage parameter. The concrete formulation also models fibers which cycle into tension and then back into compression by simulating crack opening and closure. The uniaxial stress-strain behavior of the constitutive models were calibrated to simulate the cyclic behavior of ASTM A500 cold-formed tube steel and normal strength concrete. In addition, for the steel formulation, the different stress-strain behavior exhibited in the corners and flanges of cold-worked steel tubes was modeled. For the concrete formulation, the post-failure plastic modulus of the stress-strain response was calibrated to be substantially increased from the softening modulus established for unconfined concrete. The resulting more gradual loss of strength, calibrated to match the results from CFT flexural experiments conducted by Tomii and Sakino (1979a) that yielded moment-curvature-thrust data, represents the added ductility seen in rectangular CFTs as a result of the moderate confinement exhibited in these members. The beam-column fiber model was verified against over 30 experimental results of CFT beam-columns and subassemblages subjected to monotonic and cyclic loading. Correlation was found to be strong prior to local buckling occurring in the CFTs, as the formulation does not account explicitly for local buckling.

Kawaguchi, J. et al., 1998 (no tests)

Introduction. In this paper, a database study on beam-column tests of square and circular CFTs was presented. All the test data was gathered from the Japanese literature published between 1971 and 1997. Using the database, load capacities and chord rotation capacities of CFT beam-columns were examined. In addition, a multi-linear moment-rotation model for CFT beam-columns was proposed.

Analytical Study. The database consisted of 143 square and 66 circular beam-column tests. Length (L), depth (D) and thickness of the steel tube (t) were the main parameters to be recorded for the size of the specimens. Loading methods (cyclic or monotonic) and loading systems were specified in the database. The axial load over nominal axial strength ratios (P/P_o) were also reported. Three types of loading systems were defined depending on the support and loading conditions. The first type (CL) was the cantilever beam-column system with a horizontal and vertical load applied at the free end. The second system (SS) was a beam-column having a fixed support at the bottom and a guided support at the top, putting the member into reverse curvature flexure . For this system, the horizontal and axial loads were again applied at the top end. The third system (TL) was a pin-ended beam-column with axial loads applied at the ends and a horizontal load applied at the mid-height. The yield strength (f_y) and ultimate strength (f_{su}) of the steel, the modulus of elasticity (E_s) of the steel, and the cylinder strength of the concrete (f'_c) were kept in the database. From the experimental results, the maximum bending moment (M_u) values were recorded. The axial strength of the steel tube (P_{so}), nominal axial strength of the CFT section (P_o), and ultimate bending moment (M_{pc}) were calculated according to the AIJ (1997) design code provisions using the measured material properties and placed in the database. The experimental rotation angles and secant stiffnesses at different levels of moment strength were also included in the database. The theoretical elastic stiffness (K_e) values were entered as well.

In the experiments, the *L/D* ratios were generally less than 7. A similar number of tests of the square CFTs were done for each of the three loading systems. However, CL type test setup was rarely utilized for circular CFTs. The values of the *D/t* ratios were usually less than 48 and 80 for square sections and circular sections, respectively. The range of concrete strength used in the experiments was large. However, in most of the cases, the concrete strength selected for circular CFTs was higher than that used in square CFTs. Steel strengths above 58.0 ksi were common in circular tubes. On the other hand, the steel strengths of the square tubes exhibited two trends. They were usually either less than 30.5 ksi or greater than 65.3 ksi.

The M_{pc} values for the specimens were calculated based on a full plastic stress state. It was found out that the M_u/M_{pc} ratios were usually greater than unity. The rotation capacities of the specimens were determined by using empirical equations from the literature. For square specimens, the theoretical results estimated the experimental rotation capacities fairly well. In case of circular sections, theoretical results provided a lower limit for the experimental rotation capacities.

A multi-linear moment-rotation relationship to be used in push-over analysis of CFT frames was proposed. It was composed of four linear regions. The first region had a slope equal to the theoretical elastic stiffness and it was continuing up to 1/3 of M_{pc}. The second and third parts of the curves had lower slope values, with upper limits of 85% of M_{pc} and M_{pc}, respectively. The fourth region was a horizontal line that continued until the ultimate rotation (R_u) was achieved. The rotation at 85% percent of M_{pc} (R_{85}), the rotation at ultimate moment (R_{max}), and R_u were all required to construct the proposed moment-rotation curve. The following formulations were used for this purpose (all stress values are in MPa):

Square: $$R_{85} = 2.00 - 1.53(\frac{P}{P_o}) + (-0.03 - 0.03(\frac{P}{P_o}))\frac{D}{t}$$

Circular: $$R_{85} = 0.69 + 1.61(\frac{P}{P_o}) + (0.02 - 0.06(\frac{P}{P_o}))\frac{D}{t}$$

Square: $$R_{\max} = 5.61 - 7.30(\frac{P}{P_o}) + (-0.10 + 0.16(\frac{P}{P_o}))\frac{D}{t}$$

Circular: $$R_{\max} = -0.31 + 9.94(\frac{P}{P_o}) + (0.12 - 0.31(\frac{P}{P_o}))\frac{D}{t}$$

Square: $$R_u = 5.5 - \frac{f'_c - 39}{120} - 0.045\frac{D}{t}\sqrt{\frac{f_y}{324}} - 5.0\frac{P}{P_o}$$

Circular: $$R_u = 7.5 - \frac{f'_c - 39}{55} - 0.05\frac{D}{t}\sqrt{\frac{f_y}{324}} - 5.0\frac{P}{P_o}$$

<u>Nakahara, H. and Sakino, K., 1998</u> (c, bc; m)

<u>Introduction</u>. The authors investigated the behavior of square steel tubes filled with high strength concrete. An experimental study including monotonic tests of eight stub-columns and eight beam-columns was described.

<u>Experimental Study, Results, and Discussions</u>. The test specimens were filled with concrete having a measured compressive strength of 17.26 ksi Steel tubes with measured yield strengths of either 44.96 ksi or 113.28 ksi were utilized. Two channel sections were welded to each other toe-to-toe to manufacture the steel tubes. All the specimens were square and had an *L/D* ratio of 3. The *D/t* ratio was either 30 or 60.

Four hollow steel tubes and four CFTs were subjected to axial compressive loading. The hollow columns showed a more dramatic drop in strength after achieving their peak strength as the *D/t* ratio got larger. None of the CFT columns reached their squash load capacity, due to the onset of local buckling .

Uniform bending and constant axial load was applied to the beam-column specimens. The main parameters of the beam-column tests were the *D/t* ratio and axial load ratio. Bending was provided through thick and stiff end plates welded to the specimen. The beams were thus simply supported, with rotations applied to their ends . The welded joints between the steel tube and end plates were reinforced with 1.31 in. thick trapezoidal plates. These plates were attached to the tension face of the tubes at the ends of the tubes. When the tests were conducted, it was observed that the specimens again saw a shaper drop in strength after achieving the peak strength with an increase in *D/t* ratio or axial load ratio. For most of the specimens, the experimental moment capacity was less than the plastic moment capacity.

During both series of the tests, the authors observed early concrete crushing and local buckling. Also, the scale effect of the cross section and difference of steel and concrete strains at maximum strength were among the factors affecting the test results.

<u>Nakahara, H. et al., 1998</u> (no tests)

<u>Introduction</u>. In this paper, an analytical study was presented to estimate the response of short CFT square columns under concentric axial loading. Individual stress-strain relationships were proposed for steel and concrete and then they were superimposed. The analytical results were compared with the experiments from the available literature.

<u>Experimental Study.</u> The specimens from the literature had *D/t* ratios varying from 15.5 to 73.9. The ranges for the compressive strength of concrete and the yield strength of steel were 3.68 to 13.21 ksi and 38 to 121.11 ksi, respectively. The *L/D* ratio of all the columns was 3.

In addition, the authors tested 4 CFT and 4 HT columns (see Nakahara and Sakino, 1998). The CFT columns had high strength concrete with compressive strength of 17.26 ksi. The yield strength of steel was varying from 44.96 to 113.28 ksi and the range of the *D/t* ratio was 31.3 to 64.7.

<u>Analytical Study</u>. The analytical study was performed based on three conclusions of the authors from past experimental studies. Local buckling was assumed to reduce the capacity of the columns with large *D/t* ratios, the capacity increase that took place for the columns with low

D/t ratios was attributed only to the strain hardening of the steel tube and, scale effect was introduced for the compressive strength of concrete.

For concrete in square CFT columns, a stress-strain relationship of confined concrete from the literature was adopted in the current study. The proposed model was similar to plain concrete in strength and it was similar to confined concrete in ductility. The yield strength of steel was assumed to affect the rate of strength degradation of concrete in post-peak region. Steel tube was considered to yield at large deformations and thus no restriction in steel strength was proposed. In the experiments, it was observed that the CFT columns had carried some axial load even in large deformation region. Therefore, concrete was assumed to carry 30% of its maximum stress in large deformation region.

Three stress-strain curves were proposed for steel tubes depending on the *D/t* ratio. For low *D/t* values, the steel tube was considered to experience strain hardening and the maximum strength was taken to be larger than the yield strength. In the case of the steel tubes with medium *D/t* values, the maximum strength was taken as equal to the yield strength. The maximum strength of the steel tubes with large *D/t* ratios was assumed to be smaller than the yield strength due to local buckling. The equation for the strain value corresponding to the end of strain-hardening region, for the steel tubes with low *D/t* values, was determined by a regression analysis of the test results. From the fact that CFT columns resisted some axial load in the large deformation range, the steel stress in each proposed model was assumed to remain constant following the falling branch after the peak strength. The authors named that region as the state of stability of CFT columns. In order to determine the strain value corresponding to the starting point of that region, the load-deformation curves of the tested CFT specimens were divided into 100 linear parts with a 0.04% strain increment. The strain interval with a tangent modulus smaller than 2% of the initial modulus of the column in absolute value and bigger than the tangent modulus of the previous interval was taken as the start of the state of stability. The corresponding strain value was used to calculate the stress value initiating the state of stability. The axial load at that strain level resisted by concrete was subtracted from the total axial load and the stress on steel tube was calculated. This stress value was determined for each specimen and then an equation was proposed by performing a regression analysis.

The analytical results matched accurately with the experimental load-deformation relationships of most of the specimens. However, the results for the high strength concrete specimens were not satisfactory and their axial load capacities were overestimated.

Schneider, S. P., 1998 (c; m)

Introduction. In this paper, the author presented an experimental study and results from a three-dimensional finite element analysis on the behavior of circular, square, and rectangular CFTs. The response of short columns under concentric axial load was investigated. The results were compared with the AISC LRFD (1993) design code provisions.

Experimental Study Results and Discussions. Three circular, five square, and six rectangular specimens were tested under monotonic axial load. The columns were simply supported. The *D/t* ratio of the specimens varied from 17 to 50. The nominal yield strength of steel and nominal compressive strength of concrete were 46 ksi and 2.9 ksi, respectively. The *L/D* ratios ranged between 4 and 5. The tubes were annealed and free from residual stresses.

The specimens showed different post-yield behaviors depending on their shape and *D/t* ratio. All of the circular specimens achieved hardening type post-yield response. Among rectangular and square specimens, two specimens that had the lowest *D/t* ratios also exhibited hardening behavior in their post-yield response. However, most of the other specimens underwent softening post-yield response.

The ductility level of the specimens was defined as the ratio of axial deformation at any point during the load history over the axial deformation at the point of yielding. If the yield point of the specimens were not clear, it was obtained by a 0.2% offset rule. No local buckling was observed before yielding of the columns. The local buckling of the circular specimens took place when the ductility levels were more than 10. For the square and rectangular hardening specimens, the ductility level at local buckling was between 6 to 8. In the case of the softening specimens, local buckling ductility levels were less than 6. The hardening columns underwent large deformations up to ductility levels greater than 10 before they reached the ultimate axial load. For the hardening specimens, it was found that the peak axial load was up to 1.41 times their yield load. On the other hand, for the softening specimens, the ductility level at the peak axial load was less than 2 and the peak axial load was approximately 1.07 times greater than the yield load. The increase in peak axial load observed in the tests was attributed to the confinement effect, and it was larger for circular sections.

The load carried by the steel section alone throughout the test was calculated from the longitudinal strain values of the steel tube. For thick sections, the load share of the steel tube maintained a constant value until yield load. However, it was found to increase in the case of thin sections. After yielding, the load carried by the steel tube was observed to decrease in strain hardening type specimens. The opposite trend was observed for the strain softening specimens. For most of the columns, circumferential strain over longitudinal strain ratios were close to Poisson's ratio of steel until 92% of yield load and then they were observed to increase up to 15%. This was indicative of concrete confinement. Local buckling was observed for every specimen and it was more intense in the case of large *D/t* values. The concrete at the locally buckled parts was investigated and generally it was observed to flow plastically without any granulation.

The axial strengths of the specimens were calculated by the method in the AISC LRFD (1993) specification. Good correlation was achieved with an average value of 1.08 for measured over predicted load ratios. These ratios exhibited large scatter with respect to column slenderness, λ_c, in AISC LRFD (1993) provisions. However, their correlation with respect to *D/t* ratios was better.

Analytical Study. The test specimens were analyzed using three-dimensional finite element analysis, through the use of ABAQUS. The same model was used to investigate the response of larger scale CFT columns that were common in current construction practice. Brick elements for the concrete core and shell elements for the steel tube were utilized. The available concrete material model in ABAQUS was selected, and it was assumed to represent adequately a low amount of confinement prior to yielding of the CFT column. The Von Mises yield criterion and the Prandtl-Resuss flow rule were chosen for the steel. No strain-hardening was assumed. The interaction between the steel and concrete was simulated by gap elements, and the coefficient of friction between the steel and concrete was assumed to be 0.25. Both the elastic and inelastic response of the specimens were closely correlated by the computational results, and the computational and experimental local buckling patterns were found to be similar. In addition to the experimental specimens, fifteen more finite element models for circular CFT columns were analyzed, covering *D/t* ratios ranging from 10 to 85. In these models, three different diameters of 2.7 in., 14.2 in., and 28.4 in. were selected. The nominal yield strength of the steel was taken as 46.0 ksi and the nominal concrete compressive strength was taken as 4.5 ksi in the analyses. The *L/D* ratio was chosen as 5. The column slenderness parameter, λ_c, in the AISC LRFD (1993) provisions for all of the steel tubes was approximately equal to 0.2. From the computational results, it was found that both the concrete and steel developed their compressive yield stresses when the CFT member reached its yield strength, which was calculated by a 0.2% offset rule for the analytical model . In the case of large diameter specimens, the steel tube could not develop its yield strength due to the detrimental effect of the biaxial stresses. From the concrete stress values at the peak axial load, the concrete strength enhancement due to confinement was found to be about 30% for small diameter specimens and 15% for large diameter specimens. The computational yield strengths of all of these specimens were also compared with the axials strengths predicted by the AISC LRFD (1993) provisions. It was found that for small diameter specimens, the ratio of the computational strength over design strength decreased with increasing *D/t* values. In the case of large diameter specimens, the predictions were close to the analysis results for small *D/t* values while the analysis results were overestimated for large *D/t* values.

<u>Han, L. H. and Yan, S., 2000</u> (c, m)

<u>*Introduction*</u>. An experimental study was conducted on slender columns. Both CFT and hollow steel tube columns were tested. The test results were compared with Chinese design code provisions.

<u>*Experimental Study Results and Discussions*</u>. Eleven circular CFT columns and four circular HT columns were tested monotonically under axial load. The specimens had slenderness ratios ranging from 130 to 154. The *D/t* ratio for each specimen was equal to 24. The *L/D* ratio ranged between 32.5 and 38.5. Two kinds of concrete mix were used. One of them had a measured cubic compressive strength of 4.6 ksi and the other one had a measured cubic compressive strength of 6.8 ksi. The measured yield strength of steel tube for all the specimens was 50.5 ksi. Simply supported end conditions were simulated by the test setup.

All the specimens exhibited relatively ductile behavior. For the hollow tube columns, the local buckling occurred at an earlier stage than the CFT columns. This showed that the concrete infill increased the local buckling strength. Both hollow tube columns and CFT columns failed at the mid-height by the rupture of the steel tube wall. The steel tube wall rupture took place either at the tension region following extensive concrete cracking or at the compression region due to local buckling. The capacities of the CFT specimens were found to be up to 30% higher than the capacities of the hollow tube specimens. However, there was not any significant difference between the strengths of the CFT columns filled with different kinds of concrete.

The capacities of the tested columns were also calculated using the Chinese design code provisions. They were compared with the experimental findings and the corresponding design code method was found to be conservative.

<u>Inai, E. et al., 2000</u> (no tests)

<u>Introduction</u>. In this paper, design equations to calculate the deformation capacity of circular and square CFT beam-columns were presented. For this purpose, a regression analysis was performed using the results of several experimental studies from the literature. The cyclic behavior of CFT beam-columns was also modeled.

<u>Analytical Study</u>. A database of the experiments that were selected to make the statistical comparison with the design equations was prepared. The value of the chord rotation when the lateral load capacity drops to 95% of its peak value (R_{95}) was defined as the deformation capacity for a beam-column. The factors influencing the deformation capacity were *D/t* ratio, yield strength of steel, compressive strength of concrete, axial load ratio, and shear span over depth ratio. For each of these factors, the trend of deformation capacity was determined by regression analysis of the experiments in the database. For both circular and square specimens, the value of R_{95} was found to decrease as the *D/t* ratio and the axial load ratio got larger. No clear trend was observed for the yield strength of steel and compressive strength of concrete. Using the analysis results, equations to calculate R_{95} were proposed as follows (stress values are in MPa):

For circular CFT beam-columns:

$$R_{95} = 8.8 - 6.7 \times (P / P_0) - 0.04 \times (D / t) - 0.012 \times f'_c$$

For square CFT beam-columns:

$$R_{95} = \frac{100}{0.15 + 3.79 \times (P / P_0)} (t / D) \beta$$

where $\beta = 1.0 - \dfrac{f'_c - 40.3}{566} \leq 1.0$

The proposed equations were compared with the experimentally obtained values from the database and good correlation was achieved. The shear span-to-depth ratio did not exist in the equations, but it was found that the results from these equations already gave the average values when the effect of the shear span-to-depth ratio was introduced into the equations.

To model the cyclic deformation response of the CFT beam-columns, a tri-linear skeleton moment-rotation curve was developed. The curve started with a linear part at a slope of K_e (elastic stiffness) until M_y, which was defined as the short-time allowable flexural moment strength in AIJ (1987). It was followed by a second linear region with a milder slope up to M_u (the ultimate flexural strength). The third part of the curve was a horizontal line that terminated at the ultimate chord rotation angle (R_u). A stiffness degradation factor (α_c) for the reduction in stiffness in the second part of the curve was defined. This factor was calculated by performing a statistical analysis of the experimental results, and it was taken as 0.65 for circular CFTs and 0.7 for square CFTs. The other factors required to define the skeleton curve were determined using available formulations from the literature. This model was used to estimate the cyclic response of specimens tested in the experimental studies. The hysteretic behavior was estimated well when chord rotation angles were smaller than 1%. However, energy dissipation capacity was overestimated for chord rotations outside of this range.

<u>Nakahara, H., and Sakino, K., 2000a</u> (bc; m, r)

<u>Introduction</u>. Monotonic and cyclic beam-column tests were conducted on square CFTs under constant axial load and uniform bending. The authors presented the experimental results and proposed an elasto-plastic analysis method for square CFT beam-columns.

<u>Experimental Study, Results, and Discussions</u>. The objective of the experimental study was to examine the axial and flexural behavior of square CFT beam-columns. Uniform bending moment and constant axial load were applied at the ends, which were attached to cylindrical bearings simulating pinned end conditions. The main parameters were axial load ratio, *D/t* ratio, and deformation histories (monotonic and cyclic). The *D/t* ratio ranged between 34 and 98. The axial load ratio was either 0.2 or 0.4. The nominal strengths of concrete and steel were 7.25 ksi and 58.01 ksi, respectively. The steel tubes were annealed and as such they were free from residual stresses. The *L/D* ratio was 3 for all of the specimens. The loading was controlled by a dimensionless parameter obtained by multiplying the mid-height curvature by the depth of the specimen.

The flexural behavior of the cyclic specimens was mainly affected by the axial load ratio. The specimens with high axial load ratio showed a sharp drop in strength after achieving the peak strength, as well as a relatively high axial strain. On the other hand, the response of the specimens with low axial load ratio was stable and exhibited low axial strain, as may be expected. The effect of the *D/t* ratio was negligible for low axial load ratios and the response of these specimens was similar to the monotonic behavior. A rapid decrease in moment capacity was observed as the *D/t* ratio increased for the specimens subjected to high axial load.

<u>Analytical Study</u>. The moment-curvature behavior of the specimens was obtained numerically using a fiber analysis. The concrete and steel stress-strain curves were based on a previous study by the authors. However, the descending branches were calibrated to account for the strain gradient existing in a column section under bending. The unloading and reloading parts of the steel and concrete stress-strain curves were adjusted from other curves found in the literature. The analytical results predicted the experimental behavior accurately. The moment capacities of the specimens were also calculated by full plastic moment and the results matched with experimental values unless the *D/t* ratio was large.

<u>Nakahara, H. and Sakino, K., 2000b</u> (no tests)

<u>Introduction</u>. In this paper, an analytical study to estimate the load-deflection response of square CFTs under eccentric axial load was presented. The theoretical results were compared with the load-deflection curves of specimens tested in Japan. In addition, a correlation was also investigated between the experimental peak moments and the ultimate strength moments computed by the AIJ (1997) and ACI (1989) design code provisions.

<u>Experimental Study</u>. Twenty-one square CFT columns were tested under eccentric axial load as a part of the research program. The main parameters of the tests were yield strength of

steel tube, compressive strength of concrete, magnitude of eccentricity, and *D/t* ratio. The ranges for concrete and steel strength were 3.86-11.64 ksi and 38.00-89.64 ksi, respectively. The *D/t* ratio was varying between 22.8 and 73.8 and the *L/D* ratio for all the specimens was equal to 3.

Analytical Study. In the previous work of the authors (see Nakahara, H. et al., 1998), axially loaded square CFT columns were studied. The proposed steel and concrete compressive stress-strain relations worked well to predict the experimental response of axially loaded specimens. In the current study, similar compressive stress-strain curves were used. However, the slope of descending parts was milder to simulate the more ductile behavior seen in the tests with flexure due to eccentric loading. In addition, the strain gradient along the section was accounted for, with a linear strain distribution being assumed.

While calculating the load-deflection response of the specimens, the deflected shape was taken as a sine wave. Based on this assumption, the relation between mid-height curvature and mid-height lateral deflection was derived. Using the equilibrium condition, moment-curvature-thrust curves were then formed. It was found that ductility was affected primarily by the *D/t* ratio and compressive strength of concrete rather than the yield strength of the steel or the eccentricity. The analytical moment-curvature-thrust curves showed good correlation with the experimental results and predicted the post-buckling response of the specimens accurately. However, the initial flexural stiffness was overestimated, which was attributed to residual stresses existing in the specimens. The moment capacities of the specimens were calculated by the methods in AIJ (1997) and ACI (1989). The concrete compressive strength was multiplied by a reduction factor ranging from 0.855 to 0.960 in both methods to account for the scale effect. The experimental capacities were overestimated by AIJ (1997) and underestimated by ACI (1989).

Varma, A. H. et al. (2000, 2001) (c, bc; m, r)

Introduction. This summary highlights the results presented in several papers and reports (Varma et al., 1998a, 1998b, 1998c, 1999; Varma, 2000; Varma et al., 2000, 2001a, 2001b, 2001c) on the behavior of square concrete-filled steel tubes manufactured from high strength materials. Both experimental and computational researches are presented.

Experimental Study, Results and Discussions. Monotonic and cyclic loading tests were carried out on three-fourth-scale square CFTs. The main parameters of the experimental tests included the type of steel, the *D/t* ratio and the level of the axial load. Each specimen was filled with 15.95 ksi concrete. Either A500 Grade-B or A500 Grade-80 type of steel was used.

Four stub-column specimens were tested monotically under concentrated axial compression. Force controlled loading was applied and fix-fix support conditions were provided. The response of the columns up to the peak load was approximately linear. After the peak load, unloading was observed due to concrete crushing and local buckling of all four flanges. This was followed by a sudden decrease in axial load capacity due to extensive local buckling and concrete crushing. After this stage, the loading proceeded under displacement control and two trends were observed among the specimens. Most of the specimens underwent more plastic deformation while maintaining their remaining moment capacities. However, for one of the specimens, tearing of the seam weld took place and the moment capacity decreased with additional inelastic deformation.

Eight beam-column specimens were tested with constant axial load and monotically increasing end moments. Pin-pin support conditions were simulated by the test setup. Reduction in flexural stiffness was observed with the concrete cracking and concrete crushing. Steel yielding took place both in the compression and tension flanges before the peak moment. For only one specimen, steel yielding in tension flange occurred after the peak moment was reached and failure did not take place at the mid-height. This might be due to some problems associated with the test setup of this specimen. In most of the cases, concrete crushing took place before local buckling. Local buckling of the compression face and extensive concrete crushing were observed at the peak load This resulted in a decrease in the capacity. In later stages after the peak response, local buckling of the flanges propagated to the web. Tearing of the steel tube also occurred for some of the specimens.

Eight cyclic beam-column specimens were tested under a constant axial load and a cyclically varying lateral load. The base of the specimens was fixed and the lateral load was applied at the top. The elastic cycles were performed under load control. For the inelastic cycles, displacement-controlled loading was applied at seven displacement levels ranging from Δ_y to $8\Delta_y$. The yield displacement (Δ_y) for a cyclic specimen was estimated during the elastic cycles using its secant flexural stiffness. The flexural stiffness of the specimens started to decrease with concrete cracking and steel yielding. In some cases, concrete crushing took place earlier than local buckling, but generally the peak lateral load was reached with the occurrence of both concrete crushing and local buckling of the flanges. The local buckling propagated to the web with additional displacements. This caused rapid decrease in strength. Local buckling of the tube corners and tensile facture of the steel tube wall took place in the later stages of the loading.

For the stub-column tests, the specimens with lower *D/t* ratios were found to have larger axial stiffness. Increasing the grade of the steel caused an enhancement in the peak axial load, and the degree of improvement was higher for smaller *D/t* ratios. The AISC LRFD (1993), ACI (1995), and AIJ (1987) design code provisions gave conservative estimates for the axial compressive strength of the stub-column specimens except the one that failed by elastic local buckling. On the other hand, the Eurocode 4 (1996) estimates were not conservative for all the specimens. This was attributed to the fact that the reduction in concrete strength due to size and curing effects was not applied in Eurocode 4 (1996). The transformed axial stiffnesses were found to match with the experimental results accurately.

In the beam-column tests, the initial section flexural stiffnesses of the specimens were observed to decrease with increasing *D/t* ratio. However, no significant change occurred when the steel grade was varied. Using the moment-curvature response, the authors calculated a serviceability level section flexural stiffness as the secant section flexural stiffness corresponding to 60% of the peak moment. Excluding one specimen, it was found that the serviceability level flexural section stiffness increased for higher magnitude of axial loads. However, it decreased for greater *D/t* ratios. The initial section flexural stiffness and serviceability section flexural stiffness showed good correlation with the cracked and uncracked transformed section stiffnesses, respectively. The peak moment values for the specimens improved with a decrease in *D/t* ratio and an increase in steel grade. However, the amount of enhancement varied according to the inelastic response of the high strength steel without local buckling. The authors defined a curvature ductility ratio as the ratio of ultimate curvature to the yield curvature. They computed this value for each specimen and concluded that the specimens with lower *D/t* ratio and lower axial load showed more ductile behavior. However, steel strength did not affect this ratio. The test results for the beam-column specimens were compared to the AISC LRFD (1993), ACI (1995), AIJ (1997), and Eurocode 4 (1996) design code provisions, the ACI (1995) predictions showed the best correlation.

When the cyclic test results were compared to the monotonic beam-column tests, the moment capacity was found to be approximately the same, but the peak moment of the cyclic specimens decreased more quickly. The initial flexural stiffness of the cyclic specimens was not affected by the axial load ratio and the steel strength. However, increasing the *D/t* ratio reduced the initial flexural stiffness. On the other hand, a high axial load ratio increased the contribution of concrete to the behavior and this resulted in a higher secant flexural stiffness. When the effects of steel grade and *D/t* ratio on moment capacity were examined, it was found that the increase in steel strength enhanced the moment capacity but the level of improvement was higher for the specimens with a lower *D/t* ratio. A decrease in the *D/t* ratio also improved the capacity, and the level of improvement was better for the specimens with a lower grade of steel. From the test results, curvature ductility of the specimens was also investigated. The curvature ductility was calculated for the failure segment at the base of the specimens. It was defined as the average value of the ratio of the post-peak curvature at 90% of the peak moment to the pre-peak curvature at the same moment level. For high axial loads, a significant decrease in curvature ductility was observed. When the axial load level was high, the influence of *D/t* ratio and steel strength on the curvature ductility was low. For low levels of axial load, increasing the *D/t* ratio and steel strength decreased the curvature ductility. When the section elastic flexural stiffness was examined, it was found to be close to the transformed section stiffness at early stages of loading. Due to concrete cracking and local buckling, the elastic stiffness decreased to values below the flexural stiffness of the steel tube alone. The energy dissipated throughout the test was calculated from the lateral load and lateral displacement response, as well as, from the cyclic moment curvature response of the failure segment. It was found that most of the energy was dissipated through flexure in the failure segment. The energy dissipation was low for specimens with high axial load level and larger *D/t* ratio. It was not affected noticeably by the grade of the steel. Axial shortening of the cyclic specimens was also monitored throughout the test. The axial shortening started at an earlier stage of loading and resulted in a larger permanent deformation for the specimens with a low level of axial load compared to the ones with a high level of axial load. The test results were compared to the AISC LRFD (1993), ACI (1995), AIJ (1997), and Eurocode 4 (1996) design code provisions, and the ACI (1995) predictions again showed the best correlation.

Analytical Study. Three-dimensional finite element analyses of the stub columns were done using the finite element program ABAQUS. The finite element models were able to account for local buckling, confinement, and composite interaction. In the analyses, one fourth of the CFT cross section was modeled and the Modified Riks Algorithm was executed with displacement control. For the steel tubes, S-4 shell elements were selected. The stress-strain response obtained from the coupon tests in the experimental study was used as the uniaxial constitutive steel model. In the elastic range, an isotropic multiaxial constitutive steel model was used, and in the inelastic range, a plasticity-based multiaxial constitutive model utilizing a Von-Mises yield surface was selected. The concrete was modeled by 3D continuum elements. The stress strain relationships obtained from the cylinder compression tests were taken as the uniaxial constitutive concrete model. An elastic isotropic multiaxial constitutive model and a plasticity-based multiaxial constitutive model using a 2-parameter Drucker-Prager compression yield surface were chosen for the concrete in the elastic and inelastic ranges, respectively. The composite behavior between the concrete and the steel was modeled by uniaxial gap contact elements to account for the transverse interaction. Rigidly plastic springs were generated for simulating the bond between concrete and steel. Due to the confining pressure from the corners of the square tube, the concrete has core regions in a state of tri-axial stress. The inelastic multiaxial constitutive concrete model was calibrated for these regions by adjusting the two parameters accounting for the effect of confinement on the yield surface and the effect of confinement on the plastic strain tensor. For this purpose, an equation from the literature to predict the strength of the confined concrete was used. The calibration of the concrete model for the non-core regions in biaxial stress state was not done due to lack of experimental data. The first finite element analysis was conducted without introducing any imperfection in the specimen. In the elastic range, Poisson's ratio for the steel was greater than that of the concrete. This resulted in gap opening and no interaction between the steel and the concrete. In the inelastic range, contact forces developed between the steel and the concrete. This caused composite action to initiate. The confinement of the concrete occurred and tensile circumferential stresses developed in the steel. These stresses reduced the longitudinal compressive capacity of the steel tube. The peak load was reached due to inelastic response of the steel and concrete without any local buckling. The peak loads were overestimated and the post-peak responses were not close to the experimental results. The second finite element analysis was conducted by assuming an imperfection at the mid-height of the column. The type of the imperfection was determined from the local buckling pattern of the hollow steel tubes. The correlation of the analysis results with the experimental values for the peak loads and elastic axial stiffnesses were good. However, a failure segment formed at the imperfection region due to longitudinal strain concentration. The formation of the failure segment caused elastic unloading of the remaining sections, which reduced the axial ductility of the columns. As a result, post peak response predicted by the finite element analysis was not convergent to the experimental results. The third finite element analysis was performed for the failure segments in the imperfection region. Inelastic behavior of the steel and concrete was observed but the failure was due to local buckling. The response of the failure segments were combined with the response of the remaining elastically unloaded parts to get the overall response of the column specimens. The calculated response was close to the experimental behavior.

Fiber-based finite element models were developed for the monotonic beam-column specimens in DRAIN-2DX. The inelastic responses of the specimens were mostly affected by the 12 in. length failure segments. Therefore, fiber-based models were developed for these failure segments. The tensile uniaxial stress-strain curve for steel was adopted from the coupon test results. To determine the uniaxial compressive stress-strain curves of the steel and concrete fibers, 3D finite element analysis of the failure segments were conducted in ABAQUS under axial compression. The analysis method was similar to the one presented above for the stub column specimens. Then, using the analysis results of each finite element, effective compressive stress-strain curves were defined and used for the steel and concrete fibers. These models implicitly accounted for the effects of concrete confinement, local buckling and biaxial stress state in steel tube. The fiber-based analysis results estimated the moment capacity and M versus ϕ response of the monotonic beam-column specimens with reasonable accuracy. The cyclic beam-column specimens were also analyzed with the fiber-based finite element method. The uniaxial stress-strain curves of the monotonic beam-column specimens were adopted for the envelopes of the cyclic stress-strain curves of the cyclic beam-column specimens. The same cyclic loading history in the experiments was utilized in the analysis. Interaction curves were generated by analyzing the specimens under various axial load levels. The theoretical results showed good correlation with the experimental moment capacities and *M* versus ϕ responses of the cyclic beam-column specimens.

Zhang, S. and Zhou, M., 2000 (c; m)

Introduction. An experimental and an analytical study on square CFT columns were presented. The steel tube response was isolated from the overall behavior of the specimens and the response of the concrete and steel were examined separately. Formulations were proposed for the confined concrete strength, the confined concrete strain, and the longitudinal stress in the steel tube.

Experimental Study, Results, and Discussions. Thirty-six CFT columns were tested under monotonically applied axial loading. The *D/t* ratio of the specimens ranged between 20 and 50. The measured compressive strength of concrete was 5.87 ksi and the yield strength of the steel ranged from 41.28 ksi to 58.51 ksi. The *L/D* ratio varied between 4 and 5. From the experimental results, it was found that the confinement effect increased the concrete strength and ductility. They also determined that the longitudinal stress in the steel tube was always less than the yield stress due to the biaxial stress condition. Confinement was found to be larger when the *D/t* ratio was smaller.

Analytical Study. By fitting a curve to the test data, an equation for confined concrete strength, f_{cc}, was derived as (all stress values are in MPa):

$$\frac{f_{cc}}{f'_c} = 1 + 1.891\sqrt{\frac{\sigma_r}{f'_c}} + 0.198\frac{\sigma_r}{f'_c}$$

where

$$\sigma_r = \frac{2t}{D-2t}\sigma_{sc},\ \sigma_{sc} = -0.5\sigma_{sl} + (f_y{}^2 - 0.75\sigma_{sl}{}^2)^{0.5}$$

The ultimate concrete strain and the longitudinal stress in the steel tube were also determined as follows through fitting to the test data:

$$\frac{\varepsilon_o \times 10^6}{f_y} = 0.171\xi^2 - 0.0944\xi + 1$$

$$\frac{\sigma_{sl}}{f_y} = 0.898 - 0.0607\ln(\xi)$$

where

$$\xi = \frac{A_s f_y}{A_c f_c}$$

The proposed equations were compared with the experimental results from the current study and from the available literature. Good correlation was achieved.

<u>Elchalakani, M. et al., 2001</u> (pb; m)

<u>Introduction</u>. The flexural behavior of circular CFTs was investigated. Twelve CFT beams were tested under pure bending. The strength, deformation capacity, and energy dissipation of the specimens were monitored and compared with those of hollow tubes tested by the author and reported in the literature. The authors recommended a *D/t* ratio for circular CFT beams to achieve their plastic moments. Moreover, they proposed a formulation to calculate the moment capacity of circular CFT sections.

<u>Experimental Study, Results and Discussions</u>. The testing machine for these monotonic experiments was displacement-controlled and had the capability of applying constant moment at the mid-length of the specimens without creating any significant axial load or shear force. Nine compact specimens had *D/t* ratios smaller than 40 while the *D/t* ratios for the three slender specimens varied between 74 and 110. The average measured yield strength of steel and compressive strength of concrete were 60.77 ksi and 3.39 ksi, respectively. The *L/D* ratio for the slender specimens was approximately 5.4, versus ranging from 7.9 to 23.7 for the compact specimens.The compact specimens did not undergo any local buckling or tension fracture. The moment-curvature and moment-rotation responses of the specimens indicated that the ductility and energy absorption capacity improved due to the concrete infill. The improvement was more pronounced for the compact specimens with the higher *D/t* ratios. The moment strength was also greater than the equivalent hollow tubes and the amount of enhancement, which was again more apparent in the case of higher *D/t* ratios, ranged from 3% to 37%. The ovalization of the specimens due to radial distortion was approximately 1% and it did not affect the test results significantly.

The general moment-rotation response of the slender specimens was represented by a series of linear and non-linear curves. According to the idealized moment-rotation response, the response curve became nonlinear when the rotation was equal to θ_y. At a rotation level of $1.5\theta_y$, initiation of local buckling was observed. Following this stage, the specimens underwent strain hardening in the tension region. Later, local buckling began to increase and a gradual reduction in strength took place until failure. For one specimen, failure was sudden and the gradual reduction in strength was not observed. This was attributed to the early fracture at the seam weld as it was applied below the centroid of the steel tube. For the others, failure occurred with fracture at the outermost tension fiber, with the crack propagating towards the neutral axis.

The bond between the steel and concrete was also examined. No abrupt change in the moment-curvature response was noticed and this showed that slip did not occur between the steel and concrete. It was suggested that an a/d ratio of 2.7 was sufficient to ensure no slip.

<u>Analytical Study</u>. To obtain a limiting slenderness and *D/t* ratio for transition between compact and slender sections, a slenderness parameter (α) and rotation capacity formula were

selected from literature. These two quantities were calculated for each specimen using the following equations:

$$\alpha = \frac{E/f_y}{D/t} \qquad\qquad R = \frac{\theta_{max}}{\theta_y} - 1$$

A rotation capacity versus slenderness parameter graph for the tested specimens had a clear transition point between compact and slender sections. The slenderness parameter corresponding to that point was taken as the slenderness parameter limit (α_p) and it was inserted into the equation for slenderness of the sections. The limiting *D/t* ratio and plastic slenderness limit (λ_p) for 60.77 ksi steel were then calculated as 112 and 188, respectively.

$$\lambda_p = (\frac{D}{t})_{\lim it} \times (\frac{f_y}{250}) = (\frac{E/f_y}{\alpha_p}) \times (\frac{f_y}{250}) = \frac{E}{\alpha_p .250} \quad \text{(stress values are in MPa)}$$

The limiting *D/t* ratio was about 14% greater than the limits calculated according to the AIJ (1987) design code provisions if measured steel yield strength was used. This was attributed to the presence axial force in the experiments that were used to derive the equations in AIJ (1987).

By assuming perfect bond and performing a rectangular stress block analysis, an equation to calculate the moment strength of the CFT sections was proposed. Good correlation was obtained with the experimental results and the moment strength values from EC4 (1992) and CIDECT (1995) procedures.

Han, L. H. et al., 2001 (c, bc; m)

Introduction. A series of monotonic tests were conducted on square CFTs including stub-columns, columns, and beam-columns. In addition, the authors presented analytical models to estimate the capacity and load-deformation response of the specimens.

Experimental Study, Results, and Discussions. The objective of the experiments was to investigate the strength and failure patterns of CFTs. Two sets of experiments were conducted. In the first set, twenty stub-columns were tested. Eight columns and twenty-one beam-columns were tested in the second set. The authors defined a confinement factor ξ (P_{so}/P_{co}) to account for the composite action between steel and concrete. This factor was used as a parameter in each set of experiments, with a range of values varying from 1.08 to 5.64. Other parameters included concrete strength, *D/t* ratio, eccentricity, and slenderness. The average measured yield strength of steel was 47.14 ksi and the measured cubic concrete strength ranged between 2.35 ksi and 7.15 ksi. The *D/t* ratio varied from 20.5 to 36.5.

The stub-columns were tested under load control. Two stiff base-plates were welded to the ends of the specimens. Local buckling was observed near or at failure. The columns with a high confinement factor maintained their strength after peak load. However, a rapid decrease in strength was observed for the specimens with a low confinement factor.

In the second set of experiments, the specimens were tested under axial load with eccentricities ranging from 0 to 3.15 inch. The column slenderness, which was defined as (L/r) $\times\sqrt{(f_y/235)}$ varied from 45 to 75 and the values of *L/D* ratio were between 11 and 18, approximately. The tests were performed under load control and pin-pin support conditions were simulated by the test setup. The eccentricity was provided by thick end plates with an offset triangular wedge. The failure of the specimens was due to overall buckling.

Analytical Study. The authors proposed stress-strain relations for steel and confined concrete. The stress-strain model for confined concrete was dependent on the confinement factor. Using these two models, a combined stress-strain curve was derived for the composite section. It was assumed that there was no slip between the steel and concrete. The predicted response was close to the stub column test results. The authors performed a regression analysis and obtained equations for cross-section strength and cross-section modulus (E). The equation for axial cross-section capacity was compared with AISC-LRFD (1993), AIJ (1997), and EC4 (1996) design code provisions using the experiments of different researchers. The proposed method showed the best correlation with the experimental results.

For columns and beam-columns, load versus mid-span deflection relations were derived using the proposed stress-strain models of confined concrete and steel. For this purpose, the deflection curve of the member was assumed as a sine wave. A basic layered approach was used, in which the cross section was divided into small rectangles parallel to each other along the width. The rectangle elements consisted of separate steel and concrete portions. The moment and axial load capacities were expressed as the summation of the moment and axial load from each rectangle element. Using these equations, the load versus mid-span deflection relations for the specimens were obtained. The predicted curves matched with the experimental results accurately. Simplified equations for axial load capacity and moment capacity were derived based on the linear regression studies. Interaction diagrams were generated for the specimens tested by several researchers using the proposed equations and the methods in the AISC LRFD (1993), AIJ (1997), and EC4 (1996) specifications. When the experimental and analytical results were compared, the proposed equations showed the best correlation.

Kawaguchi, J. et al., 2002 (fr; r)

Introduction. An experimental study on portal frames consisting of steel I-girders framing into square CFTs subjected to cyclic loading was presented in two papers (see also Kawaguchi et al., 1997). Using the test results, the elasto-plastic behavior of the CFT frames was examined.

Experimental Study, Results, and Discussions. Four specimens having square CFT columns and wide flange steel girders were prepared. The columns had a D/t ratio of 20.83 and an L/D ratio of 8. The measured yield strength of the steel was 58.5 ksi for the columns and 49.7 ksi for the girders. The measured average concrete strength was 2.67 ksi. The columns were fixed at the base and connected to the girders using fully-restrained connections. Through type diaphragms were used to connect the girders to the CFT columns. Two of the specimens were designed to have a shear type of failure in the panel zone of the CFTs prior to failure due to combined axial force and flexure took place in the CFT columns. These specimens were designated as panel-yielding specimens. The other two were proportioned to have failure along the length of the CFT columns occur earlier than failure within the connection region; these specimens were designated as column-yielding specimens. The girders were designed to remain in the elastic range.

The frames were tested under constant axial load, with 15% and 30% of nominal axial strength applied to the CFT columns. The base-beam of the columns was subjected to a cyclic horizontal load while the original position of the column top ends was kept stationary. The column-yielding specimen that was loaded with 15% of the nominal axial load experienced a rapid reduction in strength after achieving its peak strength. After fracture took place at the weld connecting the external diaphragm to the column, the strength of the frame decreased severely. The response of the panel-yielding specimen subjected to 15% of its nominal axial load was more stable. Local buckling did not significantly affect the response of the panel yielding-specimen. However, weld fracture between column and diaphragm also took place for this specimen. This type of fracture observed in the welded connections was attributed to defects in welding process. The remaining two specimens were subjected to 30% nominal axial load. The failure of the column-yielding specimen was governed by local buckling. The failure of the panel-yielding specimen was dominated by shear buckling of the panel zone. However, for both of the frames, no significant strength degradation was observed, and their post-peak behavior was ductile and stable.

The strengths of the frames were predicted conservatively using the AIJ design code provisions. It was concluded that local buckling of the column ends and shear buckling of the panel zone did not affect the strength of the frames significantly.

Analytical Study. The use of a D_s factor in Japanese practice to reduce the required strength of a ductile frame was discussed. This factor was dependent on the plastic deformation capacity of the frame. The cyclic hysteresis loops were utilized to obtain the plastic deformation capacity of a tested frame. For this purpose, the positive halves of the loops at each cycle were added sequentially and a curve tangent to these loops was assumed to represent the monotonic load–deformation curve of that particular frame. This curve was then fitted to an elasto-plastic curve having the same area. Using the ratio of the yield deformation to the ultimate deformation, both plastic deformation capacity and the D_s factor were calculated. This procedure was repeated for each specimen. The D_s factors were found to be less than 0.25, which was the minimum value specified for the most ductile steel structures according to the Building Standard of Japan. Thus, it was concluded that the deformation response of the CFT frames was similar to that of the steel frames.

2. CONNECTION TESTS

Dunberry, E. et al., 1987 (sp; m)

Introduction. Short square CFT columns were loaded axially by means of shear tab connections coupled with direct axial compression. The axial load capacity of the specimens and the load transfer mechanism between steel and concrete were investigated. Using the test results, the authors proposed an equation for the cross section strength of CFT columns.

Experimental Study, Results, and Discussions. A total of four series of tests (A, B, C, and D) were performed on CFT columns. Axial load was applied to the specimens both at the top of the CFT and at the connections. The ranges for the *D/t* and *L/D* ratios were 20.8-37.1 and 3.7-29.4, respectively. The nominal yield strength of the steel was 50.77 ksi and the measured compressive strength of concrete varied from 2.52 to 4.29 ksi. The ratio of the axial load applied through the connections to the total axial load (β) was changing from 0 to 1. All of the specimens were grouted at the bottom, while grouting was applied to the top if the specimens were subjected to an axial load at their top ends. In the case of no axial load acting at the top of a specimen, either grouting and steel capping were applied together or the top end was left without grouting and capping. Bracing against overall buckling was provided for all of the specimens. Test series A consisted of columns having standard-tee connections and the smallest *D/t* ratios. Test series B was identical to test series A, but higher *D/t* ratios were used. Test series C included the specimens having no grouting or capping at the top, and the specimens in this test series that did not have any load applied at the connections. In test series D, the columns had different connection details including standard- tee, extended-tee, single plate, and shortened-tee details. For all of the specimens, the steel tube and concrete core were instrumented separately to measure their strain values.

In most of the experiments, the total axial load capacity achieved in the specimens was close to their squash loads. The specimens loaded through the connections tended to have less strength compared to the specimens loaded only at the top. The difference in strength was at most 8%. The steel and concrete strains along the column length exhibited incompatibility at the connection regions and the relative slip between concrete and steel was found to be between 0.00315 and 0.00630 inches. The development length over diameter ratio was close to 3 at the top of the connections, and it was varied from 1 to 2 at the bottom of the connections. The failure pattern was generally local buckling, which took place within the connection region or below the connection region. Some specimens experienced overall buckling due to low stiffness and strength of the bracing. The axial load carried by concrete was found to increase along the connection, with the largest gain in force taking place in the bottom half of the connection. The concrete load also continued to increase below the connection region.

Local buckling was observed in the test series D. For single plate connections, the location of local buckling was a distance 4*D* below the connection, and a rapid increase in concrete load was observed in the bottom half of the connection due to concentrated pinching action of the plate. On the other hand, local buckling took place close to the connection region in extended-tee type connections, and a gradual transfer of load to the concrete was observed. The rotation of joints took place without slip at the bolt holes. This showed that load transfer to the concrete might also occur by joint rotation. Local buckling generally took place after yielding of the steel. Thus, the presence of concrete did not improve the capacity of these CFT columns significantly. However, it increased the failure strain of the specimens by between 2 to 29%. The occurrence of local buckling was mainly affected by the transfer of shear force to the concrete within the connection length. If the shear load transferred to the concrete within the connection was large, a lesser amount of load could be transferred to the concrete outside of the connection. This might cause excessive steel stress and subsequent local buckling in that region.

Using the experimental results, the following formulation was proposed to calculate the cross section strength of CFT columns (all stress values are in MPa):

$$P_o = A_s F_y + \alpha\gamma A_c f'_c$$

where

$$\gamma = 1 - 1.2\beta\left[\frac{\alpha A_c f'_c}{A_s F_y + \alpha A_c f'_c}\right]$$

The γ factor accounted for the reduction in concrete strength due to local buckling of steel tube. The reduction in concrete strength from size and curing effects was accounted for with the factor α (which ranged between 0.85 or 1).

Kamba, T. et al., 1991 (pz; m)

Introduction. The panel zone behavior in moment-resisting CFT frames was studied in this research. Monotonic tests were conducted on square HT and CFT beam-to-column connections having through diaphragm details. The strength and rigidity of the panel zone was monitored throughout the test. Furthermore, the authors proposed equations to estimate the elasto-plastic behavior of joint panels.

Experimental Study, Results and Discussions. The test setup consisted of a column having a through-diaphragm joint at mid-height. For the CFT connections, only the joint region was filled with concrete. The specimens were designed for failure to occur in the panel zone. Two series of tests were conducted. For the specimens in the first series, the panel zones were annealed, while cold-formed tubes were used in the second series. The main parameter of the tests was the D/t ratio, which was varied from 27 to 48. The range for the yield strength of the steel tube was 48.38 to 52.65 ksi and 54.07 to 56.92 ksi for the annealed and cold-formed specimens, respectively. The compressive strength of the concrete was 3.37 ksi in the first series and 3.56 ksi in the second series.

The failure of the hollow panel zones was due to shear buckling. The concrete-filled panel zones exhibited bending failure as a result of excessive shear deformation. The yield shear force (Q_y) was defined as the shear load when the stiffness decreased to one third of its initial value. The ultimate shear force (Q_u) was taken as the shear load at failure. It was found that the Q_u/Q_y ratio decreased as the D/t ratio got larger for the specimens with hollow panel zone. On the other hand, in the case of the concrete filled panel zones, Q_u/Q_y value was greater for large D/t ratios. Eliminating residual stresses caused an increase in Q_u/Q_y ratio for the hollow panel zones. In addition, it enhanced the deformation capacity. The initial rigidity of the concrete-filled specimens was found to decrease after concrete cracking. However, the specimens continued to exhibit stable behavior as the steel tube limited the concrete deformation. After the shear stress of concrete reached to 50-60% of its compressive strength, the shear strains started to increase while the shear stress remained constant.

Analytical Study. Tri-linear load-deformation curves were proposed for the steel tube and concrete parts of the panel zone. In both cases, it was assumed that shear stress remained constant after it reached its maximum value. The two load-deformation curves were then superimposed. The theoretical results approximately estimated the experimental behaviors. The authors recommended more research to be done for the load-deformation behavior of the hollow tube specimens.

Prion, H. and McLellan, A., 1992 (sp; m)

Introduction. The authors conducted experimental studies of through-bolt connections between steel wide-flange beams and concrete-filled hollow steel sections, and they compared their results to other types of CFT connection. Documentation of the transfer of beam shear forces (i.e., simple shear connections) to the concrete core was the focus of the study. Although through-bolt connections are capable of transmitting moment by thickening the end plate, extending the end plate past the flanges, and adding more bolts to the connection detail, this behavior was not investigated. The bolt shear capacity was the limiting factor in this connection design.

Experimental Study, Results, and Discussion. Only simple connections capable of transmitting shear forces were considered in his study. End plates welded to the beams were bolted to the hollow steel section with four bolts made of grade 4340 steel rod with an ultimate strength of 155 ksi. The columns were 12 in. square with a *D/t* ratio of 24 and were completely filled with 5.8 ksi concrete. The concrete should carry a large proportion of the beam's shear force, hence the need for a connection which bears directly on the concrete core. In previous studies (Kanatani et al., 1988), columns were filled with concrete only in the vicinity of the connection detail to prevent local buckling or crushing of the hollow steel section.

Three primary modes of failure governed the through-bolt connection detail. The bolts may experience a shear failure through the shank of the bolts, or the concrete may fail as a diagonal crack develops across the hollow steel section, transverse to the longitudinal axis. Finally, splitting of the concrete may occur, pushing the concrete and hollow steel section walls outward.

Specimens were divided into two categories, bearing and non-bearing, in order to separate the effects of friction and bearing. The non-bearing specimens were fitted with plastic tubes that provided clearance holes for the bolts to avoid bearing between the bolts and the concrete and to permit relative movement between he concrete and the steel casing. The bolts were cast directly into the concrete for the bearing tests. To determine the relationship between friction and bolt tension, bearing and non-bearing specimens were tested with bolts post-tensioned to 0 kips, 20.23 kips, and 40.47 kips.

For the bearing cases, slip between the hollow steel section and the concrete core was continuous with no sudden changes in load or slip deflection. However, in the non-bearing specimens, slip between the concrete core and the hollow steel section occurred abruptly and was accompanied by a decrease in load; successive occurrence of slip took place at higher loads. The increase in slip load was a result of an increase in bolt tension.

A constant relationship existed between post-tension and slip load in non-bearing specimens. The critical region for bolt failure was at or near the interface of the concrete and steel casing. The calculated bearing loads (total loads minus slip load) were far larger than those prescribed by design codes. The moment in the bolt at the midsection was also assumed to be negligible. The bolt may generate splitting forces on the concrete, especially in the presence of thin walled steel tubes that provided little confinement to the connection area.

The authors indicated that it is most important to insure that shear loads are transferred from beams to the concrete core due to the friction between the steel and concrete and bearing of the bolt on the concrete core. Checks must also be performed on the shear capacity of the bolts and the bearing capacity of the steel plates. The bearing stress distribution affects the stress on the bolts little; this is a valuable consideration since connection detailing is usually chosen to factor ductile failure modes, such as end plate binding or beam yielding, instead of bolt failure.

The authors indicated that the construction sequence initiates with the structure being erected with snug-tightened bolts. The columns are then filled with concrete and the bolts are post-tensioned after the concrete has cured.

Azizinamini, A. and Prakash B. A. 1992 (fs; m)

Introduction. A through-beam connection detail was considered both analytically and experimentally in this research. The through-beam connection detail prevents the transfer of large beam shear forces directly to the steel tube. This helps to prevent the steel tube from pulling away from the concrete core. Additionally the through-beam connection detail eliminates the need for welding thick connection elements to relatively thin steel tubes, which results in lower residual stresses then direct welding. A behavioral model was developed and the force transfer mechanism of the through-beam connection detail was identified. A tentative design approach was suggested.

Analytical Study. The qualitative behavior of the through-beam connection detail was considered using two- and three-dimensional non-linear finite element methods. Two cases were analyzed; in the direct connection detail, the beam was directly connected to the steel tube, while in the through-beam connection detail, the beam was passed completely through the column.

In the direct connection detail, very high stress concentrations occurred in both the steel tube and the concrete in the vicinity of the connection. Additionally, the steel tube separated from the concrete, resulting in large tensile stresses in the tube wall. Significant tensile stresses developed in the concrete around points modeling the shear stud locations.

When the through-beam connection detail was examined, considerably lower stresses were observed in the detail compared to the direct connection detail. Significant reductions were noted in the magnitudes of the tensile pull-out stresses in the steel tube, the compressive stresses in the steel tube, and the tube separation from the concrete at the joint interface. The portion of the steel tube between the beam flanges acted like a stiffener by mobilizing a concrete compression strut which assists the beam web inside the steel tube in absorbing shear. A compressive force block was created by the beam flanges pressing against the concrete on top of and below the flanges.

Shakir-Khalil, H. and Al-Rawdan, A., 1995, 1996 (sp; m)

Introduction. An experimental study on simple beam-to-column connections of CFT members was presented in several companion papers. A total six series of connections (A, B, C, D, E, and F) were tested, including both circular and square sections for the steel tubes (Shakir-Khalil, 1993a, 1993b, 1994a, 1994b; Shakir-Khalil and Mahmoud, 1995, Shakir-Khalil and Al-Rawdan, 1995, 1996). An analytical study was also performed for the specimens in test series F to predict their failure load and failure pattern.

Experimental Study, Results and Discussions. A total of eight specimens were tested in test series A, B, E, and F, while test series C and D consisted of two specimens. In test series A and C, circular sections were used and for the remaining test series (B, D, E, and F), square sections were utilized. Either finplates (i.e., shear tabs) or tee cleats, which were cut from wide flange sections, were used as the connection elements, with series A to D using finplates, and series E and F using tee cleats. The finplates and tee cleats were fillet welded to the steel tubes and bolted to the beam webs. Hilti nails were provided as shear connectors in the CFTs for some of the specimens in test series A and B to investigate the effect of shear connectors on the connection performance. The test setup consisted of a vertical CFT column having two beams framing in at the mid-height. However, the first group of specimens in test series F were exterior connections that had single beam attached to the columns. The beams were subjected to symmetrical upward shear loading, while an axial load proportional to and in the same direction as the beam loading was also applied at the bottom of the CFT columns. Only the beams of the specimens in the second group of test series F were loaded unsymmetrically. The specimens were laterally braced at mid-height, and the column ends were free to rotate. The nominal length of the columns was 435 in. and their *D/t* ratio varied from 30.00 to 34.78. The ranges for the measured yield strength of the steel tube and the measured compressive strength of the concrete were 43.9-60.2 ksi and 4.25-5.90 ksi, respectively.

In test series A, overall column failure took place at the upper part of the columns. The tube walls at the finplate locations underwent insignificant deformation. No connection failure occurred despite the local yielding of the steel tubes at the lower part of the connections. In this region, the local yielding of the steel tubes was caused by the combined effects of the compressive stresses from the axial load applied to the columns, transverse tensile stresses due to the moment from the beams and residual compressive stresses as a result of the welding process. The longitudinal strain of the steel tubes was found to increase at the same rate for the gauged points located away from the finplates at a distance more than the diameter of the steel tube. This showed that the load transfer between steel and concrete was completed within a distance less than the diameter of the steel tube above and below the finplates. The failure load of the specimens was found to increase with the use of shear connectors at the connection region. Providing higher finplate depth and smaller moment arms for the beam loads also increased the failure load of the specimens. At the end of the tests, no concrete crushing at the connection region was observed and the major damage was the elongation of the bolt holes.

In test series B, the specimens having low eccentricity in the beam loading exhibited column failure in the upper part of the connection. Steel yielding at the top of the column was observed and local buckling took place for some of the specimens. On the other hand, the specimens with large eccentricity in the beam loading underwent large rotations and failed by outward bending of the steel tube at the toe of the finplate. The concrete core prevented a complete yield line failure, as it did not allow the steel tube corners to deform inward and caused a membrane mechanism to develop making the section stifffer. However, the failure load of these specimens was lower than the ones that experienced column failure. The strain distribution above the finplate position showed that beam force was transferred to the composite section within a distance of approximately half of the depth of the square tube. At the end of the tests, the concrete was examined and no crushing was observed.

The performance of the finplate connections for square CFTs of test series B and D was not as good as for the circular CFTs of test series A and C. The connection stiffness of the specimens in test series A and C were found to be approximately 2.7 times those of the specimens in test series B and D. This was due to the flexibility of the rectangular tube walls. In contrast to test series B and D, tee cleats were utilized for the square CFTs of test series E. The tee cleat connections were found to be about 2.4 times stiffer than the finplate connections, comparing the result of test series E to those of test series B. For some of the tee cleat connections, transverse welds were applied along their flanges, which caused an increase in stiffness about 30%. In test series A to E, most of the specimens had a sudden increase in rotation between 88.51 kips-in. and 221.28 kips-in. moments. This was due to the slip occurring at the bolt holes and the outward bending of the steel tube walls.

The first group of specimens in test series F, which were designed as exterior connections, suffered in-plane deformations at the mid-height, since the braces were subjected to high forces close to the failure load of the columns. The strains along the column heights indicated the combined effect of axial compressive force and bending. On the connection side of the columns, the bending effect caused high compressive and tensile strains at the top and bottom of the connection regions, respectively. The maximum strains in the steel tube occurred at 2.95 or 5.91 in. above tee cleat and decreased toward the end of the column. The location of the minimum strains in the steel tube was 2.95 or 5.91 in. below the tee cleat and the strains increased gradually toward the lower end of the steel tube. On the opposite side of the column at 2.95 to 5.91 in. above the connection, the strains were minimum, although they increased toward the upper end of the column. On the opposite side of the column, below the connection region, the strains were the highest at the end of the tee cleat and decreased toward the lower end of the column. Yielding on the steel tube was observed above the tee cleats at the connection side between 204.59 and 260.79 kips axial loads. The local buckling of the steel tube generally occurred 4.72 to 7.09 in. above the tee cleats at the connection side. During the initial stages of loading, the stiffnesses of some of the connections were observed to decrease, which was attributed to slip at the boltholes. It was speculated that the specimens having no reduction in initial stiffness might have bolts that were overtightened. Moreover, it was probable that the connection with lowest initial stiffness had lightly tightened bolts. The experimental failure loads were compared with BS5950 (1985) and BS5400 (1979) design code provisions and the experimental values were conservatively estimated .

The second group of the specimens in test series F were configured like the other test series, with two beams per specimen at the mid-height. However, these specimens were subjected to unsymmetrical beam loading. Close to the failure load, large forces developed on to the lateral braces and this caused in-plane deformations at the midheight and at the ends of the columns. The specimens exhibited similar responses to the first group of specimens in stiffness and slip at the bolt holes caused sudden drop of the initial connection stiffness. For all the specimens in the second group of test series F, overall column failure was observed. The BS5400 (1979) and BS5950 (1985) procedures predicted the experimental load conservatively.

Analytical Study. An analytical study was also performed in order to estimate the behavior of the specimens in the second group of test series F. The failure loads were calculated and the bending moment-axial load interaction diagrams were generated for the column sections. For this purpose, a rectangular stress block approach and analyses using ABAQUS were used. In addition, the connections were also modeled in ABAQUS with 3D finite elements. The beams were not modeled separately and the stem of the tee cleat was extended to represent the beams. The main objective of the numerical model was to investigate the yield line failure of the tee cleats. For all the specimens, the rotation in the yield line failure took place at the level of second bolt from the top of the tee cleat. The experimental results also proved this response when the concrete core was investigated after testing. The analysis results indicted a gap along the yield line between the tee cleat flange and the column. It was observed that after sufficient amount of deformation along the yield line, the yield line failure pattern changed into a stable membrane behavior. The failure loads found by 3D finite element analysis of the connection gave the best correlation with the experimental results by giving 2% to 6% higher failure loads than experimental values.

<u>Kawano, A. and Matsui, C., 1997</u> (fs; m, r)

<u>Introduction</u>. Beam-to-column connections with external diaphragm details might create difficulty when placed near external walls if the diaphragm plates are large. New connection types were designed to overcome this problem. These connections had vertical stiffeners welded to the column face, substituting for the external diaphragm on one side. In this paper, simple tension tests and cyclic cruciform specimen tests were presented, both of which were conducted on various connection details with vertical stiffeners. In addition, the authors described formulations to calculate the strengths of these connections.

<u>Experimental Study, Results, and Discussion.</u> In the tension tests, four types of connection details for square CFT and square tubular columns were tested. Their labels were A, B, C, and D. Type A was a typical external diaphragm detail. Types B and C both had vertical stiffeners. For type B, the centerlines of the column and the girder coincided with each other. However, they did not coincide for type C. In type D, only the vertical stiffeners were used and the external diaphragms were removed. The simple tension specimens had single girder flanges that were attached to the CFT or tubular columns through vertical stiffeners or external diaphragms. Fillet welding was utilized to connect the elements. A total of fifteen specimens were tested with varying column types, connection details and dimensions. The L/D ratio for the steel tubes was 2. The columns were square and had a D/t ratio of 33.3. The girder flanges, diaphragms and stiffeners had a thickness of 0.177 in.. The average measured compressive strength of concrete for the CFT columns was 5.01 ksi. The measured yield strength of the steel was equal to 62.8 ksi for the steel tube and it was equal to 44.4 ksi for the girders, diaphragm plates and stiffeners.

The specimens were subjected to monotonic tension through the flanges until fracture. All of the specimens except for one failed at the predicted locations. Failure in the stiffener was common for the specimens with connection types B, C and D. For two of the specimens with connection type C, early fracture in the flange took place. This fracture initiated at the edge of the flange, which was close to the vertical stiffener weld, and then moved transversely along the whole flange. The fracture was due to the fact that the vertical stiffener length and the connection length were not equal to each other. This caused a discrepancy in flexural stiffness at the sides of the connection. .

Two series of cyclic tests were conducted on cruciform CFT and cruciform hollow tubular specimens. Before testing, the strengths of the specimens were calculated based on the girder capacity, column capacity, and connection capacity, separately. The specimens in series I were designed to have weak connections, while the specimens in series II were designed to have either weak columns or weak girders. The specimens had connection types A, B, or C. The same kinds of materials as were used for the simple tension test specimens were utilized to manufacture the cruciform specimens . The columns were simply supported and subjected to constant axial load throughout the tests. Cyclic shear force was applied at the girder ends until fracture.

For the first test series, the experimental capacity of the specimens showed good correlation with the predicted values. However, for the tubular specimen with connection type C and a weak stiffener, the experimental and theoretical results did not match. Panel zone buckling took place in the connection region of this specimen due to eccentricity of the girder, which caused high shear at connection region. For the specimens in series II having weak columns, the experimental yield strength values were equal to or greater than the theoretical yield strength values. However, the experimental ultimate strength values were less than the theoretical values due to the low yield strength of steel used for the vertical stiffeners. In the case of the specimens in series II having weak girders, the theoretical estimates gave conservative strength values. The horizontal load-deflection diagrams showed that the frames with vertical stiffener connections generally experienced stable hysteresis loops similar to the ones obtained for the specimens with outside diaphragm connections. For some of the specimens, the load-deflection curve behavior improved when the vertical stiffener was increased in size.

Analytical Study. The yield strength of the connections with the vertical stiffener (P_y) was taken as the minimum of the yield strengths of the connections of type A and type D. For their ultimate strength (P_u), the same method was proposed. The formulations from the AIJ (1990) design code provisions were presented to calculate the yield strength (P_{dy}) and ultimate strength (P_{du}) of connection type A. On the other hand, the following equations were proposed for the connection type D to calculate its yield strength (P_{sy}) and ultimate strength (P_{su}) (all stress values are in MPa):

$$P_{sy} = 2h_s t_s \sigma_{sy} + 2(\beta t_c + t_d) t_c \sigma_{cy}$$

$$P_{su} = 2h_s t_s \sigma_{su} + 2(\beta t_c + t_d) t_c \sigma_{cu}$$

The first and second terms in the equations above represent the force transferred to the column through the stiffener and girder flange, respectively. The empirical factor, β, which is related to the effective width of the column web, was equal to 4 in the case of connections to CFT columns and it was equal to 3 in the case of connections to hollow tubular columns. In addition, a different set of equations was proposed for connection type C to calculate its yield and ultimate strengths. These equations accounted for the unbalanced transfer of force apparent in this connection type:

$$P_y = \min\left[P_{ecsy}, P_{ecdy}\right]$$

$$P_{ecsy} = \frac{D}{2b} P_{sy}, \; P_{ecdy} = \frac{D}{2a} P_{dy}$$

$$P_u = \min\left[P_{ecsu}, P_{ecdu}\right]$$

$$P_{ecsu} = \frac{D_c}{2b} P_{su}, \; P_{ecdu} = \frac{D_c}{2a} P_{du}$$

where P_{sy} and P_{su} are the yield strength and ultimate strength of the vertical stiffener connection, P_{dy} and P_{du} are the yield strength and ultimate strength of the external diaphragm connection, a and b are distances from the beam center line to the edges of the steel tube.

According to the test results, good correlation was achieved between the experimental and theoretical strength values. Moreover, the failure in the stiffener for specimens with connection types B, C and D proved the accuracy of the given equations.

The authors concluded by presenting two conditions for the proposed equations to be used safely: the thickness of the vertical stiffener should be at least as thick as the girder flange; and the *D/t* ratio for the steel tube should be less than 35.

Schneider, S. P., Alostaz, M. Y., 1998 (fs; r)

Introduction. An analytical and a subsequent experimental study were conducted on steel girder to circular CFT beam-column exterior moment connections (Alostaz and Schneider, 1996; Schneider and Alostaz, 1998). Three-dimensional finite element analysis of various connection details was performed for monotonic flexural loading. According to the analysis results, six large-scale specimens were designed and then tested under cyclic loading. The authors compared the performance of each detail and highlighted the differences and similarities observed in their inelastic cyclic responses.

Analytical Study. A total number of 6 connection types (I, II, III, IV, V, and VI) were modeled in ABAQUS. The *D/t* ratios of the columns were 40, 53.5 and 80. The axial load over nominal axial load strength ratio was ranging between 0% and 36%. The third parameter, which was moment over shear ratio, varied from 0.0197 to 0.299. In addition, some connections were also modeled and analyzed as interior connections. The length of the columns were 144 in.. The yield strength of the steel was 45.98 ksi and 35.97 ksi for the steel tube and wide flange girder, respectively. The compressive strength of concrete was 5 ksi. The columns were pinned at the bottom and had a roller support at the top. The girders were subjected to monotonic shear force while a constant axial load was applied on to the columns. The steel tube and girders were modeled with shell elements and brick elements were used to model the concrete core. For steel, a bilinear constitutive stress-strain relationship with 2% strain hardening was selected both in tension and compression. Von Mises' yield criterion and Prandtl-Reuss flow rule were utilized for the yield surface and plastic deformations, respectively. The 3D concrete model in ABAQUS, which was derived for low confinement, was used. The bond between steel and concrete were simulated with a thin layer of brick elements having low strength and stiffness properties.

Connection type I consisted of a flared connection stub welded to the steel tube and bolted to the girder. High local deformation of the steel tube was observed close to the tension flange of the girder. This caused the stiffness of the connection to decrease. The plastic moment strength of the girder was not attained. Decreasing the wall thickness from 53.5 to 40 did not improve the initial stiffness and it resulted little gain in strength. When the connection was analyzed as an interior joint, the initial stiffness remained almost the same and little reduction in strength was obtained. Despite higher panel shear force in the interior connection, no concrete crushing was observed. The experimental test of the same connection type had been completed at the time of the analytical study and it was found that the experimental and analytical results showed good correlation.

Connection type II had the exterior diaphragm detail. Two different kinds of this connection were analyzed. The first one had the minimum amount of diaphragm plate and the diaphragm plate of the second one had a larger angle at the flange tips moving the girder away from the connection. The first connection had similar response to connection type I. However, the second connection exhibited superior strength and stiffness properties. Its strength reached the plastic strength of the girder and the deformation of the diaphragm plate was very little close to the steel tube face.

Connection type III consisted of weldable deformed bars inserted into the CFT column and welded to the girder flanges. Although the stiffness was about half of the ideal rigid connection, the strength was close to the plastic strength of the girder. The steel wall distortions due to flange forces were not significant. Two modified connection details were also analyzed for this connection type. The first one was the internal connection and the second one had a shear tab between girder and steel tube to eliminate the need for flange weld. All the connections had almost the same strength and the unmodified type had the largest stiffness. Instability was observed for the connection with no flange weld and buckling of the deformed bars occurred at the gap between the flanges and the column. Thus this connection had the lowest stiffness.

Connection type IV had shear studs inside the steel tube at the same locations with the girder flanges. The stiffness of this connection was half of the stiffness of the ideal rigid connection and it behaved linearly up to 93% of the plastic strength of the girder. The shear studs farthest from the girder web found to have insignificant contribution to the tensile capacity of the flanges. As an alternative connection detail, the same connection was analyzed with less number of shear studs and the results showed that the inelastic response remained almost the same. Another parameter investigated for this connection was the moment-to-shear ratio and the analysis made for different values of this ratio. It was found that the behavior was more ductile in the case of low moment-to-shear ratio.

In connection type V, the web plate of the girder was extended into the CFT column with shear studs attached on it. This connection exhibited the least strength of the connections with a value of about 72% of the plastic strength of the girder. Its stiffness was about 28% of the ideal rigid connection. For small moment-to-shear ratio even lower strengths were obtained. The connection was also analyzed without shear studs on the web plate and became identical to the connection type I excluding the continuing web plate. In this case better strength and stiffness properties were obtained than connection type I.

For connection type VI, the steel girder was extended into the steel CFT column. The strength and stiffness of this connection type was close to those of the ideal rigid connection. There was no significant demand on the steel tube and slight increase in stiffness was obtained when D/t ratio decreased.

Experimental Study, Results, and Discussions. The main parameter of the experiments was the connection detail. The labels for the tested connections similar to the analytical study were I, IA, II, III, VI, and VII. The test setup consisted of a W14×38 shape girder having 108 in. length and a circular CFT column having 92 in. length. The girder was connected to the mid-height of the column. For the CFT columns, the *D/t* and *L/D* ratios were 56.6 and 6.6, respectively. The nominal yield stress of the steel tube was 42.1 ksi and nominal compressive strength of concrete was 5 ksi. The supports of the columns were pinned at the bottom and roller at the top. A cyclic shear force was applied at the tip of the girder and displacement controlled loading was utilized. The axial load acting on the columns was kept constant throughout the tests. The degradation of the elastic stiffness of the connection was also monitored by means of elastic cycles in the tests.

In connection type I, the flange tip of the connection-stub fracture was followed by the fracturing of the connection-stub weld. The failure of the specimen, at which the flaring connection-stub flange fractured completely, took place at 3.75% rotation. Only one flange fractured and the other flange remained undamaged. Thus, the hysteresis loop was not stable in the positive loading direction. However, it was stable in the negative loading direction while the connection was able to reach to the plastic moment strength of the girder. The concrete in the connection region was examined and observed to be uncrushed. Among the connection details, type I had the smallest shear and bending strength. The shear capacity deteriorated rapidly after the flexural resistance was lost due to fracture of the connection-stub flange. In addition, the intensive damage occurred on the tube wall increased the flexibility. Thus, the authors concluded that it was not appropriate to use type I connection in moment-resisting frames located in seismic regions.

Connection type IA was similar to type I. However, in type IA, the web of the connection-stub was extended into the steel tube. This resulted a better cyclic response. Strain hardening response was observed until 1.25% rotation. After this level of deformation, degradation in strength was observed due to weld fracture and tearing of the steel tube. Steel tube fractured at the tip of the flanges when the flanges were subjected to tension. The fracture grew as the loading continued and this resulted a 20% decrease in strength. Local buckling occurred in the connection stub flanges at 2.25% rotation and then a web fracture was observed in the girder causing a 16% decrease in flexural strength. At the end of the test, the free side of the steel tube remained undamaged. Rapid decay in moment strength was also observed in connection type IA. Identical to the type I, there was high flexural deformation demand on the web after the flange was fractured. Therefore, for fully restraint connections, it was required to prevent fracture of the flange. The continued web might have some rotational restraint in pinned-ended connections. Yet, it should exist for the required shear strength.

In connection type II, stain hardening behavior was observed up to 1.0% rotation. Due to high stress concentrations, fracture took place at the diaphragm plate acting in tension and the moment strength decreased. At the later stages of loading, fractures in the girder flange propagated into the steel tube. This caused the flexural strength to loose half of the plastic moment strength of the girder. At high deformation amplitudes, the groove weld between the girder flange and the diaphragm plate fractured. This was followed by the fracture of the steel wall along the depth of the connection-stub web. At 2.25% rotation, local buckling of one of the diaphragm plate was observed when the girder flange was in tension. The failure of the connection took place with excessive damage at 4.5% rotation. No concrete crushing was observed in the connection region. The external diaphragm used in connection type II improved the cyclic response. However, the degradation of moment strength was still rapid. Although the elastic stiffness was equivalent to that of an ideal rigid connection at initial stages of loading, rapid decay in stiffness was apparent at high amplitude displacement levels. In addition, extensive tearing at the re-entrant corners between the diaphragm and the girder flange imposed high deformation demand on the steel tube.

Connection type III had four 0.63 in. deformed bars welded to the both flanges of the girder and this connection showed a very stable hysteresis loop. At 3.75% rotation, steel tube tearing was observed between the holes opened to insert the deformed bars. However, it did not affect the response of the connection. At the same deformation level, local flange buckling took place beyond the region covered by the steel bars. The connection maintained its stable response up to 5.0% rotation and the connection failed by the rupture of the deformed bars. The deformed bars welded to the flanges resulted a desirable cyclic behavior for connection type III. It was appropriate to use this connection type in moderately seismic regions. The strength was the largest among the other type of connection details with a maximum value of 1.5 times the plastic moment strength of the girder. The elastic stiffness varied between 98% and 106% of an ideal rigid connection. No decay in stiffness was observed throughout the test. The detailing of the deformed bars was critical and the location of first weld on the bars should be adjusted to assure proper inelastic response. The use of small diameter bars was more practical due to the requirements for weld quality and spacing.

Connection type VI was different from the connection type with the same label in the analytical study. The flanges of the connection-stub were extended into the CFT column and welded to the steel tube. During the test, the bond between the concrete and the continuity plate was not strong and this deteriorated the hysteresis loop. At 0.5% rotation fracture of the weld between the continuity plate and the steel tube took place. Initially, this did not cause any reduction in flexural strength due to interlocking between the welded parts. However, the strength was not maintained in the following cycles. At later stages of loading, steel tube tearing also occurred. This resulted insignificant reduction in strength but rapid stiffness degradation was observed. For connection type VI, the continuing flange improved the behavior compared to type I. The elastic stiffness underwent continuous degradation during the test. The slip of the connection-stub flange was critical as it imposed high deformation levels on the steel tube wall. For this connection to be used in seismic regions, this slip should be prevented. For this purpose, it was possible to use deformed bars placed on the continuing flanges and transferring the flange force to the concrete core. In addition, plates could be attached to each flange at both sides of the column and transferring the flange force directly to the CFT column.

In connection type VII, the girder continued into the CFT column entirely and welded to the steel tube. The specimen maintained its strength up to a rotation level of 4%. Local flange buckling of the connection-stub took place at about 7.5% rotation. At the same deformation amplitude, the weld between the steel tube and the connection-stub fractured. This resulted higher shear force to act on to the weld at the other side of the column and the weld in that region fractured. Web buckling also occurred and then deterioration in cyclic response was observed. At later stages of loading, fracture in the connection-stub flange also occurred. The flexural strength of this connection was 1.25 times the plastic bending strength of the girder. The concrete in the connection region was examined and there was no crushing. Type VII connection had identical response to an ideal rigid connection. Both the flexural strength and elastic stiffness remained stable throughout the test. This connection detail could be used in highly seismic regions.

<u>France, J. E. et al., 1999a, b, c</u> (fs, sp; m, r)

<u>Introduction</u>. Three series of experiments on beam-to-column connections of HT and CFT columns were presented in a series of papers (France et al. 1996, 1999a, 1999b, 1999c). The girders were connected to the steel tube wall by the flow-drill technique. Both moment connections and simple connections were tested. The stiffness and strength properties of the connections were investigated and the effect of concrete filling on the connection performance was discussed.

<u>Experimental Study, Results and Discussions</u>. With the advent of the flow-drill technique, it was possible to use standard bolts for thin-walled tubular columns. In this technique, the tube wall is heated and displaced into the column. It then becomes thick enough to form threads for bolting. The flow-drill technique is advantageous for thicknesses up to 0.492 in..

The first test series included simple connections to hollow square tubes through flow-drill technique. The main parameters of the tests were end plate type, beam size, tube thickness, and bolt spacing. Some specimens were tested under axial load. The test setup consisted of a simply supported and horizontally placed column with a vertical wide flange beam at the middle. A shear force was applied cyclically at the top of the beam excluding the specimens having partial depth end plates. These specimens were subjected to monotonic shear loading at the beam tip . Either partial end plates or flush end plates were welded to the beam end and then bolted to the steel tube face. Partial end plate connections exhibited small moment capacity when compared to the plastic moment of the beam and they were classified as pinned connections. Flush end plate connections could be regarded as semi-rigid joints rather than simple joints as they exhibited significant moment capacities. The specimens with variable end plate thicknesses did not show any significant difference in behavior. However, larger wall thickness and higher beam depth caused an increase both in strength and stiffness of the connection. When the bolt spacing was increased, the strength and stiffness of the specimens was found to improve. The level of axial force influenced the response by decreasing the post-yield stiffnesses of the specimens.

In the second test series, moment connections to hollow square tubes using the flow-drill technique were tested. The test setup consisted of a horizontally placed column having a vertical girder at the middle. The end plate and girder sizes were selected to ensure that the steel tube face would govern the strength and stiffness response of the connections. The main parameters of the test were steel tube thickness and the grade of the steel tube. Flush end plates and extended end plates were used between the girders and steel tubes. The girder ends were subjected to monotonically applied shear force. The extended end plate connections underwent large rotations mainly due to the deformation of the steel tube wall, which became less as the thickness of the steel tube increased. After the bolts were removed, it was observed that the end plates were undamaged and the bolt threads deformed considerably. The specimens either failed by bolt pull-out or failed at a load level close to that. The compression zone of the specimens experienced yielding and outward web buckling. The amount of web buckling was larger for smaller tube thickness. In general, the extended end plate specimens with large tube thickness had higher strength but less ductility compared to the ones with small tube thickness. When the response of the extended end plates having different steel grades were compared, it was found that a 30% increase in steel yield strength caused an approximately 27% improvement in moment capacity at a 0.015 radian rotation level, beyond which the connection had signs of extensive yielding. For the specimens with flush end plates, the deformation was mainly at the column face. These specimens exhibited ductile response but less moment strength than the corresponding extended end plate specimens. The difference in strength at 0.02 rad. rotations was 63% and 70% for the specimens with 0.315 in. and 0.394 in. wall thicknesses, respectively. For the specimens with flush end plates, the moment strength improved approximately 42% when the thickness increased from 0.315 in. to 0.394 in.

In the third series of the experimental study, simple and fully-restrained connections of square CFTs manufactured using the flow-drill technique were tested. The same test setup was utilized as in the previous tests. The specimens were subjected to monotonic shear load applied at the cantilever girder-end. The *D/t* ratios of the columns varied between 20 and 31.7. The range for the measured yield strength of the steel was 46.1 to 61.9 ksi and the compressive strength of concrete was either 6.29 ksi or 7.32 ksi. For the flush end plate specimens, which were classified as simple connections, ordinary bolts were inserted before the casting of concrete. The ordinary bolts were then removed and replaced with strain-gauged bolts after the concrete had hardened. For the extended end plate specimens, ordinary bolts were inserted before pouring the concrete and these bolts were used throughout the test. The flush end plate specimens had different size beams as the main parameter. It was found that the specimens with larger size beams had more strength and stiffness. For each of the flush end plate specimens, yielding was observed across the whole compression flange of the beams. However, in the case of the equivalent hollow steel tube connections tested earlier, yielding had localized at the tips of the compression flanges. In addition, it was observed that the axis of rotation was at the compression flanges since the concrete core prevented the web of the columns from local buckling, and this caused an increase in initial stiffness. At the end of the tests, it was found that yielding at the column flanges had not extended to the webs. For some of the flush end plates, a sudden change in stiffness was observed in the transition region from elastic response to the nonlinear response while the equivalent hollow tubes achieved a smooth transition. This was attributed to crushing of the concrete in the steel tube. When the concrete was investigated after the test, it was found that crushing took place at some of the bolt regions as a result of bolt rotation. The two extended end plate specimens had different yield strength values for the steel tube. At the initial stages of the loading of the first specimen, a sudden drop in moment capacity at a rotation value of 0.0035 rad. was observed, and this was attributed to bolt slip. The loading then continued up to the failure of the outermost bolts at the tension region. When the specimen was examined after the test, it was found that there was no damage at the end plates. Yielding occurred at the girder webs and at the girder flanges in the compression zone. The steel tube did not deform at the compression region, while extensive deformation took place at the tension region. When the response was compared to an equivalent unfilled specimen, improvements in strength and stiffness with reduced ductility were observed. The second extended end-plate specimen had higher strength steel than the first one. It exhibited greater moment capacity and initial stiffness. The end plate underwent some slight deformation and extensive yielding in compression flange and web of the girder. The column face deformation was similar to the first specimen.

<u>Cheng, C. et al., 2000</u> (fs; r)

<u>Introduction</u>. Beam-to-circular CFT column connections were tested under cyclic loading in this work. The authors investigated the effect of concrete infill and *D/t* ratio on the seismic performance of the connections. The research program included four external diaphragm and two through diaphragm details. In this paper, test results of external diaphragm specimens were presented.

<u>Experimental Study, Results, and Discussion</u>. Each specimen consisted of two girders connected to the mid-height of a circular column. Three specimens had CFT columns and the other three had tubular columns. Only one specimen, which had a CFT column with external diaphragm detail, was designed for strong column-weak beam condition and the others were designed for weak column- strong beam condition. The *L/D* ratio for all of the columns was 6.5. The *D/t* ratio of the steel tubes was either 40 or 66.7 for 0.394 and 0.236 in. wall thicknesses, respectively. The yield strength of steel tube was 56.86 ksi for 0.236 in. thick tubes and 45.5 ksi for 0.394 in. thick tubes. The compressive strength of concrete was either 3.771 or 3.916 ksi. The yield strengths of the flange and web of the steel beam were 44.7 and 47.6 ksi, respectively. The columns were subjected to constant axial load throughout the test. Cyclic shear forces were applied at the girder-ends and displacement controlled loading was utilized.

Among the external diaphragm connections, the ones designed for weak column-strong beam condition experienced local buckling of the column wall. In addition, some inelastic deformation at the panel zone was observed. In the case of the strong column-weak beam connection, plastic deformation initiated at the girder-flange. The plastic deformation proceeded with local buckling of the column wall and inelastic deformation at the panel zone. The failure occurred at the weld between the girder-flange and external diaphragm. This connection exhibited a stable hysteresis loop and it had increasing stiffness after yield.

For all of the specimens, panel zone deformations did not contribute to the total girder-end deformations while the contributions of column and beam deformations were 60% and 40%, respectively. The concrete infill improved the cyclic behavior and CFT specimens had better strength, post-yield stiffness and ductility properties. As the tube wall thickness reduced, the gain in strength due to concrete infill increased. However, the specimens became more susceptible to local buckling and panel zone deformation.

<u>Fujimoto, T. et al., 2000</u> (pz; r)

<u>Introduction.</u> An experimental study on CFT beam-to-column connections was presented. Half-scale internal and external connections were tested under reversed cyclic loading. The performance of the connections manufactured from high strength materials was the main focus of the research. In addition, the connection details and loading conditions varied for the specimens to study their effects. The experimental results were also compared with AIJ-SRC (1997) design code provisions.

<u>Experimental Study, Results, and Discussion.</u> Seven square and four circular CFT beam-to-column connections were tested. The specimens were made up of CFT columns and wide flange steel girders. The steel tubes were manufactured from two channel sections. One connection detail was 3D and the others were 2D. Either through diaphragm or outer diaphragm

connection details were used. The thickness of the panel zone webs and flanges were less than that of the webs and flanges of the CFT columns away from the panel zone. This was to encourage shear yielding would take place in the connection rather than at the column. The range of the *D/t* ratio for the square and circular panel zones were 51 to 55 and 58 to 61, respectively. For the CFT columns, the *D/t* ranges were 17 to 21 for square shapes and 19 to 31 for circular shapes. The measured yield strength of the steel varied between 63.2 ksi and 110.7 ksi. The measured compressive concrete strength ranged from 7.89 ksi to 15.95 ksi. One exterior and all of the interior connections were subjected to constant axial load. Variable axial load was applied to the remaining exterior connections. The specimens were loaded by antisymmetric cyclic shear forces at the girder ends, and the 3D connection detail was loaded at a 45 degree angle relative to the CFT cross section principal axes.

The panel zone shear strengths calculated by the AIJ-SRC (1997) procedures were less than the experimental shear strengths of the CFT specimens. The theoretical yield strength values for the CFT columns and steel girders were much larger than the experimental shear strengths of the connections. The stiffness of the connections showed good agreement with the calculated values from AIJ-SRC (1997) design provisions in the elastic range. The square interior connection with an outside diaphragm and the square exterior connection that was subjected to variable axial loading exhibited larger decay in stiffness compared to the others. This might be due to possible local yielding and deformation at the connection region. The ratio of the experimental ultimate strength versus the ultimate theoretical strength from AIJ-SRC (1997) for the circular and square specimens excluding the 3D connection ranged between 1.66-1.84 and 1.31-1.62, respectively. Consequently, the effect of confinement should be accounted for in circular connections due to their high experimental over theoretical strength ratios. The strength of the 3D specimen was examined separately for strong axis flexure and about an axis that was 45 degrees to the principal axes of the cross section. It was found to be stronger in the latter direction. All of the specimens exhibited ductile behavior and the rotation capacities ranged from 0.028 to 0.041. For the 3D specimen, the loading direction did not affect its deformation capacity.

<u>Fukumoto, T. and Morita, K., 2000</u> (no tests)

<u>Introduction</u>. An analytical study was carried out on panel zone behavior of steel beam-to-square CFT column connections. The authors proposed a nonlinear stress-strain model to estimate the elasto-plastic response of the panel zone. The analytical findings were compared with the experimental results.

<u>Analytical Study</u>. For modeling of the panel zone, the load-deformation relations of the steel tube and concrete were superimposed. The steel tube had a tri-linear stress-strain relation defined by a yield point, a plastic stiffness degradation point, and an ultimate strength point. The concrete stress strain relation was also tri-linear and it consisted of a cracking strength point, an ultimate strength point, and strain point at which the ultimate strength is retained. An arch mechanism in the connection was introduced to calculate the ultimate strength of the concrete. The arch mechanism had a main arch compression strut and a confining arch compression strut. The ultimate strength of the concrete was obtained by summing the ultimate the strengths of the two compression struts.

The proposed analytical model for the joint panel was applied to a wide range of specimens from the literature. The specimens included welded connections having either internal or external diaphragms. Good correlation was obtained with the experimental results up to a shear deformation level of 0.04 radians. Beyond that level, the analytical curves remained below the experimental ones.

Peng, S. et al., 2000 (fs; r)

Introduction. An experimental study on beam-to-column connections of square CFT systems was presented by (Ricles et al., 1997; Peng et al., 2000). Full-scale cruciform specimens were tested under cyclic loading. The connection detailing and load transfer mechanism in the connection region were the focus of the research. Three types of connection details were utilized and their strength, stiffness, and ductility properties were investigated.

Experimental Study, Results, and Discussions. The test setup consisted of a square CFT column and two W24×62 steel girders. The column was pinned at the bottom and free to rotate and translate in the plane of the girders at the top. The girders were connected to the mid-height of the column. The measured yield strength of the flange and web of the steel girders were 43.2 and 49.6 ksi, respectively. The measured yield strength of the steel tubes was 55.0 ksi. The *L/D* ratio was 9 and the *D/t* ratio was 32.5. The measured compressive strength of the concrete was 8.96 ksi. The connection types used for the specimens included interior diaphragm, extended-tee, and split-tee moment connection details. The latter two details were designed for the plastic hinge to occur in the girders. Among the remaining specimens, two of them (C1R2 and C2R) were designed for panel zone yielding while the others were again proportioned for a weak-beam-strong column condition. The specimens were tested under constant axial load and cyclic shear force, both of which were applied at the top of the column.

Four specimens (C1, C1R, C2, C1R2 and C2R) were manufactured with the interior diaphragm detail. The specimens with R in their labels were retrofitted from the previously tested equivalent connections. Full-penetration groove welds around the perimeter of the CFT were used to connect the diaphragm to the steel tube. For specimens C2 and C2R, the weld along one of the webs of the tube was omitted. Full penetration groove welds were also used for welding the girder flanges to the steel tube; these welds were then strengthened with a fillet weld. In specimens C1R2 and C2R, tapered plates were attached to the flanges of the steel girders. Short cover-plates on the flanges were used for specimen C1. In the test, an early fracture occurred at the toe of the cover plate weld of specimen C1. At the end of the testing of specimen C1, extensive girder flange yielding and insignificant shear yielding of the panel zone were observed. The location of fracture for specimens C1R and C2 was the toe of the weld connecting the flange of the girder to the steel tube. Specimen C2 was overloaded unintentionally and the loading was kept at that level to provide symmetric cycles. Fracture occurred when the specimen C2 was overloaded in the other direction. Specimens C1R2 and C2R exhibited extensive shear yielding mainly in the panel zone, while the other specimens with interior diaphragm underwent inelastic deformation in the girders. For specimens C1R2 and C2R, the formation of diagonal shear cracks in the concrete in the panel zone region was also observed after the test and shear buckling of the steel tube web occurred in the panel zone as the story drift got larger. However, the load-deformation response maintained its stable trend for specimens C1R2 and C2R. These specimens achieved shear strengths of approximately 1.76 and 1.49 times the nominal shear capacity of the steel tube, respectively. The groove weld that was omitted in specimen C2R caused the shear strength to decrease.

Extended-tee connection details were utilized for the specimens with labels C3 and C3R. These connections had external diaphragms made from ST7.5×12 structural tees and welded to the column at the corner as well as to the girder flange. Specimen C3R was manufactured by retrofitting of specimen C3 after testing. The only difference between the two specimens was the tapered plates of specimen C3R welded to the flanges of the girders. Specimen C3 experienced a fracture in the tension flange of the girder during the test and extensive yielding of the flange took place. Shear yielding of the panel zone also occurred for specimen C3, although it was not significant. While testing specimen C3R, plastic hinges occurred in the girders and yielding of extended-tee flanges took place. In addition, the panel zone of the specimen C3R also yielded as the tension flange force transferred directly to the steel tube through the extended tees.

The split-tee specimens were labeled as C4, C5, C6 and C7. Post-tensioned through-bolts were utilized to connect the split-tees to the CFT columns. The bolts of all specimens were post-tensioned as per the AISC LRFD (1993) Specification except for specimen C7, whose bolts were post-tensioned to the snug-tighten condition of 40% of the pre-tension level specified in the AISC LRFD (1993) provisions. Thus it was possible to investigate how the amount of bolt pre-tensioning force affected the connection behavior. For specimens C4 and C5, the stem of the tee was connected to the flanges of the girders by bolting, and washer plates were used to prevent flange local buckling along the bolt line. Welding was utilized for connecting the stem of the tee to the girders for specimens C6 and C7. All of the split-tee specimens experienced intensive yielding and local buckling in the web and flanges of the steel girders beyond the connection region.

The extended-tee and split-tee specimens exhibited maximum moment values of 1.23 to 1.56 times the plastic moment of the girders. This overstrength was attributed largely to strain hardening of the girders. The moment capacities started to degrade following local flange buckling of the girders. All split-tee specimens had deformation capacities satisfying the FEMA (1997) requirement of 0.03 rad of plastic rotation. For specimen C4, hole elongation was noted, which caused pinching in the moment-rotation response. The specimens with washer plates and welded tee stems had higher resistance to hole elongation. Specimen C7 also experienced an increase in the bolt force due to the moment acting from the connection on to the bolts; however, no increase in bolt force occurred for the other split-tee specimens as a result of prying action.

From the test results of specimens C1, C3, and C4, it was observed that the initial lateral stiffness (EI) of the columns were well predicted by the AISC LRFD (1993) modified CFT column stiffness and uncracked transformed stiffness. However, for the specimen C3R, its lateral stiffness was closer to the lateral stiffness of the steel tube alone. When the lateral deformations increased, it was found that the lateral stiffness of the column degraded due to concrete cracking and loss of bond between steel and concrete. Among the specimens, the ones with internal diaphragms had a smaller lateral elastic stiffness since the external diaphragms and split tees contributed to the stiffness of the tested assemblies more than internal diaphragms achieved.

3. BOND TESTS

Morishita, Y. et al., 1979a, b (c; po, m)

Introduction. Two series of experiments were conducted to investigate the bond stress between steel and concrete in CFTs. In the first series, only circular columns were tested, while the second series covered both square and octagonal columns. The changes in bond strength due to the variations in cross section shape and the compressive strength of concrete were examined.

Experimental Study, Results and Discussions. The specimens were tested under monotonic axial compression, which was applied to the steel tube alone at the top of the CFT. However, both the steel and concrete was supported simultaneously at the bottom. The *D/t* ratios of the specimens ranged between 34.9 and 46.9. The measured yield strength of steel was either 36.6 or 37.0 ksi and the measured compressive strength of concrete was ranging from 2.73 to 4.85 ksi. The steel tubes were annealed so that they were free from residual stresses.

Strain compatibility between the steel and concrete along the whole column length was observed when small strain values were applied at the top of the steel tubes. However, as the strain level increased, bond stresses and some slip started to develop between the steel and concrete. Thus, the strains became the highest at the top of the steel tube and decreased gradually toward the bottom. The strain compatibility was observed only at the bottom region, which indicated that the axial force transfer from the steel tube to the concrete was completed at the bottom region. For a strain level at the top of the CFT of 5×10^{-4}, no strain compatibility between the steel and concrete was observed. The circular specimens with high strength concrete, as compared to those with low strength concrete, had lower strain values at the limit of exhibiting strain compatibility along the entire length of the column. This was attributed to the greater modulus of elasticity of high strength concrete. Moreover, the bond strength was smaller when high strength concrete was used for circular specimens. This was due to the shrinkage of high strength concrete, which was larger than that of low strength concrete. In addition, it was also due to the fact that Poisson's ratio of steel was greater than that of concrete in the elastic region. The same trends for the strain distribution along the length of the column were also observed in the case of square and octagonal CFT specimens as were observed in circular CFTs . However, in these specimens, high strength concrete was observed to have no effect on the bond strength and on the strain values at the limit of strain compatibility along the entire length and

Analytical Study. The following formulation was presented for the average bond stress (f_{mb}) :

$$f_{mb} = (\frac{P}{A_s} - \sigma_{sl}) \times t / l_a$$

where the first term in the parenthesis represents the normal stress of the steel tube at the top of the column, while the second term (σ_{sl}) is the normal stress of the steel tube at the point where strain compatibility between steel and concrete occurred. The length from the top of the column to the point of strain compatibility is represented by l_a.

The formulation for the average nominal slip (d_s) was given as:

$$d_s = \frac{1}{2} d_{so} = \frac{1}{2} \left(\int_0^{l_o} \varepsilon_s dz - \int_0^{l_o} \varepsilon_c dz \right)$$

where d_{so} is the nominal slip at the top of the column, l_0 is the length of the column, and dz stands for the infinitesimal column length.

Using the equations presented above, the range of bond strength for the circular specimens was determined to be 28.5 to 56.9 psi, and it was found to be 21.3 to 42.7 psi for the square and octagonal specimens. The bond stresses were found to decrease at the initial stages of slip, after which they were observed to remain constant with increasing slip.

Tomii, M. et al., 1980a, b (c; po, m)

Introduction. Bond tests were conducted on circular, square, and octagonal CFT columns and the methods to improve the bond strength between steel and concrete along the interface were investigated. In addition, the authors presented formulations to estimate the bond strength and slip.

Experimental Study, Results, and Discussions. The steel and concrete of the CFTs was supported simultaneously along the bottom. The axial load was applied at the top on to the steel tube alone, and the steel tubes were completely filled with concrete. Two methods were discussed to improve the bond strength between steel and concrete. In the first method, expansive concrete was used to fill the steel tubes. In the case of second method, steel tubes with checkered inside walls were provided. The checkered and smooth steel tubes were filled with either expansive or ordinary concrete. The *D/t* ratio was 46.9 and the *L/D* ratio was 4.9. The measured compressive strength of concrete ranged between 2.28 and 4.85 ksi. The measured yield strength of steel tube was 36.6 ksi and 36.3 ksi for the smooth and checkered tubes, respectively. All the tubes were annealed before testing.

The longitudinal strains at different locations of the steel tube were measured. When the strain distribution along the length of the column was plotted, it was observed that as the strain at the top of the steel tube increased, the strain distribution became nonparellel to the column axis. This was due to the transfer of load from the steel tube to the concrete core and the formation of bond stresses between the steel and concrete. When the strain value at the top reached 9×10^{-4}, it was not possible to see the continuity of strains between concrete and steel along the length of the column except for the circular and octagonal specimens having both expansive concrete and a checkered steel tube inner surface. Thus, the bond strength for these specimens was the largest.

The circular specimens with ordinary concrete and a smooth internal tube surface had average bond strengths of 28.5 to 56.9 psi, which were lower than the bond strengths for round steel reinforcement in concrete. The bond strengths were found to decrease as the concrete strength got higher. These bond characteristics were attributed to the facts that shrinkage of high strength concrete was larger, and steel had a larger Poisson ratio in the elastic range than that of concrete.

The average bond strength for square and octagonal specimens with ordinary concrete and smooth internal tube surfaces ranged from 14.2 to 28.5 psi, which were half of the average bond strength values for the smooth circular specimens with ordinary concrete. The average bond strength for the square and octogonal columns remained constant as average slip increased.

When the columns were filled with expansive concrete, the same type of bond response was obtained for all of the specimens. The average bond strength decreased as the average slip increased. The bond strength of expansive concrete was found be greater than ordinary concrete initially, while the same strength was attained as the loading proceeded. For the squarespecimens, the concrete strength did not have any significant effect on bond strength. However, in the case of circular specimens, the bond strength was found to be larger when the concrete strength was high. The average bond strength of the square specimens was lower than circular ones when expanive concrete was utilized.

The columns with checkered surface had greater bond strength than the ones having a smooth surface. For some of the specimens, it was observed that average bond strength improved gradually as slip increased.

Analytical Study. The same equations discussed in Morishita et al. (1979a, b) for f_{mb} and d_s were proposed again in this work. In addition, the formulation to calculate the average bond stress when the strain compatibility between steel and concrete is not satisfied along the column length was given as:

$$f_b = (\frac{P}{A_s} - \sigma_{sb}) \times t / l_0$$

In the above formulation, the longitudinal stress of steel at the bottom (σ_{sb}) was calculated approximately, as it was not measured. For this purpose, the longitudinal stress at the lower most measured point and the bond stress distribution between this point and bottom of the specimen were used.

For circular columns, the following equation was presented to calculate the required column length to transfer axial load at the beam-to-column connection from steel to concrete.

$$l_a = (P / A_s - \sigma_{sl}) \times t / f_b$$

Virdi, P. J. and Dowling, K. S., 1980 (c; po, m)

Introduction. A study of concrete-filled steel tubes was undertaken to investigate the ultimate bond strength between the concrete core and steel tube in the absence of shear connectors. A number of parameters were studied including the concrete compressive strength, the length of the steel-concrete interface, the *D/t* ratio, the surface roughness, and the effect of manufacturing imperfections, among others. Several push-out tests were conducted, each varying one parameter at a time. Three specimens were tested for each given parameter. The authors gave each parameter a detailed and comprehensive treatment, resulting in an very thorough examination of bond.

Experimental Study, Results, and Discussion. The specimens were tested by loading the concrete with a steel plate 0.5 in. smaller than the inside diameter of the steel tube. The concrete was loaded at a rate of 3.0 kips per minute and allowed 1.5 in. of travel. All specimens showed a uniform load-longitudinal deflection response, with a high initial bond stiffness followed by a marked reduction in stiffness to a relatively flat slope. Only the tubes with smooth inner surfaces did not exhibit some residual strength (i.e., have a positive load-deflection slope) after the concrete core ran out of travel.

The critical bond strength in the tests was taken as the bond strength at a local critical strain at the interface of 0.0035 (in correlation with the ultimate crushing strain of concrete). It was required to establish a value that ignored the frictional effects encountered in the latter stages of the load-deflection curve. This could have been accomplished using the 0.2% offset stress, but the former method was preferred since the offset stress method would require an initial stiffness value, which is difficult to obtain due to initial settlement of the concrete. The bond strength value at 0.0035 was termed the "ultimate bond strength."

The tests showed that the ultimate bond strength was not affected to a large extent by the concrete strength, interface length, tube thickness, or tube diameter. The main factor in the amount of bond is the mechanical keying of the concrete with the irregularities of the steel tube. This keying may be manifested in two ways. First, the concrete is bonded by the interaction with surface irregularities due to the roughness of the tube. This type of interlocking, termed microlocking, must be overcome if the concrete core is to move as a whole. It is microlocking which defines the ultimate bond strength. At the ultimate bond strength, the concrete at the interface crushes and the stiffness decreases substantially. The second type of interlocking, macrolocking, occurs due to the nonuniformity of the tube, i.e., out-of-straightness or out-of-roundness. Macrolocking accounts for the frictional resistance provided beyond the ultimate bond strength and was illustrated by the remarkably parallel load-deformation slopes of all the tests in the later stages of the curve. It was further confirmed by tests of tubes that had smooth inner surfaces and were manufactured to more exact tolerances. These tests showed a complete absence of macrolocking. Both microlocking and macrolocking may be improved by better compaction resulting in an increase in bond strength.

By applying a statistical analysis to the test values, the characteristic ultimate bond strength recommended for design was determined as 150-160 psi.

<u>Morishita, Y. and Tomii, M., 1982</u> (c; po, r)

<u>Introduction</u>. An experimental study on bond strength of square CFTs was presented. The effect of cyclic shearing force, concrete compressive strength, and magnitude of axial load on bond strength were examined.

<u>Experimental Study, Results and Discussions</u>. A total of twenty-four specimens were tested. The specimens were divided into six groups having different concrete compressive strength and magnitude of axial load. One specimen in each group was filled with concrete up to the top of the steel tube and identified as the confined specimen, while the others were identified as unconfined specimens, as they had a gap at the top of the steel tube. The measured yield strength of steel and the measured compressive strength of the concrete were varying from 49.2 to 50.9 ksi and 3.32 to 5.08 ksi, respectively. All the tubes were annealed from residual stresses. The ratio of the applied axial load over the nominal axial load capacity of the columns ranged from 3 to 22%. The D/t ratio for the specimens was 35.1. The cantilever specimens were tested under constant axial compression and cyclic shear force, both applied at the top of the column. The axial load was applied through end plates attached to the columns. Thus, the axial load was applied on to the steel tube alone for unconfined specimens and on to the steel tube and concrete simultaneously for the confined specimens. All the specimens were supported by steel and concrete simultaneously at the bottom.

The chord rotation versus shear force diaphragms of the confined and unconfined specimens showed good correlation under low axial load. However, the confined specimens had a larger ultimate shear force than the unconfined specimens in the case of a high level of axial compression. It was also found that when the axial load level was high, the ultimate shear force of the confined specimens increased with an increase in the compressive strength of the concrete. Nevertheless, the unconfined specimens were not affected with concrete strength at high axial load levels.

When the specimens were subjected to axial compression only, the confined specimens exhibited continuity between the axial strains of the steel and concrete along the whole length of the column. For the unconfined specimens, the continuity was limited to the bottom portion of the CFT, where both the concrete and the steel were in contact with the endplate at the bottom of the CFT. However, when high axial load was applied, a portion of the load still shifted to the concrete towards the bottom of the CFT, but the continuity of strains was not satisfied along the length of the columns in the unconfined specimens.

In the case of cyclic shearing force, which resulted in bending moment in the specimens and thus in cyclic stresses being applied to the CFTs, it was found that the constant axial compression acting on the steel tube decreased along the column length as the cyclic shear load value increased. The rate of decrease was larger at the bottom than it was at the top. However, the steel tube recovered its axial load after the shear load was removed.

The inelasticity of the steel tubes was examined at 1% and 2% chord rotation levels through assessment of the strain gage readings on the steel tubes. It was found that more sections along the length of the specimens towards the top exhibited inelastic response as the axial load level increased.

The following formulas were proposed for the mean bond stress and average slip between steel tube and concrete, respectively:

$$f_b = \frac{P - A_s E_s \varepsilon_s}{\rho_i (l_0 - l_3)}$$

where l_0 is the height of the column and l_3 is the length from the fixed end to the section numbered as 3, which was close to the mid-height; and:

$$d_s = d_{so} - \frac{1}{2}\left(\int_{l_3}^{l_0} \varepsilon_s dz - \int_{l_3}^{l_0} \varepsilon_c dz\right)$$

where d_{s0} is the slip at the top of the column, , l_0 is the length of the column, and dz is the infinitesimal column length.

From the experimental results, it was found that average bond strength varied noticeably with the shear force. The bond strength was also a little larger in the case of high axial load, due to enhanced confinement, and it was not affected by the compressive strength of concrete. As the slip got larger in later cycles of loading, the bond strength within each cycle remained approximately constant, thus reaching a plateau . The range of the bond stress at the peak chord rotation was 21.3 to 49.8 psi and 21.3 to 42.7 psi for small and high axial load levels, respectively. Therefore, it was decided that 21.3 psi was a reasonable, conservative estimate of bond strength.

<u>Shakir-Khalil, H., 1993</u> (c; po, m)

<u>Introduction</u>. Push-out tests were conducted on circular, rectangular and square CFT members (Shakir-Khalil, 1993a, 1993b). The main purpose of the experimental study was to investigate bond strength of the steel-concrete interface. In addition, the performance of shear connectors and load-slip behavior of the specimens were examined.

<u>Experimental Study, Results and Discussions</u>. Two types of test setup were prepared. In the first one, the specimens rested on the steel tube wall having a 1.97 in. gap at the bottom and an axial load was applied on to the concrete alone at the top. In the second setup, an axial force was applied to the concrete alone while steel brackets or plates were attached to the steel tube and placed on supports having no restraint for rotation. The specimens were also resting on the steel tube with 1.97 in. gap at the bottom. Ten series of tests, which were named as X, Y, A, B, C, D, E, F, G, and H were conducted. The specimens in test series Y were 9.84, 17.71, and 23.62 in. in length and had a gap of 1.97 in. at the bottom. All the remaining specimens had a length of 17.72 in. and an interface length of 15.75 in.. Mild steel Grade 43 was used to manufacture the steel tubes. Either Hilti nails or black bolts were used as shear connectors.

The test series X included sixteen rectangular specimens in groups of four. For each specimen, the first type of experiment test setup was used and the *D/t* ratio was equal to 24. The specimens in the first group were specified as control specimens, which had no shear connectors. The other specimens had black bolts as shear connectors. Two specimens in each group were loaded up to failure. Two cycles of repeated axial loading was applied to the remaining two specimens before loading them to failure. The cycles were made at load levels of 30%, 60% and 90% of the average failure load of the first two specimens. The control specimens had an average bond strength of 120 psi. The failure load of the specimens with shear connectors seemed to increase at the same rate with the number of bolts. The specimens with shear connectors exhibited almost a bilinear load- slip response in the elastic range. This showed that the shear connectors were observed to be effective after the steel-concrete bond resistance was lost. As the black bolts rotated, the steel tube walls also deformed and the bolts had a larger shear area combined with tensile resistance against slip. Both of these effects caused failure loads larger than expected. For the repeated axial loading, no significant detrimental effect on the load-slip response was observed.

In the test series Y, the first type of test setup was utilized and twenty-four specimens were tested in groups of four without any shear connectors. The *D/t* ratio was 30 for the square tubes and it was equal to 33.7 for the circular tubes. The length of the specimens in each group was selected among the values of 9.84, 17.72, and 25.59 in.. The interfaces of some of the tubes were oiled before casting of concrete. After the test, the friction marks inside the steel tubes showed that higher frictional forces were generated at the steel-concrete interface of the circular specimens compared to the square specimens. The average bond strength for the circular specimens was found to be 82% and 64% greater than the square specimens in the case of dry and oiled interfaces, respectively. Moreover, the bond strength of the dry specimens was approximately two times the bond strength of the equivalent oiled specimens. Square specimens of this test series had smaller bond strengths than rectangular specimens of the test series X. This was attributed to the smaller *D/t* ratio of the rectangular tubes making steel tube wall stiffer and less shrinkage of concrete in the rectangular tubes due to the smaller cross-section area. The strains recorded on the steel tubes showed a gradual increase from the top to the bottom. In the linear part of the load-slip curves, the oiled specimens had a higher slip rate. However, the load-slip curves of the oiled and dry specimens had the same characteristics after failure load.

Test series A and B each consisted of ten specimens and they were tested in the first type of test setup. Either Hilti nails or black bolts were used as shear connectors. In test series A, square specimens with a *D/t* ratio of 30 were tested. The specimens in test series B were circular and had a *D/t* ratio of 33.6. The control specimens of test series A, which did not have any shear connectors, exhibited an average bond strength of 29 psi. The shear connectors appeared to become effective following the loss of bond strength and the load-slip relationship of the specimens with shear connectors were bilinear in the elastic range. Some specimens were subjected to subsequent unloading and reloading during their post-peak load-slip response. Among those specimens, the ones having Hilti nails as shear connectors regained their failure load when they were reloaded. However, this was not possible for the bolted specimens as the bolts were sheared off at failure. The steel strains were found to increase gradually toward the bottom. The rate of strain increase for the bolted specimens was found to be greater at the location of bolts due to the load transfer from the concrete core to the steel tube through the bolts. In test series B, similar response to test series A was obtained and the bolted specimens could not reach their failure load after they were subjected subsequent unloading and reloading. Due to the rotation of the shear connectors, the steel tube walls distorted and this caused an increase in bond strength, which was not noticeable in test series A. The strain in the steel tube increased toward the bottom and the rate of strain increase was observed to change at the load level, at which the control specimens failed. In addition, the rate of strain increase was greater at the bolt locations due to load transfer from the concrete core to the steel tube through the bolts. As a result of the higher friction in the circular specimens of test series B, their failure load was about two times greater than the failure load of square specimens of test series A.

Test series C had six square specimens with a *D/t* ratio of 30 and test series D included two circular specimens with a *D/t* ratio of 33.6. In both test series, each specimen had two 7.87 in. high steel brackets. The brackets were attached at the middle of two opposite sides for both of the circular and for two of the square specimens. For the remaing four square specimens, the steel brackets were located at two opposite corners. Shear connectors were used only in two square specimens, which had brackets at their corners and Hilti nails were selected as shear connectors. In test series C, the longitudinal strains were much lower than the ones measured in the previous test series. In addition, tensile strains were recorded on the steel tube below the brackets. This resulted circumferential contraction and improved the resistance to slip. The control specimens with steel brackets at the corners had 17% more failure load than the ones with steel brackets at the sides. The average bond strength for control specimens was 140 psi. The failure load of the control specimens and the specimens with shear connectors were 5 times and 2 times that of the eqauivalent specimens in test series A, respectively. The control specimens experienced sudden drop in strength after failure with a slip of about 0.118 in.. However, for the specimens with shear connectors, no deterioration of strength in post failure region was observed. The specimens in test series D had an average bond strength of more than 580 psi. Their failure was due to excessive deformation and tearing of the steel brackets. The amount of slip was insignificant at the end of the tests. The failure loads of the specimens in test series D were much greater than the failure loads in test series B.

Test series E and F consisted of specimens with steel brackets at the sides. Test series E had five pairs of square specimens having *D/t* ratio 31.7 and test series F had five pairs of circular specimens having a *D/t* ratio of 34.8. Only Hilti nails were used as shear connectors. In test series E, the control specimens exhibited a bond strength of 58 psi. The specimens having nails at the same steel walls with the brackets had better load-slip response than the ones having nails at different steel walls. Most of the specimens did not have a definite failure load. After the test, this was attributed to large deformation of the Hilti nails without releasing their grip in the concrete core. In addtion, bracket distortion and separtion of steel and concrete along the bracket height also occurred for some of the specimens at the end of the test. In test series F, two control specimens had failures at the brackets. Thus these specimens were retested by seating them on their base. This method of testing was applied for the remaining specimens. They exhibited no definite failure and the tests were stopped when the applied loads reached four times the estimated failure loads.

In test series G and H, four specimens were tested. The specimens in test series G were square and had a *D/t* ratio of 30. The ones in test series H were circular and their *D/t* ratio was 33.7. The control specimens were tested by seating them on their base and the other specimens were tested by supporting them on the side plates attached to steel tubes. The side plates were placed either at the top of the steel tube or at a distance of 3.94 or 7.87 in. below the top of the steel tube. No shear connectors were provided for the specimens. In test series G, the control specimen and the specimen with side plate at the top of the steel tube experienced sudden slip followed by a drop in the applied load. They had 61 and 67 psi bond strengths, respectively. The other specimens had a bilinear load-slip response with no defirnite failure load. The change of slope of their load slip curve occurred approximately at the failure load of the first two specimens. For these specimes, it was possible that tensile strains occured in the steel tubes and this increased the resistance to slip by contracting the steel tubes laterally. Thus these tests were terminated when excessive slip took place. The specimens of test series H showed similar responses with the specimens in test series G. However, the bond strengths and failure loads in test series H was more than two times the ones obtained in test series G. In addition, the specimens of test series H exhibited larger slip under increasing load compared to equivalent specimens in test series D.

<u>Shakir-Khalil, H. and Hassan, N. K., 1994</u> (c; po, m)

<u>Introduction</u>. Push-out tests were conducted on rectangular CFT columns in this research. Several factors that could affect the bond strength, including the shear connector pattern, concrete grade, concrete age, and water-cement ratio were studied.

<u>Experimental Study</u>. A total of five series of specimens (A, B, C, D, and E) were tested. The axial load was applied to the concrete core at the top, while there was a 1.97 in. gap below the concrete at the bottom of the CFT. Thus, the steel tube alone supported the columns at the bottom. All of the specimens had the same steel tube size with a *D/t* ratio of 30. The cubic compressive strength of the concrete varied from 3.19 to 9.86 ksi. The columns had a total length of 17.72 in.

All CFTs in test series A included shear connectors. Either black bolts or threaded bars were attached to all of the steel tubes, excluding the control specimen, which had no shear connectors. The threaded bars were inserted either straight or at a 45° inclined position. The direction of inclination and position of these bars also varied. A bond strength of 36.3 psi was obtained for the control specimen. The specimen that used threaded bars as shear connectors exhibited a lower failure load and a more dramatic decrease in strength when it was compared to the specimen with bolts for shear connectors. This was because the threaded bars could not undergo enough rotation, while the black bolts resisted the slip of the concrete core with a significant amount of rotation. However, the initial stiffness of the specimen with the threaded bars was higher. The specimens having inclined threaded bars were simultaneously subjected to shear force and compressive or tensile force as the slip proceeded. It was found that the specimen with shear connectors having inclination in the opposite direction to the slip, in which the shear connectors were subjected to shear and compression, had 30% higher failure load than the specimens having shear connectors inclined in the same direction with the slip. The former specimen also showed the best slip response in this test series in terms of both strength and ductility. The position of the inclined bars that were in the same direction with the slip was found to have no effect on the failure load. However, more ductile slip response was obtained when the threaded bars were placed at the same level on the two sides of the steel tube.

The specimens in tests series B were divided into two subgroups having different concrete strengths as the concrete was casted at different times. Both subgroups had black bolts as shear connectors, although there was also control specimens with no shear connectors. The number and spacing of the shear connectors varied. In addition, for some of the specimens, the steel tube was reinforced with steel plates at the bolt locations. The bond strength of the specimen with no shear connectors was 30.5 psi for the subgroup having relatively higher concrete strength, and it was 60.9 psi for the subgroup with lower concrete strength. The steel plates reinforcing the steel tube caused a 60% increase in failure load, as it caused the bearing capacity of the bolts to increase. On the other hand, this resulted in a more rapid decay in strength as the bolts were forced to fail in shear, which was a more sudden type of failure. The specimens with no stiffening plate behaved in a ductile manner, as the bolts underwent large amounts of rotation. In addition, as the bolts deformed, the shear area increased and the bolts resisted the push-out force both in shear and in tension. The spacing of the bolts did not have any significant affect on the response of the specimens.

Test series C was divided into three subgroups with different concrete strengths. All of the specimens had no shear connectors. The average bond strength of the subgroup with the highest concrete strength was lowest. This was attributed to the large shrinkage as a result of high cement content. The average bond strength for all of the groups was less than 58 psi.

Test series D, E and, F had subgroups with concrete ages of 28, 56, and 85 days. Test series D had the highest cement content and its bond strength was the lowest. For each subgroup, when the age of the concrete increased, it was found that the bond strength decreased. The amount of reduction in bond strength was lowest for the test series F, which had the lowest cement content. Although the load-slip response for the specimens with low strength concrete had a high failure load, the specimens exhibited a dramatic drop in strength after the peak load. On the contrary, the response of the specimens with high cement content showed a ductile response and lower failure load. When the post-peak response of the specimens were compared, the ones having low and medium strength concrete had higher strength and this might be due to low shrinkage of concrete in these specimens.

The authors concluded that the amount of shrinkage is important for bond strength. It is possible to limit the shrinkage by using a low cement content and high aggregate-cement ratio.

Kilpatrick, A. E. and Rangan, B. V., 1999 (c, bc; m)

Introduction. In this paper, an experimental program was presented to study the effect of bond on the load-deformation response of circular CFT columns and girders. To represent typical steel-concrete interfaces, different bond conditions were prepared. Moreover, tubular columns were tested and the results were compared with the CFT specimens.

Experimental Study, Results and Discussions. A total of thirteen circular CFT columns and girders were tested. The main parameters of the tests were the bond conditions, the *L/D* ratio, and the type of axial loading. Three bond conditions were prepared and they were classified as maximum bond, partial bond, and minimum bond. For the maximum bond condition, self-tapping screws were inserted into the steel tube and the specimens were treated in an acid bath and then in an alkaline bath to improve the chemical bond between steel and concrete. In the case of the partial bond condition, the grease on the inside of the steel tube was removed. To prepare the minimum bond condition, the specimens were kept in as-received condition and the inside of steel tube was oiled. The *D/t* ratio of the specimens was 42.3. The nominal yield strength of steel was 50.8 ksi and the average measured compressive strength of concrete was 15.30 ksi. A displacement-controlled axial load was applied to three CFT stub columns and one HT stub column. The *L/D* ratio of the stub columns was equal to 3.5. A total of eight short columns and slender columns, which included 2 HT columns, were tested under eccentric displacement-controlled axial load. The eccentricity was provided by displacing the columns from the axis of loading. The approximate *L/D* ratios for the short and slender columns were 10 and 19, respectively. The girders were simply supported and had an 82.67 in. span length. The total number of the CFT girder specimens was three. They were loaded at their third points and displacement-controlled loading was utilized. All of the girder specimens, excluding one of them, were capped at the ends.

For the column specimens, the bond strength did not influence axial load capacity and load deformation response significantly. The difference was not more than 3.6% for the minimum and maximum bond conditions when the axial load capacities were compared. All of the column specimens exhibited stable and ductile load deformation responses. The largest enhancement in axial load capacity and load deformation response, when the bond conditions were improved, was observed in the stub columns. However, these improvements were mainly attributed to discrepancies in the compaction of the concrete, which was harder to achieve in short tubes. The concrete infill increased the axial load capacity by a factor of 3.86 for the stub columns while this factor became 2.61 and 2.13 for the short columns and slender columns, respectively. The reduced improvement of capacity was due to the fact that less amount of concrete was under compression in the short columns and slender columns as they were loaded eccentrically. Similar to the column specimens, the bond strength was also found to have insignificant effect on the behavior of the CFT girders. For the girders having capped ends, the capacity was higher by approximately 8% versus the comparable specimen with uncapped ends.

In addition, the specimen with uncapped end experienced reduced stiffness at an elevated load . These findings showed that the longitudinal confinement could affect the behavior of CFT girders.

Roeder, C. W. et al., 1999 (c; po, m, r)

Introduction. The load transfer mechanism between steel and concrete in circular CFT columns was examined. The authors presented bond strength and slip characteristics observed in experimental studies from the available literature. Moreover, they processed the experimental results and proposed equaitions for bond strength.

Experimental Study, Results, and Discussions. Static and dynamic analysis of CFT systems in past studies from the literature showed that bond stress demand was highest at the connections and foundation supports. In addition, braced frames were found to have more bond stress demand compared to unbraced frames. This was attributed to high vertical load coming from the braces at the connections.

Three states of bond (A, B, C) were presented depending on radial displacement of concrete, amount of shrinkage and amplitude of irregularity on steel tube surface. The radial displacement of concrete could be in the form of expansion due to lateral concrete pressure or in the form of contraction due to shrinkage. In state A, both cohesive chemical bond and mechanical resistance was effective for bond strength. However, concrete pressure should be about 174 psi to reach state A in typical applications. The steel and concrete was separated from each other and mechanical resistance was poor in state B. The most common case for CFT applications in the U.S. was state C. The chemical bond was not significant and the mechanical resistance was decreasing rapidly. The behavior of state C was changing with *D/t* ratio and shrinkage. The mechanical resistance was low for large *D/t* ratios due to reduced radial stiffness. However, the effect of concrete shrinkage was not clear particularly for long columns.

In the available literature, bond tests were mostly conducted on specimens with *D/t* ratios between 15 and 35. The maximum diameter was on the order of 12 in.. According to the results, the average bond strength of rectangular CFTs was about 70% smaller than that of circular CFTs. The concrete strength did not have a significant effect on bond strength. Due to large scatter of the data, it was not possible to determine a trend of bond stress depending on diameter and *D/t* ratio.

Two series of experiments were conducted to examine bond strength of circular CFT columns. The specimens were tested under axial load, which was applied to concrete alone. The specimens had an air gap of 2.31 in. at the bottom and rested on the steel tube. The axial load was either concentrically or eccentrically applied. The tests were monotonic, except that one specimen was loaded cyclically. The main variables of the tests were the diameter of the concrete core, the steel tube thickness, and the shrinkage of concrete. The diameter of steel tubes varied from 9.84 to 25.6 in.. The range for the *D/t* ratio was 20 to 110. Specimens in test series I had concrete with moderate shrinkage, while the specimens in test series II had concrete with low shrinkage. The compressive strength of the concrete was 7.40 ksi. The mechanical properties of the steel tubes was not provided as the applied loads were not high enough to yield the steel tube.

After achieving the ultimate load, the slip resistance was found to decrease with the increase in slip. This trend was typical among the specimens of test series II. In the axial load versus slip curves of the specimens in test series II, a point at which slip started to increase sharply was identified. At that point, the initial bond between steel and concrete was broken and friction started to dominate the bond resistance. This occurred at load levels of 40 to 80% of the ultimate capacity of the column. Initial breakage of the concrete-steel bond was not evident among the specimens of test series I. The bond stress potentials of the specimens were estimated using their acoustic responses. For this purpose, the type of sound generated by tapping outside of the steel tubes was examined. If the sound was solid, bond response was estimated to be close to state A or in state C. If the sound was hollow, the bond response was close to state B or in state C. The majority of the specimens in test series II had a solid sound, while hollow sounds were common in test series I. The acoustic responses agreed well with the experimental bond stresses. The bond stress had an exponential distribution at low load levels, and the distribution became more uniform after slip took place. This type of bond stress-slip response was observed for all of the specimens. For the specimens with low bond strength, the circumferential strains were negligible. Those strains were significant if the bond capacity of the specimens was high. This showed that the specimens with high bond strength developed large contact stresses between the steel and concrete due to the Poisson effect. The eccentrically loaded columns had average bond stress values 1.23 and 2.52 times the average bond stress of the concentrically loaded specimens. The cyclic loading caused deterioration in ultimate strength and bond strength of the specimen. The reduction in bond strength was about 50%.

The bond strength of the specimens in test series II was approximately two to three times larger than the specimens of test series I. This trend and the acoustic response of the columns proved that shrinkage took place in long CFT members.

Analytical Study. CFT columns were analyzed using finite element analysis for typical section sizes to study the interaction between the steel and concrete. Three-dimensional solid elements were utilized and columns were subjected to axial forces and bending moments. At the beginning, loading was applied to either steel or concrete alone. Elastic load transfer then took place until composite action was developed. The results showed that the bond stress had an exponential distribution when slip was prevented. The maximum bond stress occurred at the end of the column where loading was applied. However, it decreased to zero over a distance of approximately $D/2$. This distance was smaller for high D/t ratios and larger for low D/t ratios. When slip was allowed, the bond stress was found to have a uniform distribution and then decreased to zero exponentially after a distance of $0.2D$ from the loaded end.

The bond strength data obtained in this experimental study was compared with other experimental data from the literature. It was found that bond strength decreased rapidly with an increase in the D/t ratio. However, the individual effects of D and t did not exhibit a clear trend. A linear regression analysis on the available data was performed and the following formula was proposed for maximum average bond strength (in MPa) for circular CFTs:

$$f_b = 2.109 - 0.026(D/t)$$

Type of element tested
bc - beam-column
br - bracing member (axial compression and tension)
c - column
ct - column with torsion
pb - pure bending
sh - shear

fr - CFT frames or subassemblies

fs - fully-restrained connection
pz - panel zone of the connection
sp - simple connection

Type of loading
m - monotonic
r - repeated or cyclic

po - push-out

III. Tables of Experimental Data

Table 1. Axially Loaded Column Tests

Article	**Experiment Synopsis**	**Number of Tests**	**Tube Sizes** (in.) O: diam. (D) : depth (D) × width	**Wall Thickness (t)** (in)	**Diameter/ thickness (D/t)**
Klöppel, Goder, '57 (in German)	Concentrically loaded CFTs and HTs	104 tests	3.75, 4.75, 8.5 (circular)	0.079-0.472	7.9-60.5
Salani, Sims, '64	Seamless mortar-filled columns	17 Mortar-Filled Tubes 9 HTs	1, 1.5, 2, 2.75, 3 (circular)	0.035, 0.065, 0.109	28.6-46.2
Gardner, Jacobson, '67	Short and long CFT columns (experimental. vs. theoretical)	32 CFTs	3, 4, 4.75, 6 (circular)	0.067-0.194	30-48
Gardner, '68	Short and long columns w/ spiral welded tubes	17 CFTs 2 HTs	6.62-6.66 (circular)	0.104, 0.142, 0.197	34-64
Knowles, Park, '69	Concentric and eccentric loading w/ varying KL/r (experimental, theoretical)	28 CFTs (18 conc., 10 ecc.) 30 HTs (20 conc., 10 ecc.)	3.25, 3.5 (circular) 3.0 × 3.0 (square)	0.23, 0.055 (circular) 0.132 (square)	15.2, 59.1 (circular) 22.6-22.9 (square)
Neogi et al., '69	Elasto-plastic behavior of pinned eccentrically and concentrically loaded CFTs (exp. vs. theor.)	18 CFTs (ecc.) C: cold-drawn (8) M: mild, hot-finish (10)	C: 5.0; (circular) M: 5.5, 6.625 (circular)	0.064-0.384	14.4-78.1
Knowles, Park, '70	Design eqns. developed and compared w/ tests by author and others	111 CFTs (previous tests)	1.0-14.0 (circular) 4 × 4, 5 × 5 (square)	0.035-0.502	N.A.

Type of Steel, f_y (ksi) (measured)	f_c' (ksi)	Length (L) (in) (Eff. length or slenderness where noted)	Length/ Diameter (L/D)	Eccen-tricity (in.)	Residual Stresses (ksi)	Initial Out-of-Straight-ness (in)
N.A. 38.3-57.3	2.94-4.32	L_{eff} = 33.86-90.95 λ = 36.9-114.1	8.68-20.80	N.A.	N.A.	N.A.
Cold-drawn seamless finish-annealed 76	3.35-4.95 (Mortar)	60.0	20-60	N.A.	Steel tubes annealed to remove stresses	N.A.
Cold-drawn seamless finish-annealed 52.7-91.9	3.0-6.3	6, 8, 9.5, 12, 24, 41.34, 60, 66	2, 8, 8.7, 11, 15, 20	N.A.	N.A.	N.A.
Spiral welded 28.6-48.3	2.6-5.3	CFT: 12.0, 72.0, 84.0 HT: 68.0 L_{eff} = L + 6.0	1.8-12.8	N.A.	Incorporated into f_y	N.A.
Hot-finish mild seam-less (cir) welded (sq) 58, 70 (cir) 47 (square)	No cylinder test done (avg. max stress = 5.925)	concentric: 9-68 (λ = 7.76-60.2) eccentric 32, 44, 56	2.60-22.67	0.30, 1.00 (initial) 0.50-1.45 (at failure)	N.A.	N.A.
Seamless M: mild, hot-finish, gr.16; C: cold-drawn 25.0-40.4	f_{cu} = 4.64-12.10	C: 55.5, 67.5, 80.0; M: 131.0	11.1-23.7	C: 0.25-0.88; M: 1.25-1.88	N.A.	0.022-0.224
N. A. 36.9-87.8	2.94-9.60	L_{eff} = 10.0-91.0	N.A.	N.A.	N.A.	N.A.

End Conditions	Loading Method	Results Reported	Main Test Parameters, Comments
Pinned-pinned	N.A.	-P vs. various σ's and ε's in both concr. and steel (tabular & graphical)	-Individual tests tabulated by load increment (very detailed)
Fixed-fixed	-Concentric -Slow loading in equal increments	-P vs. ε -P vs. ν, P vs. E_c -Comparison: P_u, P_{cr}	-Parameters: D, CFT vs. HT -Mortar-filled
Pinned-pinned (long cols.) Pinned-fixed (short)	-Concentric loading to failure	-P_o, P_u, several allowable load calcs incl. ACI, NBC, Klöppel ('57), & author's own -K (author's "lateral restraint factor") vs. P/P_u	-Parameters: end conditions, D/t -Discussion by Furlong and Knowles followed (1968)
Pinned-pinned (long cols) Pinned-fixed (short)	-Concentric loading to failure	-Manufacturing ε, ε_{circum} -P vs. ε, P vs. ε_{circum} -ACI & NBC P_a, P_u, P_o, P_{cr}	-Parameters: type of tubing, D/t, L -Thorough examination of residual stresses
Pinned-pinned (knife edge) Stub col: fixed ends Ends packed w/ cardboard-uniform load on both mat'ls	-Offset one end of column to produce eccentricity -Uniaxial bending	-σ vs ε, σ vs. E, σ vs. λ (HTs, concrete cores) -P_u/P_o (HT, CFT) -P_u, M_u, P_u/P_o, M_u/M_o (HT, CFT) -P_u/P_o vs M_u/M_o (HT, CFT)	-Parameters: HT vs. CFT, λ -Very thorough, detailed; HT/CFT comparison useful
Pinned-pinned (knife edge) Uniaxial bending Ends capped such that both mat'ls were loaded	-Eccentricity varied by moving top end of column laterally (single curvature) -Short duration, constant loading	-P vs. ε (exp, calc) -P vs. δ, M vs δ (exp, calc.) -Selected load ratios using: P_u, P_y, 'exact' calc. load, calc. cos method (Neogi & others)	-Parameters: type of tubing, D/t, L/D, e
N.A.	N.A.	-P_o, P_u, P_u/P_o for all tests	-Parameters vary with author -Test of the authors' proposed formulas

Article	Experiment Synopsis	Number of Tests	Tube Sizes (in.) O: diam. (D) : depth (D) × width	Wall Thickness (t) (in)	Diameter/ thickness (D/t)
Bridge, '76	Square pin-ended eccentrically loaded CFTs (experiment theory)	8 CFTs (ecc.)	5.91 × 5.91, 7.87 × 7.87 (square)	0.256, 0.394	20.0, 23.1
Kitada et al., '87	Short CFT columns subjected to axial compression	14 CFTs	4.5 (circular)	0.118, 0.177, 0.197	22.9, 25.4, 38.1
Zhong, Miao, '88	Short CFT columns subjected to axial compression	11 CFTs	4.27 (circular)	N.A.	N.A.
Liu, Goel, '88	Cyclic load behavior of CFT bracing	6 CFTs (brace) 3 HTs (brace)	6 × 3, 4 × 2 (rectangular)	0.188, 0.125, 0.25	14, 30
Kawano, Matsui, '88	Cyclic axial loading of CFT braces	10 CFTs 10 HTs	2.38 (circular)	0.091	26.3
Matsui, Kawano, '88	Monotonic and cyclic loading of trusses with CFT and HT chords	1 CFT and 1 HT (monotonic) 1 CFT and 1 HT (cyclic)	2.39 (circular)	0.083	28.9
Shakir-Khalil, Zeghiche, '89	Concentric and eccentric loading of rectangular CFTs	1 CFT (conc.) 6 CFTs (ecc.) addt'l squash load and bond strength tests performed	4.72 × 3.15 (rectangular)	0.197	major axis: 24 minor axis: 16

Type of Steel, f_y (ksi) (measured)	**f_c'** (ksi)	**Length (L)** (in) (Eff. length or slenderness where noted)	**Length/ Diameter (L/D)**	**Eccen-tricity** (in.)	**Residual Stresses** (ksi)	**Initial Out-of-Straight-ness** (in)
N.A. 36.8-46.3	4.38-5.48	83.9, 120.1	10.65, 15.25, 20.33	0, 1.50, 2.52	N.A.	0.011-0.055
Welded seam & seamless 40.5-52.6	2.50, 4.96	9.94	2.21	N.A.	N.A.	N.A.
N.A. Stress determined analytically	4.35, 5.80	see L/D	2, 2.5, 3, 3.5, 4, 4.5, 5	N.A.	N.A.	N.A.
A500 gr. B, cold-formed 54, 60	4, 6, 8	λ = 58-100	N.A.	N.A.	N.A.	N.A.
Cold-formed, mild steel 48.5	4.75, 4.98, 5.08	L = 16.2-97.4 λ = 19.9-119.5	6.81-40.88	N.A.	N.A.	N.A.
Cold-formed, mild steel 48.5	4.98	L = 47.9 λ = 59	28.67	N.A.	N.A.	N.A.
Rolled, grade 43 49.8-56.0	f_{cu} = 5.80-6.53	108.7 L_{eff} = 115.7-126.4	23.0	e_x = 0.0, 0.94, 2.36 e_y = 0.0, 0.63, 1.57	N.A.	Acknow-ledged as possible error

End Conditions	Loading Method	Results Reported	Main Test Parameters Comments
Pinned-pinned (rocker bearings) Expansive mortar endcap (for equal load distribution)	-Incr. loading to failure -Ends offset equally (single curvature) -Biaxial bending (inclination of axis: 0°, 30°, 45°)	-P vs. δ (exp., calc.) -M-ϕ-P curves -P_y, P_u, P_o, P_u/P_o, P_y/P_o	-Parameters: λ, e, inclination of loading axis -Excellent paper -- very clear and detailed
Fixed-fixed Loading through bearing plates	-3 cases: load steel only, concrete only, & both mat'ls simultaneously	-σ_{sl}/f_y vs. σ_{sc}/f_y for steel tubes -P vs. ε_{circum}, P vs. δ_{axial} -P_u (exp. vs. calc.)	
Pinned-pinned: (2 knife hinges; 1 knife, 1 plate hinge; 1 spherical, 1 pl hinge; or 2 pl. hinges)	-Concentrically-applied load	-σ vs. ε -P vs. ε	-Parameters: L/D ratio, steel ratio, end conditions -Mostly theoretical
Welded gusset plates, weld and plate strength 33% > steel tube	-Rectangular, pinned frame loaded laterally w/ diagonal brace put in alternate compression & tension	-P/P_y vs. δ_{axial} -P/P_y vs. # of cycles (vary D/t) -First buckling load (exp, theor) -Load histories -Energy absorption (CFT, HT)	-Parameters: HT vs CFT, f_c', D/t, λ -Very good failure descr. -Steel fibers used in 3 of the concr. mixes to vary f_c'
Pinned-pinned	-Cyclically applied axial load at both ends -Displacement controlled loading with either large or small amplitude	-P vs. λ -P vs. δ_{axial} -Energy absorption (CFT, HT) -H vs. Δ (K-braced frames with HT and CFT braces)	-Parameters: HT vs. CFT, λ, axial load amplitude
Pinned-pinned (monotonically loaded trusses) Cantilever (cyclically loaded trusses)	-Monotonic uniform moment applied at the ends -Constant axial load and cyclically applied lateral load applied at the top	-M vs. θ -P vs. δ_{axial} -H vs. Δ	-Parameters: HT vs. CFT
Pinned-pinned Crossed knife edges (free biaxial rotat.) End plates (0.6 in. thick) welded to tube	-Cols tested horizontally -Equal end eccentricities -Incremental loading -Tests: 1 axial, 4 uniaxial (2 maj.,2 min.), 2 biaxial	-δ (x and y directions) -P vs. ε, P vs. δ, P vs. e/D -M vs. ϕ (varying P/P_o) -P_u, P_o, M_u -Bond tests: P_u, bond strength	-Parameters: L/D, λ, e -Bond test & squash load test conducted (see summary) -Detailed tabulation

Article	Experiment Synopsis	Number of Tests	Tube Sizes (in.) O: diam. (D) : depth (D) × width	Wall Thickness (t) (in)	Diameter/ thickness (D/t)
Cedarwall et al., '90	Eccentrically loaded rectangular CFT columns	19 CFT (ecc.) 19 short col. tests	4.72 × 4.72 (square)	0.197, 0.315	15, 24
Shakir-Khalil, Mouli, '90	Concentric and eccentric loading of rectangular CFTs	1 CFT (conc.) 8 CFT (ecc.) 9 short col tests bond strength tests	5.91 × 3.94, 4.72 × 3.15 (rectangular)	0.197	maj. axis: 24, 30 minor axis: 16, 20
Cai, '91	Eccentrically loaded CFT columns	27 CFTs	6.5 (circular)	0.197	33.2
Luksha, Nesterovich, '91	Large diameter CFTs under axial compression	30 CFTs 10 HTs	6.25-40.15 (circular)	0.20-0.52	31.4-105.8
Masuo et al., '91	Concentric testing of lightweight concrete CFTs	26 CFTs: 18 lightweight, 6 normal weight	7.51, 10.53 (circular)	0.236, 0.276	32, 38
Nakai et al., '91	Study on creep and drying shrinkage of CFTs	4 CFTs 2 plain concr. (3 creep tests, 3 shrinkage tests)	6.5 (circular)	0.0, 0.177, 0.197	33.0, 36.7
Sakino, Hayashi, '91	Concentrically loaded stub columns	7 CFTs 5 HTs	6.85, 7.00, 7.05 (circular)	0.118, 0.217, 0.354	20, 32, 58

Type of Steel, f_y (ksi) (measured)	**f_c'** (ksi)	**Length (L)** (in) (Eff. length or slenderness where noted)	**Length/ Diameter (L/D)**	**Eccen-tricity** (in.)	**Residual Stresses** (ksi)	**Initial Out-of-Straight-ness** (in)
N.A. 44.1-63.7	5.65-14.90	118	25	0.39, 0.79	N.A.	N.A.
Rolled, grade 43 49.3-52.6	f_{cu} = 5.18-5.87	L_{eff} = major : 126.4 minor: 115.7 short col tests: 3.9, 7.9	major: 21.4, 26.75 minor: 29.5, 36.7	e_x = 0.0-2.95 e_y = 0.0-1.97	N.A.	Acknow-ledged as possible error
N.A. 40.2-45.5	f_{cu} = 5.05, 7.45	26.2-117.7	5.24-19.30	0.79, 1.57, 2.36, 3.94	N.A.	N.A.
Electronical-ly welded 42.3-56.8	2.18-6.67	18.8-120.5	3.0	N.A.	N.A.	N.A.
Cold-formed 73.3, 66.8	Light: 8.11 Normal: 7.01	45.3-189.0	6.0-18.0	N.A.	N.A.	Initial defl. = L/2000 at mid-height
N.A. 60.7, 63.9	4.04	39.4 (1 m)	6.05	N.A.	N.A.	N.A.
Annealed 36.0, 38.6, 41.1	3.21, 3.47, 6.33, 6.59	14.2	2.0	N.A.	Steel tubes annealed to remove stresses	N.A.

End Conditions	**Loading Method**	**Results Reported**	**Main Test Parameters Comments**
Pinned-pinned	-Axial load applied eccentrically to failure -Load applied to steel, concrete, and both mat'ls	-P vs. δ -P_u, P_o -P_o/P_u vs. P_{co}/P_{so}	-Parameters: t, f_y, f'_c, e, method of load application
Pinned-pinned Crossed knife edges (free biaxial rotat.) End plates (0.6 in. thick) welded to tube	-Cols tested horizontally -Equal end eccentricities -Incremental loading -Tests: 1 axial, 1 uniaxial, 7 biaxial	-ε distribution at mid-length -δ (x and y directions) -P vs. ε, P vs. δ, P vs. e/D -P_u, P_o for short columns -P_u, M_u	-Parameters: L/D, λ, e -Extension of Shakir-Khalil,'89
Pinned-pinned Stiff 1 in. cap plates	-Eccentric axial load -18 cols: ecc. at one end, single curvature -9 cols: opposite eccs., double curvature	-P_u, P_u/P_o	-Parameters: λ, e/(concr. radius), β
Fixed-fixed	N.A.	-P_o, P_y -% shrinkage	-Parameters: D, t -Variables not well defined
Pinned-pinned (cylindrical bearings)	-Concentrically-applied load	-σ-ε relations for steel & concr. -initial δ -P vs. δ curves -P_u, P_u/P_o -P_u/P_o vs. slenderness factor (see paper for definition)	-Parameters: D/t, slenderness ratio -Several detailed P-δ curves -Very thorough tests, highly detailed and documented
Fixed-fixed	-Concentrically-applied load	-ε_{concr} vs. time (both tests) -P vs. time (creep test) -Creep coefficients -Evaluated visco-elastic parameters (for theoretical)	-Parameters: time -Creep and shrinkage tests conducted simultaneously for 160 days
Pinned-fixed	-Concentrically-applied load	-σ vs. ε -$\varepsilon_{circum}/\varepsilon$ vs. ε -P vs. ε -P_o, P_y, P_u	-Parameters: D/t, f_c' -Paper attempted to estimate the strain hardening effect as well as triaxial confinement

Article	**Experiment Synopsis**	**Number of Tests**	**Tube Sizes** (in.) O: diam. (D) : depth (D) × width	**Wall Thickness (t)** (in)	**Diameter/ thickness (D/t)**
Tsuji et al., '91	Axial compression of short CFTs	3 CFTs	4.5 (circular)	0.138, 0.177	25.4, 32.7
Rangan, Joyce, '92	Eccentrically loaded slender columns w/ high-strength concr.	9 CFTs	4 (circular)	0.063	64
Bridge, Webb, '93	Axial compressions of thin-walled CFTs and HTs	2 CFTs 2 HTs	9.8 (circular)	0.079	124
Matsui et al., '93	Axial compression of square CFTs with varying lengths and eccentricities	24 CFTs 6HTs	6.12 × 6.12 (square)	0.177	33.3
Tsuda et al., '96	Series I - Concentrically and eccentrically axially loaded slender CFT's	48 CFT's (24 circ., 24 sq.) 12 HT's	6.51 (circular) 5.91 × 5.91 (square)	0.161 (circular) 0.168 (square)	40.4 35.2
Kawano, Matsui, '97	Cyclic axial loading of circular CFTs and HTs	6 HTs 44 CFTs	2.38, 4.00 (circular)	0.043-0.214	18.6-53.1
Kilpatrick, Rangan, '97	Eccentric loading of circular CFTs in double and single curvature	24 CFTs(ecc) 1 CFT(conc.)	4 (circular)	0.094	42.3

Type of Steel, **f_y** (ksi) (measured)	**f_c'** (ksi)	**Length (L)** (in) (Eff. length or slenderness where noted)	**Length/ Diameter (L/D)**	**Eccen-tricity** (in.)	**Residual Stresses** (ksi)	**Initial Out-of-Straight-ness** (in.)
Mild steel 49.2, 50.8	4.84	9.0	2.0	N.A.	N.A.	N.A.
N.A. 31.6	9.77	L_{eff} = 31.8-91.4	8.0-22.9	0.39, 1.18	N.A.	N.A.
N.A. 37.7	8.63	29.5	3.0	N.A.	N.A.	N.A.
Cold-formed from mild steel plate 66	4.6-5.0	23.6, 47.2, 70.9, 106.3, 141.7, 177.2	4, 8, 12, 18, 24, 30	0, 0.98, 2.95, 4.92	N.A. (yield stress measured by 0.2% offset)	N.A.
Mild steel; STK400 51.2 (circular) STKR400 59.8 (square)	4.62 (circular) 5.93 (square)	Noted values are kL; 26.0-195.1 23.6-177.2	kL/D; 4, 8, 12, 18, 24, 30 (same for both)	0-4.06 (circular) 0-4.92 (square)	N.A.	N.A.
Cold-formed STK400 44.8-61.2	4.37, 6.69	10.0, 12.8, 33.8, 71.3	5, 10, 20	N.A.	N.A.	N.A.
Cold-formed 59.5	13.92	85.6	21.4	-1.97-1.97	N.A.	N.A.

End Conditions	Loading Method	Results Reported	Main Test Parameters, Comments
Pinned (spherical)-fixed	-Concentrically-applied load	-ε vs. ε_{circum} -ν vs. ε -P vs. ε -P vs. ε (exp. vs calc.)	-Parameter: t -Tests conducted to check validity of analytical formulation
Pinned-pinned End plates on rollers	-Axial load offset by eccen. -Equal end eccentricities -Single curvature	-P vs. ε -P vs. δ -P_u, P_o, P_u/P_o	-Parameters: λ, e -Compared to results by Neogi, Sen, Chapman
Pinned (spherical seat)-fixed Ends plastered to ensure flush loading	-Concentrically-applied, incremental loading -Loaded into post-ultimate region	-P vs. ε_{concr} -P vs. δ_{axial} -P_u, P_o	-Parameter: CFT vs. HT -Tests performed for high-rise construction project
Pinned (spherical seat)-eccentricity imposed by moving bearing plate	-Concentrically and eccentrically applied -Slow loading near ultimate load -Loaded into post-ultimate region	-P vs. $\delta_{lateral}$ -P_u -P_{cr} vs λ for various eccentricities	-Parameters: λ, e, CFT vs. HT -Compared to AIJ design formulas
Pinned-pinned: Specimens are loaded through hemispherical oil film bearing at each end	-Concentrically and eccentrically applied axial load on CFT's -Concentric load only on steel tubes	-P vs. δ (lat. defl.) -M vs. P -M_u vs. P_u (inter. diagrams)	-Parameters: Magnitude of eccentricity, buckling length-section depth ratio (kL/D)
Pinned-pinned	-Concentric axial loading -Displacement-controlled	-P vs δ_{axial} - n_c, n_b vs. ε -W / $P_y \times \delta_y$ vs. ε -n_c vs. L/D -W vs. L/D	-Parameters: CFT vs HT, L/D, D/t, loading pattern - Energy absorption and fracture of steel tube
Pinned-pinned : The ends were clamped to knife-edge assemblages	- Eccentric axial loading -Displacement-controlled	-P vs. e -P vs. δ (lat. defl.)	-Parameters: magnitude and direction of eccentricity

Article	Experiment Synopsis	Number of Tests	Tube Sizes (in.) O: diam. (D) : depth (D) × width	Wall Thickness (t) (in)	Diameter/ thickness (D/t)
O'Shea, Bridge, '97a	Concentric and eccentric loading of circular HTs and concentric loading of circular CFTs filled with unbonded concrete	7 HTs (conc.) 5 CFTs (conc.) 10 HT (ecc.)	6.50, 7.48 (circular)	0.034-0.111	58.5-220.9
O'Shea, Bridge, '97b	Concentric loading of square box HTs and square box CFTs filled with unbondedand bonded concrete	17 CFTs (conc.) 12 HTs (conc.)	3.15 × 3.15, 4.72 × 4.72, 6.30 × 6.30, 7.87 × 7.87, 9.45 × 9.45, 11.02 × 11.02 (square)	0.084	37.3-130.7
O'Shea, Bridge, '97c	Concentric and eccentric loading of circular CFTs w/ unbondedand bonded high strength concrete	22 CFTs (conc.) 7 CFTs (ecc.)	6.50, 7.48 (circular)	0.034-0.111	58.5-220.9
O'Shea, Bridge, '97d	Concentric and eccentric loading of circular CFTs w/ unbonded/bonded high strength concrete	18CFTs (conc.) 7 CFTs (ecc.)	6.50, 7.48 (circular)	0.034-0.111	58.5-220.9

Type of Steel, f_y (ksi) (measured)	**f_c'** (ksi)	**Length (L)** (in) (Eff. length or slenderness where noted)	**Length/ Diameter (L/D)**	**Eccen-tricity** (in)	**Residual Stresses** (ksi)	**Initial Out-of-Straight-ness** (in)
Cold-rolled 26.9, 29.5, 30.6 Hot-rolled 37.2, 44.4 Cold-drawn 52.7	6.89	22.7, 26.2	3.5	0.28-0.83	-6.37-51.96 (bending) -11.40-47.30 (membrane)	N.A.
Mild steel plate 40.9	2.41-3.13	8.4-38.0	0.8, 1.2, 1.7, 2.3, 2.9, 3.5	N.A.	7.70-22.24	N.A.
Cold-rolled 26.9, 30.7 Hot-rolled 37.2, 44.4 Cold-drawn 52.7	5.54-11.63	22.7, 26.2	3.5	0.28-0.82	-6.37-51.96 (bending) -11.4-47.30 (membrane)	N.A.
Cold-rolled 26.9, 30.6 Hot-rolled 37.2, 44.4 Cold-drawn 52.7	11.18-16.00	22.7, 26.2	3.5	0.26-0.67	-6.37-51.96 (bending) -11.40-47.30 (membrane)	N.A.

End Conditions	Loading Method	Results Reported	Main Test Parameters, Comments
-Fixed (conc.): Ends attached to grooved plates filled with low temperature metal -Pinned-pinned (ecc.): Eccentricity provided through thick endplates with an offset half round	-Concentric load only on steel tube -Displacement-controlled with incremental small displacements	-σ-ε relations for steel & concr. -Residual stress distribution -P vs. ε_{axial} & σ_{sl} vs. ε_{axial} -(ε_{circum} & ε) vs. ε_{axial} -P_u -M_u vs. P_u (inter. diagrams) -Plate buckling curves -Lateral imperfection of the steel tube walls	-Parameters: CFT vs. HT, D/t, loading type (BS, BSC, E1, E2), magnitude of eccentricity -Effect of internal restraint on local buckling behavior
Fixed: Ends attached to grooved plates filled with low temperature metal	-Concentric load only on hollow tube -Displacement-controlled -Incremental small displacements	-σ-ε relations for steel & concr. -Residual stress distribution -P vs. ε_{axial} & σ_{sl} vs. ε_{axial} - ε_{axial} vs. δ (lat. defl.) -P_u -Plate buckling curves -Lateral imperfection of the steel tube walls -Buckled shapes (analy. & exp.)	-Parameters: CFT vs. HT, L/D, D/t, loading type (BS, BSU, CS) -Effect of internal restraint on local buckling behavior.
-Ends were ground flat and the top loading plate had a hemi-spherical head (conc.) -Pinned-pinned (ecc.): eccentricity provided through thick endplates with an off-set half round	-Displacement-controlled with incremental small displacements (CFTs with moderate strength concrete) -Force-controlled with slow loading near ultimate load (CFTs with high strength concrete)	-σ-ε relations for steel & concr. -P vs. ε_{axial} - (ε_{circum} & ε) vs. ε_{axial} -P_u -Residual stress distribution -M_u vs. P_u (inter. diagrams) -Lateral imperfection of the steel tube walls -Steel reduction and concrete enhancement due to confinement	-Parameters: f'_c, D/t, loading type (CS, CL, E1, E2), magnitude of eccentricity -Effect of concrete confinement on cross section strength
- Ends were ground flat and the top plate had a hemi-spherical head (conc.) -Pinned-pinned (ecc.): eccentricity provided through thick endplates with an off-set half round	-Force-controlled with slow loading near ultimate load	-σ-ε relations for steel & concr. -P vs. ε_{axial} -(ε_{circum} & ε) vs. ε_{axial} -P_u -Residual stress distribution -M_u vs. P_u (inter. diagrams) -Lateral imperfection of the steel tube walls -Steel reduction and concrete enhancement due to confinement	-Parameters: f'_c, D/t, loading type(CS,CL, E1, E2), magnitude of eccentricity -Effect of confinement on cross section strength

Article	Experiment Synopsis	Number of Tests	Tube Sizes (in) O: diam. (D) : depth (D) × width	Wall Thickness (t) (in)	Diameter/ thickness (D/t)
Shakir-Khalil, Al-Rawdan '97	Concentric & eccentric loading of full-scale rectangular CFTs	11 CFTs (conc.) 11 CFTs (ecc.)	5.91 × 3.94 (rectangular)	0.197	major axis: 30 minor axis: 20
Schneider, '98	Monotonic axial loading of circular, square and rectangular CFTs	14 CFTs	5.51 (circular) 5.00 × 5.00 (square) 5.98 × 2.99 , 5.98 × 4.02 (rectangular)	0.118-0.294	17.0-50.8
Han, Yan, '00	Monotonic axial loading of slender circular CFTs and HTs	11 CFTs 4 HTs	4.25 (circular)	0.177	24
Zhang, Zhou, '00	Monotonic axial loading of CFTs	36 CFTs	3.94 × 3.94 (square)	0.079-0.197	20-50
Han et al., '01	Monotonic loading of square CFTs	20 CFTs stub col. 8 CFTs (conc.) 21 CFTs (ecc.)	4.72 × 4.72, 5.51 × 5.51, 7.87 × 7.87 (square)	0.151, 0.231	20.5-36.5

Type of Steel, f_y (ksi) (measured)	**f_c'** (ksi)	**Length (L)** (in) (Eff. length or slenderness where noted)	**Length/ Diameter (L/D)**	**Eccen- tricity** (in)	**Residual Stresses** (ksi)	**Initial Out-of-Straight- ness** (in)
Rolled, grade 43 48.0-53.4	5.42-6.16	L_{eff} = major: 126-187 minor: 116-193 short col tests: 3.9, 5.9, 7.9	major: 21.3-31.7 minor: 29.5-49.0	e_x = 0.24, 0.59, 1.77, 2.95 e_y = 1.18	N.A.	Acknow-ledged as possible error
Cold-formed 41.3-77.9	3.45, 4.42	see L/D	4.0-4.8	N.A.	Steel tubes annealed to remove stresses	N.A.
N.A. 50.5	f_{cu} = 4.61, 6.79	138.2-163.7	32.5-38.5 (λ = 130-154)	N.A.	N.A.	N.A.
N.A. 34.8-58.5	f_{cu} = 5.87	N.A.	3-4	N.A.	N.A.	N.A.
N.A. 46.6, 47.9	1.54-5.31	see L/D	3 (stub cols.) λ= 45-75 (cols)	0-3.15	N.A.	N.A.

End Conditions	Loading Method	Results Reported	Main Test Parameters, Comments
Pinned-pinned Crossed knife edges (free biaxial rotat.) End plates (0.6 in. thick) welded to tube	-Cols tested horizontally -Equal end eccentricities -Incremental loading	-δ and axis of failure for each member -P vs. δ -P_u, P_o -P_a from BS5400, BS5940, ratios P_u/P_a	-Parameters: L/D, λ, e_x, e_y -BS5400 & BS5940 are the current British standards
Pinned-pinned through spherical bearings	-Concentric axial loading -Force-controlled	-P vs. δ_{axial} -P_y vs. λ -P_y vs. D/t -P vs. P_s -P vs. $\varepsilon_{circum}/\varepsilon$	-Parameters: D/t, cross-sectional shape -Results compared with AISC-LRFD (1994)
Pinned-pinned through loading plate with triangular wedge inserted into the grooved endplate.	-Concentric axial loading	-P vs. δ (lat. defl.) -P_a, P_u	-Parameters: CFT vs. HT, f'_c
N.A.	-Concentric axial loading	-P vs. ε -P vs. ε_{circum} -f_{cc}/f'_c vs. σ_r/f'_c	-Parameters: D/t, f_y -Degree of confinement compared with literature
-Stub cols.: Thick, stiff endplates welded to the tube -Cols.: pinned ends through loading plate with triangular wedge inserted into the grooved endplate	-Stub Cols.: Concentric loading, force-controlled -Cols: Eccentric axial loading, force-controlled	-P vs. ε_{axial} -P_u -P vs. δ (lat. defl.) -P_a vs. P_u - M_u vs. P_u (interaction diagrams)	-Parameters: ξ (stub cols.: 1.08-5.64, bmcols & cols : 1.07-3.27), f_c', D/t, magnitude of eccentricity, slenderness

Table 2. Beam, Beam-Column, and Shear Tests

(Note: studies containing column tests in addition to beam, beam-column, and/or shear tests are included in this table)

Article	**Experiment Synopsis**	**Number of Tests**	**Tube Sizes** (in.) O: diam.(D) : depth (D) $\times$ width	**Wall Thickness (t)** (in.)	**Diameter/ thickness (D/t)**
Furlong, '67	Ultimate strength of CFT bm-cols. (experimental vs. theoretical)	52 CFTs (13 col, 39 bmcol)	4.5, 5.0, 6.0 (circular) 4×4, 5×5 (square)	0.061-0.189	26.3-98.4
Furlong, '68	Design of CFT bm-cols; previous exps. examined	28 CFTs (col) (tests by others tabulated); also includes 52 CFTs from Furlong, '67	1.0-4.74 (circular)	0.064-0.465	5.6-74.4
Tomii, Sakino, '79 (a, b)	Examination of M-P-ϕ relationship for square CFTs	28 CFTs (bmcol) 8 CFTs (col)	3.94×3.94 (square)	0.164, 0.119, 0.089	24, 33, 44
Tomii, Sakino, '79 (c)	Shear behavior of square CFTs	40 CFTs (bmcol)	3.94×3.94 (square)	0.164, 0.119, 0.089	24, 33, 44
Sakino, Tomii, '81	Hysteretic behavior of beam-columns failing in flexure	15 CFTs (bmcol)	3.94×3.94 (square)	0.085, 0.088, 0.117, 0.166	24, 34, 45, 46
Sakino, Ishibashi, '85	Monotonic and hysteretic behavior of beam-columns failing in shear	21 CFTs (bmcol) (12 monotonic, 9 cyclic)	3.94×3.94 (square)	0.087, 0.117, 0.167	24, 34, 45

Type of Steel, f_y (ksi) (measured)	**f_c'** (ksi)	**Length (L)** (in) (Eff. length or slenderness where noted)	**Length/ Diameter (L/D)**	**Eccen-tricity** (in.)	**Residual Stresses** (ksi)	**Initial Out-of-Straight-ness** (in.)
Seam-weld cold-rolled 42.0-60.0 (circular) 48.0-70.3 (rect.)	3.05-5.10 (circular) 3.40-6.50 (rect.)	Approx. 36.0	5.5-12.0	constant for all tests	N.A.	N.A.
Seam-welded cold-rolled 39.6-76.0	2-5	33.9-102	8.7-40.0	N.A.	extensive stresses noted in plain tube tests	N.A.
Mild cold-worked, welded, annealed 28.7-50.2	2.7-5.5	11.8	3.0	N.A.	Tubes annealed	N.A.
Mild, cold-worked, welded, annealed 28.2-44.9	3.3-6.6	6.5-23.6	1.66-6.0 (a/D = 0.83-3.0)	N.A.	Tubes annealed	N.A.
Mild, cold-worked, welded, annealed 42.1-44.9	2.9-3.7	15.7-23.6	4.0-6.0 (a/D = 2.0-3.0)	N.A.	Tubes annealed	N.A.
Mild, cold-worked, welded, annealed 41.8-45.8	2.4-3.7	7.9-11.8	2.0-3.0 (a/D = 1.0-1.5)	N.A.	Tubes annealed	N.A.

End Conditions	Loading Method	Results Reported	Main Parameters, Comments
-Pinned-pinned -Spherical bearings	-Axial load applied incrementally -Eccentric load applied via hydraulic ram and yokes attached to each end of col. -Single curvature	-P vs. ε, P vs. ε_{concr} (cols) -P_o, P_u (cols) -P_u/P_o vs. M_u/M_o (other test plotted also) (bm-cols)	-Parameters: tube shape, D/t, f_y, f_c', A_s -Variety of results, detailed graphs
N.A.	N.A.	-σ vs. ε, exp vs. calc stiffness -P vs. ε -M vs. ϕ (bonded, unb.) -P_o, P_u -Analytical interaction diags.	-Parameters: Based on tests by other authors -Bond, residual stress -Analytical formulas presented and discussed
-Pinned-pinned -Knife-edge and spherical seat supports	-Axial load applied first -Moment applied incr. via jacks at ends -Beam-col. restrained at 3rd pts, inducing bending	-P vs. ε (D/t varied) -M vs. ϕ (D/t, P varied) -P_u/P_o vs. M_u/M_o curves	-Parameters: D/t, f_y, f_c', P/P_o -Several detailed curves -Expansive cement used
-Fixed-fixed (embed-ded in cross-beams) -Load applied through spherical seats	-Axial load -Transverse shear force applied at ends	-V vs. R (P/P_o, a/D, D/t varied) -P_o, M_u, V_{max} -P/P_o vs. M_u/M_o -ϕ, γ along length	-Parameters: P/P_o, D/t, a/D, f_y, f_c' -Very detailed V-R curves -Expansive cement used
-Fixed-fixed (embedded in cross-beams) -Load applied through spherical seats	-Axial load -Transverse cyclic shear force applied at ends (3 cycles at increments of R=0.5%)	-V vs. R hysteresis loops (P/P_o, a/D, D/t varied) - V/V_{max} vs. R (P/P_o varied) -P vs. M_u	-Parameters: P/P_o, D/t, a/D, f_y, f_c' -Very detailed V-R hysteresis loops -Expansive cement used
-Fixed-fixed (embedded in cross-beams) -Load applied through spherical seats	-Axial load -Transverse shear force applied at ends (cyclic: 3 cycles at increments of R=0.5%)	-V vs. R (P/P_o, a/D, D/t varied) -V vs. R hysteresis loops (P/P_o, a/D, D/t varied) - V/V_{max} vs. R (P/P_o varied) -P/P_o vs. V/V_{max} (exp vs. calc)	-Parameters: P/P_o, D/t, a/D, f_y, f_c' -Very detailed V-R curves and hysteresis loops -Expansive cement used

Article	Experiment Synopsis	Number of Tests	Tube Sizes (in.) O: diam.(D) : depth (D) × width	Wall Thickness (t) (in)	Diameter/ thickness (D/t)
Matsui, Tsuda, '87	Axial load & bending (cyclic & monotonic)	14 CFT (8 monotonic, 6 cyclic) 12 HT (mono.)	5.9 × 5.9 (square)	0.063-0.125	47-94
Cai, '88	7 test phases: 2 col (short, long); 5 bmcol (pure bending, applied ecc.-- single & double curvature, restrained cantilever)	phases 1-2: 93 CFTs (col) phases 3-7: 80 CFTs (bmcol)	see L/D	N.A.	N.A.
Prion, Boehme, '89	Axial, pure bending, & combination (cyclic & monotonic) Thin-walled CFTs w/ high-strength concr.	10 CFT (col) 5 CFT (bm)(1 cyclic) 11CFT (bmcol)(9 monotonic, 2 cyclic)	6.0 (circular)	0.065	92.0
Konno et al., '90	Cyclic loading of square CFT bmcols	19 CFTs (bmcol)	9.84 × 9.84 (square)	0.178-0.469	21.0-55.0
Huang et al., '91	Cyclic, lateral loading of CFTs	46 CFTs (bmcol)	3.75-6.50 (circular)	0.079-0.197	25.2-54.0 (A_s/A_c = 0.074-0.134)
Ichinohe et al., '91	Monotonic & cyclic loading of CFTs w/ high-strength steel & concrete	20 CFTs (bmcol) 11 moment-curvature tests (M-ϕ plotted) 9 shear bending tests (M-R plotted)	6.5, 11.8 (circular)	0.167-0.461	25.6-70.6

Type of Steel, f_y (ksi) (measured)	**f_c'** (ksi)	**Length (L)** (in.) (Eff. length or slenderness where noted)	**Length/ Diameter (L/D)**	**Eccen-tricity** (in.)	**Residual Stresses** (ksi)	**Initial Out-of-Straight-ness** (in.)
Mild-steel plates 51.4-71.5	4.6-6.0	29.5	5.0	N.A.	N.A.	N.A.
N.A.	N.A.	see L/D	1) <= 4.0; 2) 3-50; 3) ?; 4) 4-22; 5) 5-13; 6) 9-19; 7) 4-8	e / (concrete radius) = 0-1.28	N.A.	N.A.
Electrical-ly welded long. seam 36-48	10.6-13.3	cols: 19.7-35.4 bms: 43.3, 83.5 bmcols: 83.5	cols: 3.3-6 bms: 7.25, 14 bmcols: 14	3 bmcol tests: 0.43-0.59	N.A.	N.A.
N.A. 45.9-69.3	4.38-12.31	66.9	6.8	N.A.	N.A.	N.A.
N.A. 34.2-44.1	3.96-5.32	35.4-65.4	5.45-17.5 (λ = 22-75)	N.A.	N.A.	N.A.
Some specimens annealed 50.8-85.3	9.0-9.6	35.4, 90.6	2.0 (M-ϕ), 3.0 (M-R)	N.A.	Annealed speci-mens noted	N.A.

End Conditions	Loading Method	Results Reported	Main Parameters, Comments
Vertical cantilever: fixed base, free end	-Non-proportional -Axial load applied, then lateral load at column top	-H vs. Δ (monotonic and cyclic) -M/M_{pc} vs. θ/θ_{pc}	-Parameters: D/t, CFT vs. HT -Excellent load-defl. curves
-Phases 1-6: pinned-pinned -Phase 7: fixed cantilever	-No details of end conds. -Phase 3: pure bending; -Phases 4,5: single-curv; -Phase 6: double-curv, ratio of eccs = -1/3, -1/2, -1; -Phase 7: comb axial & latl.;	-Deflection curves (along length of col) for phase 6 -P vs ε (exp, theor.) -Global strength reduction vs. L/D, e/concr. radius -P-M interaction diagrams	-Parameters: L/D, e/(concr. radius) -Limited data -- refers to previous Chinese articles -Primarily theoretical
-Cols: fixed-fixed -Bms, bmcols: pinned-pinned	-Cols: load conc(6), both(4) -Bms: load applied at 2 pts. -Bmcols: load at 2 pts (6 mono, 2 cycl), apply eccen. (3) through spherical bearings	-P_u/P_o vs. avg. ε -M vs. ϕ, M_u/M_o vs. ϕ -P_u/P_o vs. M_u/M_o -Load ratios (exp., theor.)	-Parameters: L, loading type -Emphasis on level of ductility achieved -Compared w/ design codes
Pinned-pinned	-Transverse load applied at midpt. of bmcol -Const. axial load applied at member ends	-M_u -V vs. R	-Parameters: D/t, f'_c, f_y, P/P_o -Compared w/ proposed design equations.
-Base fixed, top fixed to movable frame -Details sketchy	-Lateral load applied via a frame fixed to top of beam	-H vs. δ -Hysteretic loops (H vs. δ) -Ductility ratio (2 spcms.) -Absorbed energy (2 spcms.)	-Parameters: λ, A_s/A_c, P/P_o -Monotonic and cyclic loading
Both pinned-pinned	-Both tests axially loaded -M-ϕ test: loaded at 35.4 in. from each end, s-s beam -Shear bending test: loaded at midpt; bmcol at load pt. considered fixed	-σ vs. ε (exp. & calc.) -Biaxial stresses in tubes -M vs. ϕ, M vs. R (monotonic, cyclic, exp. vs. theoretical)	-Parameters: P/P_o for given cyclical loading, D/t

Article	Experiment Synopsis	Number of Tests	Tube Sizes (in.) O: diam.(D) : depth (D) × width	Wall Thickness (t) (in)	Diameter/ thickness (D/t)
Sato et al., '91	Reversed cyclic shear loading of circular CFTs	3 CFTs	5.91 (circular)	0.315	18.75
Sugano et al., '92	Cyclic loading of square and circular beam-columns	19 circular CFTs 20 square CFTs	11.8 (circular) 9.84 × 9.84 (square)	0.157-0.472	25-75 (circular) 20.8-62.5 (square)
Kawaguchi et al., '93	Cyclic loading of cantilever CFT beam-columns	14 CFTs (bmcol) 12 HTs (bmcol)	3.94 × 3.94 (square)	0.118, 0.177	22.2, 33.3
Lu, Kennedy, '94	Monotonic uniaxial loading (pure bending, simple supports) of rectangular CFTs	12 CFTs (bm) 5 HTs (bm) 5 HT stub col. 5 CFT stub col.	6.0 × 6.0 (square) 10.0 × 6.0 (rectangular)	0.189, 0.252, 0.347	16, 23.8, 26.7, 31.6, 39.6
Sakino, '95	Monotonic loading of circular CFT beam-columns	28 CFTs (bmcol)	4.25-17.72 (circular)	0.117, 0.179, 0.255	26.9-152.0

Type of Steel, f_y (ksi) (measured)	**f_c'** (ksi)	**Length (L)** (in.) (Eff. length or slenderness where noted)	**Length/ Diameter (L/D)**	**Eccen-tricity** (in.)	**Residual Stresses** (ksi)	**Initial Out-of-Straight-ness** (in.)
Annealed 55.8	5.15	43.3	7.33	N.A.	Annealed	N.A.
N.A. 47.9-72.1	4.5-12.8	78.7 (circular) 66.9 (square)	6.67 (circular) 6.8 (square)	N.A.	N.A.	N.A.
Cold-formed 49.2	3.1-3.6	39.3 (1 m)	10	N.A.	N.A.	N.A.
Cold-formed 50	5.87-6.83	77.75-167.72	8.78-20	Pure bending	N.A.	N.A.
59.2, 93.7 ,127.5	3.55, 5.79, 11.26	12.75-53.15	3	N.A.	N.A.	N.A.

End Conditions	Loading Method	Results Reported	Main Parameters, Comments
-Pinned-pinned (pin & roller) -Axial load applied thru plates welded to ends of specimen	-Rigid rectangular frame -Reversed cyclic loading applied to rigid stub welded to column at mid-height from frame	-σ vs. ε for varying ecc. dist. -P vs. δ -Eccentric distance (as calc. in paper) vs. δ -V vs. R, P vs. R	-Main parameter: P/P_o for given cyclic loading
Pinned transverse supports	-Axial load applied concentrically -Lateral load applied transversely at bmcol midpt.	-R (max) vs. P/P_o -Hysteresis loops: V vs. R (1 circular and 1 square test) -P_u vs. M_u	-Parameters: P/P_o, D/t, f_y, f'_c -Very detailed plots, but only for selected sections
Fixed base, pinned top (roller)	-L-shaped load frame -Lateral load applied to column top from frame	-σ vs. ε -H vs. ε -Hysteresis loops (H vs. δ) (deflection at column top) -P_u deterioration (P_u vs. cycle) -Energy dissipation (hysteresis loop area vs. loading cycle	-Parameters: hollow vs. filled, P -Alternately repeated lateral load w/ constant axial load -Failure: local buckling at base
Simply supported beam with load applied symmetrically at two points. Stiffeners under loads	N.A.	-σ vs. ε (stub col.'s) -P vs. ε (HTs, conc., CFTs) -M vs. ϕ (exp. vs. theory) -M vs. slip -N.A. movement vs. M	-Parameters: D/t, L/d, a/D -Slip measured and reported -Discussion of slip -Pure bending behavior and discussion of rigidity
Two loading plates welded to the ends	-Constant axial load, monotically increasing end moments	-M vs. ϕ -M_u vs. P_u (interaction diagrams) -P/P_o, M_u -M_u/M_{pc} vs. D/t	-Parameters: D/t, f_y, axial load ratio P/P_o, f'_c

Article	Experiment Synopsis	Number of Tests	Tube Sizes (in.) O: diam.(D) : depth (D) × width	Wall Thickness (t) (in)	Diameter/ thickness (D/t)
Fujimoto et al., '96	Cyclic load-deformation and ultimate strength of CFT beam-columns with variable axial load	13 Circular CFT 20 Square CFT	9.45, 6.3 (circular) 8.27 × 8.27, 7.09 × 7.09 (square)	0.177, 0.235, 0.354	Circular 17.8-53.3 Square 20.0-53.3
Tsuda et al., '96	Series II - Slender Cantilever CFT's subjected to cyclic horizontal load while under constant axial load	10 Circular CFT's 10 Square CFT's	6.51 (circular) 5.93 × 5.93 (square)	0.165 0.172	39.6 34.5
El-Remaily et al., '97	Cyclic loading of circular CFTs w/ high strength concrete	4 CFTs (cyclic)	12.01 (circular)	0.252, 0.374	32, 48
Zhang, Shahrooz, '97	Monotonic loading of square CFT beam-columns at a horizontal position	2 CFTs (bmcol)	10 × 10 (square)	0.313	32.0
Nakahara, Sakino, '98	Monotonic loading of square hollow tubes and square CFTs w/ high strength concrete and steel	4 HT stub col. 4 CFT stub col. 10 CFTs (bmcol)	7.87 × 7.87 (square)	0.122-0.252	30, 60

Type of Steel, f_y (ksi) (measured)	**f_c'** (ksi)	**Length (L)** (in) (Eff. length or slenderness where noted)	**Length/ Diameter (L/D)**	**Eccen-tricity** (in)	**Residual Stresses** (ksi)	**Initial Out-of-Straight-ness** (in)
Cold-formed 58.0, 85.6, 113.1	5.80, 13.05	37.8-56.7	6	N.A.	N.A.	N.A.
Mild steel; STK 400 51.5 STKR 400 57.5	5.04	All noted are kL; 39.10-156.28 35.6-142.3	6, 9, 12, 18, 24 6, 9, 12, 18, 24	N.A.	N.A.	N.A.
54	10, 15	110.83	7.16	N.A.	N.A.	N.A.
Cold-formed 53.7	6.05	143.98	14.4	N.A.	N.A.	N.A.
Cold-formed channel sections 45.0, 113.3	17.26	23.62	3	N.A.	N.A.	N.A.

End Conditions	Loading Method	Results Reported	Main Parameters, Comments
Fixed CFT columns attached to CFT stubs at ends to guarantee sufficient stiffness	- Both constant and variable axial load on columns -Lateral load applied on top CFT stub by hydraulic jack	-M vs. R (per conc. type) -R vs. ε (per conc. type) -Interaction diagrams (per conc. type) All graphs for both variable and constant axial load	-Parameters: Tube shape, f_u, f_c', D/t, P/P_o, loading angle (biaxial bending) -Maximum strength and rotation capacity were smaller in the variable axial load specimens
Col: Fixed base, unspecified connection at top (poss. free or pin)	-Axial load: Applied by a testing machine and kept constant throughout -Horiz. Load: A hydraulic jack kept the top of the col. fixed while the frame on which it was mounted moved side-to-side	-V vs. δ -M_u vs. P_u (interaction diagrams)	-Parameters: Axial load ratio P/P_o, and buckling length-section depth ratio (Lk/D)
Pinned ends, rigid stub at the midheight	-Horizontally placed specimens -Constant axial load and cyclically applied lateral load at the mid-height -Displacement controlled loading	-H vs. δ (lat. defl.) -H vs. δ_{axial} -M_u	-Parameters: D/t, axial load ratio P/P_o, f'_c
Pinned ends through cylindrical bearings	-Constant axial load, monotically increasing point loads applied at two points along the specimen length	-σ-ε relation for steel -M vs. ϕ -P vs. δ (vert. defl.at midspan) -M vs. θ -Strain distribution over depth	-Parameters: Axial load ratio P/P_o
-Cols: Two loading plates welded to the ends -Bmcols: Two thick loading plates welded to the ends, two trapezoidal plates welded to the tension face at the ends	-Cols: Concentric loading -Bmcols: Constant axial load, monotonically increasing end moments, curvature controlled loading	-P vs. ε_{axial} -M vs. ϕ -M_u vs. P_u (interaction diagrams) - P_u, M_u	-Parameters: D/t, f_y, axial load ratio P/P_o

Article	Experiment Synopsis	Number of Tests	Tube Sizes (in.) O: diam.(D) : depth (D) × width	Wall Thickness (t) (in)	Diameter/ thickness (D/t)
Nakahara, Sakino, '00	Monotonic and cyclic loading of square CFT bmcols	6 CFTs (bmcol) (monotonic) 5 CFTs (bmcol) (cyclic)	7.87 × 7.87 (square)	0.080, 0.167, 0.233	33.7, 47.1, 98.0
Varma et al., '00, 01	Monotonic and cyclic loading of square CFTs w/ high-strength steel & concrete	4 CFTs stub col. 8 CFTs (bmcol) (monotonic) 8 CFTs (bmcol) (cyclic)	12.01 × 12.01 (square)	0.230-0.350	34.3-52.2
Elchalakani et al., '01	Uniaxial flexural loading (pure bending) of circular CFT beams	12 CFTs	1.33-4.37 (circular)	0.039-0.132	12.8-109.9

Type of Steel, f_y (ksi) (measured)	**f_c'** (ksi)	**Length (L)** (in) (Eff. length or slenderness where noted)	**Length/ Diameter (L/D)**	**Eccen-tricity** (in)	**Residual Stresses** (ksi)	**Initial Out-of-Straight-ness** (in)
30.6, 36.7, 46.4	6.90	23.62	3	N.A.	Annealed	N.A.
A500 Grade B 37.6, 68.3 A500 Grade 80 81.2, 95.7	4.33	48.03, 58.50, 60.00	4.00, 5.00, 4.87	N.A.	N.A.	N.A.
Cold-formed 52.9-66.7	3.39	23.62, 31.49	5.41-23.68	N.A.	N.A.	N.A.

End Conditions	Loading Method	Results Reported	Main Parameters, Comments
Two loading plates welded to the ends	-Bmcols(m): Constant axial load, displacement controlled bending moment applied at the ends -Bmcols(c): Constant axial load, curvature controlled bending moment applied at the ends	-σ-ε relations for steel & concr. (analytical) -M vs. ϕ (experimental & analytical) -ϕ vs. ε_{axial}	-Parameters: D/t, axial load ratio P/P_o, deformation histories(m, c)
-Col: Fixed -Bmcol (m): Pinned ends through cylindrical bearings -Bmcol (c): Fixed at base, not specified at top	-Cols: Concentric loading, force controlled until failure, displacement controlled after failure -Bmcols (m): Constant axial load, monotically increasing end rotations -Bmcols (c): Constant axial load, cyclically applied lateral load at the top	-σ-ε relations for steel & concr. -P vs. δ_{axial} -P_u, M_u -M vs. ϕ & M vs. θ -H vs. δ (lat. defl.) -μ vs. D/t, f_y , P/ P_u -W vs. D/t, f_y , P/ P_u -EI / EI_s vs δ_{axial} / δ_y -δ_{axial} vs. δ (lat. defl.) -M_u vs. P_u (interaction diagrams)	-Parameters: Axial load ratio P/P_o, D/t, type of steel
Attached to rotational fixtures at each end	-Pure bending through applied rotation at both ends -Bending was applied through coupling forces acting on the pinned points at each end	-M_u, R_{max}, R_{cm}, θ_{pc} -M vs. ϕ	-Parameters: D/t, L/D

Table 3. Frame Tests

Article	Experiment Synopsis	Number of Tests	Connection Type	Beam Sizes	Type of Steel, f_{yb} (girder) (ksi)
Matsui, '86	Monotonic and cyclic loading of portal frames having CFT and HT columns	2 HT frames, 10 CFT frames	Outside stiffener, Through stiffener	BH200 × 200 × 6 × 6	38.7, 51.8
Morino et al., '93	Non-proportional cyclic loading of 3D CFT/steel cruciform subassemblies	10 subassemblies (6 w/ sym out-of-plane loads, 4 asym) 5--connection failure 5--column failure	Through stiffener	H250 × 250 × 6 × 9	N.A.
Kawaguchi et al., '02	Cyclic loading of portal frames made up of CFT columns	4 CFT frames	Through stiffener	BH125 × 150 × 16 × 25	SS400 49.68

Length of girder (in)	Tube Sizes (in) O: diam.(D) □: depth (D) × width	Wall Thickness (t) (in)	Diameter/ thickness (D/t)	Type of Steel, f_y (ksi) (measured)	f_c' (ksi)	Length (L) (in) (Eff. length or slenderness where noted)
59.1	5.91 × 5.91 (square)	0.177, 0.126, 0.091	33, 47, 68	Mild, cold formed channel sections 59.8, 41.69, 41.83	5.2-5.8	39.4
Bms (in-plane): 70.87 Bms (out-plane): 49.21	4.92 × 4.92 (square)	0.23	22.0	STKR400 steel (Jap.) 57.3	2.8-3.0	39.67
59.1	4.92 x 4.92 (square)	0.236	20.83	STKR400 58.50	2.7	39.4

Length/ Diameter (L/D)	Eccentricity (in)	Residual Stresses	End Conditions	Loading Method
6.67	N.A.	N.A.	Col: Fixed base Beam to col: Rigid	Constant axial load on both columns and monotonic or cyclic lateral load at one beam-to-column connection
8.1	N.A.	N.A.	Fixed base, pinned top (roller)	-Constant axial load on col. -Sym or asym. const loads on 2 out-of-plane bms. -Anti-symmetric cyclic loading on 2 in-plane bms
8	N.A.	N.A.	Col: Fixed base Beam to col: Rigid	Constant axial load on both columns and cyclic lateral load applied to the base-beam of the columns,

Results Reported	Main Parameters, Comments
- H vs. Δ - H_{max}, M_u , M_{pc}	- Parameters: Type of connection, D/t - Cyclic vs. monotonic loading
-V vs. ε -V vs. R -V vs. lat'l displ. of connection	-Parameters: conn. design, out-of-plane loading -Biaxial bending of CFT -Failure: typ. instability due to lat'l displ of connection
- H vs. R - H vs. ε_d	- Parameters: Failure mode, axial load ratio (P/P_o)

Table 4. Torsion Tests

(Note: includes tests w/ both torsional and concentric loading or a combination thereof)

Article	**Experiment Synopsis**	**Number of Tests**	**Tube Sizes** (in.) O: diam. (*D*) □: depth (*D*) × width	**Wall Thickness (*t*)** (in)	**Diameter/ thickness (*D/t*)**
Lee et al., '91	Monotonic & cyclic studies of CFTs under compression & torsion	47 CFTs (col, monotonic) 6 CFTs (col, cyclic)	4.0, 4.5 (circular)	0.063, 0.177	64, 25.3
Xu et al., '91	Tests of medium & long CFTs under compression and torsion	27 CFTs (3 pure compr., 3 pure torsion)	3.54, 4.49 (circular)	0.157, 0.177	22.5, 25.4

Type of Steel, f_y (ksi) (measured)	**f_c'** (ksi)	**Length (*L*)** (in) (Eff. length or slenderness where noted)	**Length/ Diameter (L/D)**	**Residual Stresses**	**Initial Out-of-Straight-ness** (in)
N.A. 35.1, 40.6	4.77, 3.97	16.0, 15.2	4, 3.4	N.A.	N.A.
Seamless steel tube 44.6	3.0, 3.2	see *L/D*	7, 13, 20	N.A.	N.A.

End Conditions	**Results Reported**	**Main Test Parameters, Comments**
N.A.	-*T* vs. P/P_o (monotonic) -*T* vs. θ (monotonic & cyclic)	-Limited description of experiment
N.A.	-*P* vs. ε -*T* vs. θ -*T* vs. P/P_o	-Parameters: *L/D*, α

Table 5. Connection Tests

Article	**Experiment Synopsis**	**Number of Tests**	**Tube Sizes** (in.) O: diam. (D) : depth (D) × width	**Wall Thickness (t)** (in.)	**Diameter/ thickness** D/t
Dunberry et al., '87	Axial loading of square stub columns through shear connections	18 CFTs	4.0-8.0 (square)	0.187-0.249	20.8-37.1
Kamba et al., '91	Shear tests of through diaphragm connections to square CFT and HT columns	5 CFTs 5 HTs	8.52 × 8.52 (square)	0.177-0.315 (panel zone-web & flange) 0.315, 0.500 (column-web & flange)	27, 36, 48 (panel zone) 17, 27 (column)
Prion, McLellan, '92	Through-bolt connections between steel wide-flange girders and CFTs	8 CFTs	12 × 12, 8 × 8 (square)	0.472	25
Azizinamini, Prakash, '92	Pass-through girder with internal stiffeners	1 CFT	24 × 24 (square)	0.5	48
Shakir-Khalil, Mahmoud, '95	Full-scale simple interior beam-to-column connections of circular and square CFTs	28 CFTs	6.63, 8.63 (circular) 5.91 × 5.91, 7.87 × 7.87 (square)	0.197, 0.248	30.0-34.8

Type of Steel, f_y (ksi) (measured)	f'_c (ksi)	Beam size	Beam f_y (ksi)	Plate & Stiffener Description	Plate & Stiffener fy (ksi)
Class H Grade 50W 51.3-64.3	2.52-4.29	N.A.	N.A.	-Standard tee: 14.2 × (0.31 or 0.47) gusset plate -Extended tee: 22.83 × 0.31 gusset plate -Shortened tee: 8.66 × 0.31 gusset plate - Single plate had only web without any gusset plate	N.A.
STK400 48.4, 52.6 (annealed) 54.07-56.92 (cold-formed)	3.37, 3.56	N.A.	N.A.	-Through diaphragm plates manufactured from PL25	N.A.
50.8	6.53	N.A.	N.A.	N.A.	N.A.
36.1	14.07	W30 × 99	N.A.	-Four #11 reinforcing bars with 4 × 2 × 1 in. plates welded to the ends were attached to the girder inside the CFT	60.05
44.0-60.2	4.25-5.90	406 × 178 × 67 UB	N.A.	-10.24 or 14.17 in. long and 3.94 × 0.39 in. fin-plates -14.17 in. long tee cleats manufactured from 305 × 127 × 48 UB type beam -0.146 in. diameter 2.44 in. long Hilti nails	N.A.

Residual Stress (ksi)	**End Conditions**	**Loading Synopsis**	**Results Reported**	**Main Parameters, Comments**
N.A.	N.A.	-Axial load applied at the top and at the connections simultaneously	-P vs. ε_s, ε_{concr} -L vs. d_s (slip) -L vs. P_c, P_s -θ_c vs.P	-Parameters: Connection detail, connection load / total axial load, D/t, end conditions (capped, uncapped)
Only joint region annealed or no annealing	Pinned-pinned (column)	-Shear couples acting on the diaphragm plates	-σ - ε relations for steel - V_u, V_y, K_o - V_{cu}, V_{cy}, K_{co} - V vs. γ - τ_c vs. γ	-Parameters: D/t (column and panel zone), CFT vs. HT, annealing of steel tube
N.A.	N.A.	-Tension pull-out and compression push-in tests	- Concrete and steel stress-strain relationships; - Bolt slip-load and load-strain relationships	-Parameters: Post-tensioning of the bolts in the connection
N.A.	N.A.	-Shear forces were applied to the girder ends in opposite directions	N.A.	N.A.
N.A.	Pinned-pinned (column)	-Symmetric loading of the beams and axial loading of the column, simultaneously. - Beam-to-column load ratios were 1:8 or 1:5	-M vs. θ_c - K_o, P_u -P vs. ε_s	-Parameters: Fin-plate vs. tee cleat, D, t, shear connector, beam load to column load ratio, eccentricity of beam load

Author	Experiment Synopsis	Number of Tests	**Tube Sizes** (in.) O: diam. (D) : depth (D) × width	**Wall Thickness (t)** (in)	**Diameter/ thickness** D/t
Shakir-Khalil, Al-Rawdan, '95	Monotonic tests for full-scale simple exterior beam-to-column connections of square CFTs	4 CFTs	5.91 × 5.91 (square)	0.197	30
Shakir-Khalil, Al-Rawdan, '96	Monotonic tests for full-scale simple interior beam-to-column connections of square CFTs	4 CFTs	5.91 × 5.91 (square)	0.197	30
Kawano, Matsui, '97	Simple tension tests on CFT and HT connections and cyclic tests on cruciform frames including both CFTs and HTs	6 CFTs, 9 HTs (simple tension) 9 CFTs, 11 HTs (cruciform frame)	5.91 × 5.91 (square)	0.177 (simple tension) 0.166, 0.171 (cruciform frame)	33.3 (simple tension) 34.5, 35.5 (cruciform frame)
Schneider, Alostaz, '98	Cyclic testing of 2/3 scale beam-to-column connections of circular CFTs	6 CFTs	14 (circular)	0.269	52.0

Type of Steel, f_y (ksi) (measured)	**f'$_c$** (ksi)	**Beam size**	**Beam f_y** (ksi)	**Plate & Stiffener Description**	**Plate & Stiffener f_y** (ksi)
47.9, 48.2	4.54-4.98	406 × 178 × 67 UB	N.A.	-14.17 in. long tee cleats manufactured from 305 × 127 × 48 UB type beam	N.A.
49.3, 49.8	4.29-4.84	406 × 178 × 67 UB	N.A.	-14.17 in. long tee cleats manufactured from 305 × 127 × 48 UB type beam	N.A.
Cold formed STKR 400 62.8	4.97, 5.19	0.177 in. flange (simple tension) H-200 × 100 × 3.2 × 4.5, H-250 × 1000 × 6.0 × 6.0 (cruciform frame)	44.39 (simple tension) 43.11-55.35 (cruciform frames)	-0.177 in. thick external diaphragms and vertical stiffeners manufactured from SS400 plates	44.39
57.5	7.8-8.2	W14 x 38	Girder 44.3 (flange) 52.0 (web) Connection stub 48.2 (flange) 39.4 (web)	-Connection stub with flared flange -Connection stub web and/or flange continued into the column -External diaphragm plate -Deformed bars welded to the connection stub flange	N.A.

Residual Stress (ksi)	**End Conditions**	**Loading Synopsis**	**Results Reported**	**Main Parameters, Comments**
N.A.	Pinned-pinned (column)	- Eccentric loading of the beam and axial loading of the column, simultaneously. - Beam-to-column load ratios were 1:8 or 1:5	-M vs. θ_c -K_o, P_u -Longitudinal strain distribution along the column length	-Parameters: Beam load to column load ratio, eccentricity of beam load
N.A.	Pinned-pinned (column)	-Symmetric loading of the beams and axial loading of the column, simultaneously -Beam-to-column load ratios were 1:3, 1:5 or 1:8	-M vs. θ_c -K_o, P_u -P vs. in-plane displacement along the column length	-Parameters: Beam load-to-column load ratio, eccentricity of beam load
N.A.	Pinned-pinned (columns in cruciform frames)	- Tensile loading until fracture through flanges (simple tension test) - Constant axial load on to the columns and cyclic shear force at the girder ends (cruciform frames)	-P_u, P_a, P_u/P_a -V_u/V_c -V vs. R	-Parameters: CFT vs. HT, connection type, size of diaphragm and vertical stiffener
N.A.	-Roller at the top and pinned at the bottom (column)	-Constant axial load on to the column and cyclic shear load at the girder end	-M vs. θ_c	-Parameters: Connection details

Author	**Experiment Synopsis**	**Number of Tests**	**Tube Sizes** (in.) O: diam. (D) : depth (D) × width	**Wall Thickness (t)** (in)	**Diameter/ thickness** D/t
France et al. '99 a, b, c	Cyclic and monotonic loading of beam-to-column flow-drill connections of square CFTs and HTs	20 HTs 6 CFTs	29 × 29 (square)	0.248-0.492	31.8-9.7
Cheng et al., '00	Cyclic loading of beam-to-column connections of circular CFTs	3 CFTs 3 HTs	15.75 (square)	0.236, 0.394	40, 66.7
Fujimoto, T., et al., '00	Cyclic loading of interior (2D and 3D) and exterior CFT beam-to-column connections manufactured from high strength materials	11 CFTs	9.84 × 9.84, 6.30 × 6.30 (square) 7.09 × 7.09, 11.02 × 11.02 (circular)	0.121-0.186 (square panel zone web) 0.355-0.480 (square column flange) 0.122-0.188 (circular panel zone) 0.357-0.484 (circular column)	51.9-54.6 (square panel zone web) 17.7, 20.5 (square column flange) 58.3-60.3 (circular panel zone) 19.8-22.8 (circular column)
Peng, et al., '00	Cyclic tests of full scale moment connections to square CFTs	11 CFTs	8 × 8, 16 × 16 (square)	0.492	32.5

Type of Steel, f_y (ksi) (measured)	f'$_c$ (ksi)	Beam size	Beam f_y (ksi)	Plate & Stiffener Description	Plate & Stiffener f_y (ksi)
Hot-rolled Grade 43 44.5-61.9	6.29, 7.32	457 × 152 × 52 UB 356 × 171 × 45 UB 254 × 146 × 31 UB	Grade 43	-Partial depth end plate (smaller than the beam sections) -Flush end plate (same size with the girder section) -Extended end plate (larger size than the girder section)	Grade 43
Cold-formed 45.5, 56.9	3.77, 3.92	H-600 × 200 × 11 × 17	44.67 (flange) 47.57 (web)	-Through diaphragm plates with 3.94 in. opening at the middle -External diaphragms	N.A.
Welded channel sections 71.4-109.7 (square panel zone web) 63.8-110.1 (square column flange) Cold formed 63.7-108.9 (circular panel zone) 64.1-110.7 (circular column)	7.89-15.95 (square) 7.12-14.45 (circular)	H-250 × 250 × 9.04, 9.16 × 12.0, 12.1, H-160 × 160 × 11.9 × 16.2 (square) H-250 × 250 × 9.04, 9.16 × 11.9, 12.1, H-160 × 160 × 11.9 × 16.2 (circular)	63.23-107.19 (square) 64.98-10719 (circular)	-Through diaphragms and external diaphragms	N.A.
ASTM A500 Grade B 54.68-59.76	5.66-6.24	W24 × 62	51.77-52.22	-0.63 in. thick tapered plates -0.47 in. thick extended tees from the girder flange to the steel tube corner -Tees cut from W24 × 146	-40.0 (extended tee) -62.1 (tee)

Residual Stress (ksi)	**End Conditions**	**Loading Synopsis**	**Results Reported**	**Main Parameters, Comments**
N.A.	-Pinned-pinned with fully-restraint or simple beam-to-column connections at the middle	-Monotonic or cyclic shear force applied at the girder tips with either constant or no axial load acting on the columns	- M vs. θ, M_u, θ_u	-Parameters: CFT vs. HT, connection type (simple or fully restraint), beam size, t, P, bolt spacing, endplate type, f_y
N.A.	Pinned-pinned (column)	- Constant axial load on to the columns and cyclic shear load at the girder ends	-V vs. girder-end deflection -Contribution of girder, column, and panel zone to the total deformation	-Parameters: CFT vs. HT, D/t, connection type
N.A.	N.A.	-Constant axial load on to the column and reversed cyclic load at the girder ends (interior connections) -Variable axial load on to the column and reversed cyclic shear load at the girder ends (exterior connections)	-V_u for column, connection and girder -V vs. R	-Parameters: Diaphragm type, 2D vs. 3D connection, interior vs. exterior connection, f_y, f'_c, D, t, D/t
N.A.	N.A.	-Constant axial load and cyclic lateral load applied to the top of the columns simultaneously	- M vs. θ - V_u, V_s, V_p - K_o, θ_u	-Parameters: Use of diaphragms, connection details, design approach(panel zone yielding vs. girder yielding)

Table 6. Bond Tests

Article	Experiment Synopsis	Number of Tests	Tube Sizes (in) O: diam. (*D*) : depth (*D*) × width	Wall Thickness (t) (in)	Diameter/ thickness (D/t)
Morishita et al., '79 a, b	Push-out tests of circular, square and octagonal CFTs	28 (circular) 28 (square) 4 (octagonal)	5.91, 6.51 (circular) 5.91 × 5.91 (square, octagonal)	0.126, 0.142 (circular) 0.126, 0.169 (square, octagonal)	45.9, 46.9 (circular) 34.9, 46.9 (square)
Tomii et al., '80 a, b	Push-out tests of circular, square and octagonal CFTs	28 (circular) 28 (square) 28 (octagonal)	5.91 (circular) 5.91 × 5.91 (square and octagonal)	0.126	46.9
Virdi, Dowling, '80	Push-out tests of circular CFTs	91 (circular)	5.84-12.05	0.22-0.40	14.8-32.3
Morishita, Tomii, '82	Cyclic push-out tests of square CFTs	24 (square)	5.88 × 5.88	0.166-0.170	35
Shakir-Khalil, '93a	Push-out tests of square and circular CFTs	30 (square) 26 (circular)	5.91 × 5.91, 7.87 × 7.87 (square) 6.63, 8.63 (circular)	0.197, 0.248	30.0, 31.7 (square) 33.6, 34.8 (circular)

Type of Steel, f_v (ksi) (measured)	**f_c'** (ksi)	**Length (L)** (in)	**Interface Length** (in)	**Type of Shear Connector**	**Number of Shear Connector(s) and Location (from top)** (#)-in	**Length of Shear Connector** (in)
Cold-formed mild steel 36.6, 37.0 (circular) 36.7, 37.8 (square, octagonal)	2.76-4.85 (circular) 2.73-4.81 (square) 4.85 (octagonal)	29.53	29.53	N.A.	N.A.	N.A.
Cold-formed mild steel 36.6 (smooth) 36.3 (checkered)	circular 2.42, 3.26 (expansive) 2.89, 3.91 (ordinary) square & octahedral 2.46-6.69 (expansive) 3.06-4.92 (ordinary)	28.94	28.94	Checkered surface	N.A.	N.A.
Mild steel	3.19-6.72	5.88-18.25	4.38-16.75	N.A.	N.A.	N.A.
Cold-formed mild steel 49.2 – 50.9	3.32, 3.86, 5.08	35	29	N.A.	N.A.	N.A.
Mild steel Grade 43	5.22-6.16	17.7	15.8	Hilti nails, Grade 4.6 Black bolts	(6)-3.94, 7.87, 11.81; (12)-3.94, 7.87, 11.81; (2)-7.87; (4)-7.87; (4)-3.94; (8)-3.94, 7.87	2.44 (Hilti nails) 1.97 (Black bolts)

Diameter of Shear Connector (in)	**Residual Stress**	**Bond Strength (f_b)** (psi)	**Loading Method**	**Results Reported**	**Main Test Parameters, Comments**
N.A.	Annealed	28.5-56.9 (circular) 21.3-42.7 (square and octagonal)	-Axial load applied at the top on the steel tube alone and the columns supported on both steel and concrete at the bottom	-L vs. ε_s & f_{bs} vs. d_s -Distribution of longitudinal stress difference of steel along the column	Parameters: f'_c, cross-section shape
N.A.	Annealed	56.9-85.4 (checkered) 28.5-56.9 (smooth) (circular) 56.9-71.1 (checkered) 21.3-42.7 (smooth) (sqr.&oct.)	-Axial load applied at the top on the steel tube alone and the columns supported on both the steel and concrete at the bottom	-L vs. ε_s -f_{bs} vs. d_s -Distribution of longitudinal stress difference of steel along the column	Parameters: Smooth surface vs. checkered surface, expansive concrete vs. ordinary concrete, f'_c
N.A.	N.A.	48.1-433.8 (avg. values)	-Axial load applied at the top on the concrete alone and the column supported on the steel tube at the bottom	-f_b, f_{bs} vs. d_s	Parameters: Age of concrete, f'_c, L, D, compaction of concrete, surface treatment
N.A.	Annealed	21.3-49.8	-Constant axial load applied with cyclic shear force at the top -Axial load applied either to the conc. and steel tube simult. or only to the steel tube -Cols. supported on the steel tube and concrete simult.	-H vs. R -L vs. ε_s & P -f_{bs} vs. d_s	Parameters: P, f'_c, method of applying axial load at the top of the column (steel tube alone or steel tube and concrete simult.)
0.146 (Hilti nails) M12 black bolt	N.A.	63.8, 120.4	-Axial loading on to the concrete at the top -Axial loading through steel brackets or plates attached to the steel tube -Columns supported on the steel tube alone at the bottom	-P vs. d_s -P vs. ε_s -P_u, f_b, d_s	Parameters: Tube shape, type and number of shear connectors, type of loading

Article	Experiment Synopsis	Number of Tests	Tube Sizes (in) O: diam. (D) : depth (D) × width	Wall Thickness (t) (in)	Diameter/ thickness (D/t)
Shakir-Khalil, '93b	Push-out tests of rectangular, square and, circular CFTs with dry or oiled interface	16 (rectangular) 12 (square) 12 (circular)	4.72 × 3.15 (rectangular) 5.91 (square) 6.63 (circular)	0.197	24 (rectangular) 30 (square) 33.7 (circular)
Shakir-Khalil, Hassan, '94	Push-out tests of rectangular CFTs	52 (rectangular)	5.91 × 3.94	0.197	30
Kilparick, Rangan, '99	Bond tests of circular CFT columns and beams having various interface conditions including maximum bond, partial bond and minimum bond. HT tubes were also tested under axial load	3 CFTs, 1 HTs (stub columns) 6 CFTs, 2 HTs (short and slender columns) 4CFTs, 1 HT (beam)	4	0.094	42.3
Roeder, Cameron, '99	Push-out tests of circular CFTs filled with concrete having moderate or little shrinkage potential	20	10.3-23.8 (circular)	0.28-0.53	19.4-108.0

Type of Steel, f_v (ksi) (measured)	**f_c'** (ksi)	**Length (L)** (in)	**Interface Length** (in)	**Type of Shear Connector**	**Number of Shear Connector(s) and Location (from top)** (#)-in	**Length of Shear Connector** (in)
Mild steel Grade 43	5.50-6.43	9.8, 17.7, 25.6	7.9, 15.7, 23.6	Grade 4.6 Black bolts	(2)-7.87; (4)-3.94, 11.81; (6)-3.94, 8.86, 13.78	1.97
N.A.	7.25 (series A) 6.09, 6.67 (series B) 3.19-8.99 (series C) 4.06-12.18 (series D) 7.11-8.41 (series E) 6.38-7.83 (series F)	17.72	15.74	Grade 4.6 Black bolts, Grade 8.8 Black bolts, threaded bars	-Black bolts: (2)-7.87; (4)-7.07, 8.68; (4)-6.67, 9.07 -Threaded bars (1)-7.87; (1)-N.A.; (2)-N.A.	N.A.
Cold-formed 50.8	15.30	13.78 (stub col.) 40.83 (short col.) 76.85 (slender col.) 82.68 (beam)	13.78 (stub col.) 40.83 (short col.) 76.85 (slender col.) 82.68 (beam)	Self-tapping screws	Double helix pattern with a pitch length of 3.15 in.	1.18
N.A.	4.05-6.86	46	41.5	N.A.	N.A.	N.A.

Diameter of Shear Connector (in)	**Residual Stress**	**Bond Strength (f_b)** (psi)	**Loading Method**	**Results Reported**	**Main Test Parameters, Comments**
M12 black bolt	N.A.	29.0-580.2	-Monotonic or repeated axial load applied on the concrete concentrically -Columns supported on the steel tube alone at the bottom	-P vs. d_s -P vs. ε_s -P_u, f_b, d_s	Parameters: Tube shape, interface condition, interface length, loading type, number and location of shear connectors
0.630 (black Bolt) N.A. (threaded bar)	N.A.	28.4-77.3	-Monotonic axial loading of the concrete alone at the top while the column supported on the steel tube at the bottom	-P vs. d_s -P_u, f_b, d_s	Parameters: Shear connector type, number and location of shear connectors, f'_c
0.193	N.A.	1.5-114.0	-Axial loading applied to the steel and concrete simultaneously -Columns supported on the steel and concrete simultaneously at the bottom	-P vs. ε_s -P vs. δ	Parameters: Interface condition, L/D
N.A.	N.A.	N.A.	-Concentric, eccentric and cyclic concentric load applied on the concrete alone -Columns supported on the steel tube at the bottom	-f_b vs. L -f_b vs. D -f_b vs. D/t -P vs. d_s -P_u, f_b	Parameters: D, t, shrinkage strain of concrete

IV. Categorization of Beam-Column Tests

The tables on the following pages categorize individual beam-column tests frame the papers summarized above based on salient test parameters. The beam-column tests are separated into circular and rectangular and then subdivided into three categories based on the *L/D* ratio and the type of loading: cross-section tests $(L/D < 10)$, monotonic beam-column tests $(L/D \geq 10)$, and cyclic beam-column tests (all *L/D*). Each of the resulting six tables consists of a matrix of nine entries. Tests were placed in the appropriate entry in the matrix based on the *D/t* ratio and f'_c.

This categorization faciliates a quick overview of the types of CFTs in the collected literature that have been tested and shows what areas are lacking test results. The tables were motivated by the need for a concise experimental summary as a first step in calibrating analytical results.

All of the tests are beam-column experiments. Tests loaded purely by axial force are not included in these tables, with the exception of cyclic tests of CFTs used as braces. Tests in which the experimental setup consisted of more than one member (e.g., subassemblies) are noted in the tables. Also, the tests presented in the following tables are only those in which the experiments were documented in sufficient detail to permit their use in calibrating an analytical model. Complete references for each paper are shown in Appendix B. Each tabular entry is presented in the following format:

Author(s), Year (Number of Tests)

[*L/D; D/t;* f'_c *(ksi);* f_y *(ksi)*]

Table R1. Rectangular CFT Cross-Section Tests $(L/D < 10)$

	Low *D/t* (5 - 24)	Medium *D/t* (24 - 50)	High *D/t* (50 -)
Low f'_c (2 – 5)	*Tomii, Sakino, '79ab* (7) [3; 24; 2.7-2.9; 41.4]	*Furlong, '67* (13) [9.0; 32, 48; 3.4, 4.2; 48] *Tomii, Sakino, '79ab* (21) [3; 33-44; 3-5.5; 28-49]	
Medium f'_c (5 – 9)		*Furlong, '67* (4) [7.2; 26; 6.5; 70.3] *Nakahara, Sakino, '00* (4) [3; 34, 47; 6.9; 31, 46]	*Nakahara, Sakino, '00* (2) [3; 98; 6.9; 37]
High f'_c (9 -)		*Nakahara, Sakino, '98* (5) [3; 30; 17; 45, 113] *Varma et al., '00* (8) [5; 32, 48; 15; 46, 80]	*Nakahara, Sakino, '98* (5) [3; 60; 17; 45, 113]

Table R2 Monotonic Rectangular Beam-Column Tests $\left(L/D \geq 10\right)$

	Low *D/t* (5 - 24)	Medium *D/t* (24 - 50)	High *D/t* (50 -)
Low f'_c (2 - 5)	*Bridge, '76* (4) [10.5, 15; 20, 24; 4.6; 44] *Han et al., '01* (3) [18.52; 20; 3.4; 47]	*Shakir-Khalil, '90* (4) [21-37; 30; 4.4-4.9; 50-56] *Tsuda et al., '96* (18) [4-30; 35.1; 4.6; 59.8] *Han et al., '01* (18) [11-18; 24-36; 2.8-5.3; 47, 48]	
Medium f'_c (5 - 9)	*Knowles, Park '69* (4) [11, 18.7; 23; 5.9; 47, 58] *Bridge, '76* (4) [11-20; 20, 24; 5.3; 37-45] *Shakir-Khalil, '89* (7) [23, 35; 16, 24; 5.4; 53] *Cederwall et al., '90* (4) [25; 15; 5.7-6.8; 44-64] *Shakir-Khalil, '90* (12) [23; 24; 4.9-5.4; 49-53] *Lu, Kennedy, '94* (1) [20; 16; 6.8]	*Cederwall et al., '90* (2) [25; 24; 6.8; 44-64] *Shakir-Khalil, '91* (11) [21-32; 30; 5.4-6.2; 48-53] *Lu, Kennedy, '94* (11) [8.8-20; 24-39.6; 5.9-6.8; 51-63] *Zhong, Shahrooz, '97* (2) [14.4; 32; 6.1; 54]	
High f'_c (9 -)	*Cederwall et al., '90* (10) [25; 15; 11.6-14.9; 44-64]	*Cederwall et al., '90* (2) [25; 24; 13.9; 44-64]	

Table R3 Cyclic Rectangular Beam-Column Tests

	Low *D/t* (5 - 24)	Medium *D/t* (24 - 50)	High *D/t* (50 -)
Low f'_c (2 - 5)	*Sakino, Tomii, '81* (4) [6; 24; 2.9; 42-45] *Sakino, Ishibashi, '85* (4) [2; 24; 3.1-3.7; 42-46] *Kawaguchi, '91, '93* (14) [10; 22, 31; 3.1-3.6; 49] *Morino et al. '93* (5--2D and 3D subassemblies) [14.3; 21.3; 2.9; 57] *Kawaguchi et al.'02* (4--2D 1 bay frames) [8; 21.6; 2.7; 58.5]	*Sakino, Tomii, '81* (11) [4, 6; 34-46; 3.5; 42-45] *Sakino, Ishibashi, '85* (8) [2; 34, 45; 2.4-3.7; 42-46] *Liu, Goel, '88* (2) [23, 45; 30; 4; 54, 60]	*Matsui, Tsuda, '87* (6) [5.0; 47-94; 5.7-6.0; 71.5]
Medium f'_c (5 - 9)	*Fujimoto et al., '96* (2) [6; 20.0, 23.3; 5.8; 85.6, 113.1]	*Matsui, '86* (2--2D 1 bay frames) [6.7; 33, 47; 5.4; 42, 60] *Liu, Goel, '88* (4) [23-68; 14, 30; 6-8; 54-60] *Sugano et al., '92* (1) [6.8; 31.3; 5.5; 54] *Tsuda et al., '96* (10) [6-24; 34.5; 5; 58.6] *Fujimoto et al., '96* (4) [6; 30-46.7; 5.8; 58.1-113.1] *Nakahara, Sakino, '00* (3) [3; 34, 47; 6.9; 31, 46]	*Matsui, '86* (1--2D 1-bay frame) [6.7; 68; 5.5; 42] *Nakahara, Sakino, '00* (2) [3; 98; 6.9; 37]
High f'_c (9 -)	*Fujimoto et al., '96* (7) [6; 20.0, 23.3; 13.1; 85.6, 113.1]	*Varma et al., ‘00* (8) [5; 32, 48; 15; 46, 80] *Fujimoto et al., '96* (7) [6; 30-46.7; 13.1; 58.1-113.1]	

Table C1 Circular CFT Cross-Section Tests $(L/D < 10)$

	Low *D/t* (5 - 24)	Medium *D/t* (24 - 50)	High *D/t* (50 -)
Low f'_c (2 - 5)	*Kitada et al., '87* (3) [5.0; 23; 5.0; 41-53] *Elchalakani et al., '01* (2) [7.9, 9.9; 20.2, 23.5; 3.4; 59.2, 66.1]	*Furlong, '67* (5) [8.0; 36; 4.2; 42-60] *Kitada et al., '87* (11) [5.0; 25, 33; 2.5, 5.0; 41-53] *Sakino, '95* (2) [3.0; 34; 3.6; 121] *Elchalakani et al., '01* (5) [5.9, 9.9; 24.9, 40.2; 3.4; 52.9, 62.8]	*Furlong, '67* (5) [6.0; 98; 3.0-3.8; 42-60] *Sakino, '95* (4) [3.0; 53, 101; 3.6; 42, 84] *Elchalakani et al., '01* (3) [7.2, 7.3; 73.9-109.9; 3.4; 58]
Medium f'_c (5 - 9)	*Knowles, Park '69* (2) [9.1; 15; 5.9; 48-53] *Sakino, '95* (2) [3.0; 17; 5.8; 121]	*Cai, '91* (9) [5.2-9.3; 33; 5.0; 40-46] *Sakino, '95* (2) [3.0; 27 ; 5.8; 84]	*Furlong, '67* (12) [7.2; 53; 5.1; 42-60] *Knowles, Park, '69* (2) [9.8; 59; 5.9; 48-53] *Bridge, Webb, '92* (2) [3.0; 124; 8.6; 38] *Sakino, '95* (12) [3.0; 51-152; 5.8; 42, 84] *O'Shea, Bridge, '97c* (9) [3.5; 63-190; 7.3; 36]

Table C1 Circular CFT Cross-Section Tests $(L/D < 10)$ (cont'd)

	Low *D/t* (5 - 24)	Medium *D/t* (24 - 50)	High *D/t* (50 -)
High f'_c (9 -)		*Ichinohe et al., '91* (4) [2.0; 26-48; 8.5-9.6; 51-85] *Sakino, '95* (2) [3.0; 34; 11.3; 121]	*Prion, '89* (3) [7.0; 92; 13.3; 36-48] *Ichinohe et al., '91* (7) [2.0; 52-71; 9.0-9.6; 51-85] *Sakino, '95* (4) [3.0; 53, 101; 11.3; 42, 84] *O'Shea, Bridge, '97c* (7) [3.5; 63-190; 12; 36] *O'Shea, Bridge, '97d* (7) [3.5; 63-190; 15; 36]

Table C2 Monotonic Circular Beam-Column Tests $\left(L/D \geq 10\right)$

	Low *D/t* (5 - 24)	Medium *D/t* (24 - 50)	High *D/t* (50 -)
Low f'_c (2 - 5)	*Neogi et al., '69* (5) [20-24; 14-24; 2.6-4; 25-40] *Elchalakani et al., '01* (2) [17.6, 17.8; 12.8, 17.0, 3.4; 64.1, 66.7]	*Neogi et al., '69* (3) [20-24; 26-30; 3.7-4.1; 25-40] *Cai, '91* (18) [10.2-19.3; 33; 3.4-5.0; 40-46]	
Medium f'_c (5 - 9)	*Knowles, Park '69* (3) [; 15; 5.9; 48-53]	*Neogi et al., '69* (6) [16-20; 32-45; 5.0-8.1; 25-40]	*Neogi et al., '69* (4) [11-16; 69-78; 5.2-8.1; 25-40]
High f'_c (9 -)		*Kilpatrick, Rangan '97* (24) [21.4; 42; 13.9; 59]	*Prion, '89* (6) [14.0; 92; 13.3; 36-48] *Rangan, Joyce, '92* (7) [13-23; 64; 9.8; 32]

Table C3 Cyclic Circular Beam-Column Tests

	Low *D/t* (5 - 24)	Medium *D/t* (24 - 50)	High *D/t* (50 -)
Low f'_c (2 - 5)	*Kawano, Matsui, '97* (3) [5.1; 20; 4.6; 51.2]	*Huang et al., '91* (19) [9.1-17.5; 25-32; 4.0-4.6; 34-44] *Kawano, Matsui, '97* (26) [5-20; 25-46; 4.4-4.9; 50-61]	*Huang et al., '91* (4) [12.8-15.4; 54; 4.0; 34-44]
Medium f'_c (5 - 9)	*Sato et al., '91* (3) [7.3; 19; 5.2; 55.8] *Kawano, Matsui, '97* (2) [5.0; 19; 5.8; 60.0]	*Kawano, Matsui, '88* (10) [6.8-40.9; 26.3; 4.8-5.1; 48.5] *Huang et al., '91* (16) [5.5-8.8; 27-52; 5.3; 34-44] *Sugano et al., '92* (1) [6.7; 37.5; 5.4; 58] *Kawano, Matsui, '97* (7) [5.1, 19.7; 29.1, 46.5; 5.0-6.7; 47, 51]	*Kawano, Matsui, '97* (6) [5.0; 50, 53; 5.2, 5.4; 47-54]
High f'_c (9 -)		*Ichinohe et al., '91* (2) [2.0; 48-53; 9.6; 51-85]	*Prion, Boehme, '89* (2) [14.0; 92; 13.3; 36-48]

V. Appendices

Appendix A. List of Symbols

a = shear span (in)
A = effective cross-sectional area of composite section (in^2)
A_c = cross-sectional area of concrete (in^2)
A_e = effective area of steel tube (Bridge, Webb) (in^2)
A_s = cross-sectional area of steel tube (in^2)
A_{sw} = cross-sectional area of the web of square steel tube (in^2)
CFT = concrete-filled steel tube
d_s = slip distance (in)
D = outer diameter of circular tubes or depth (largest dimension) of rectangular tubes (in)
D_e = effective diameter to account for local buckling (in)
e = eccentricity of applied load (in)
e_x, e_y = eccentricities of applied load about the major and minor axes, respectively (in)
E = effective modulus of elasticity of composite section (ksi)
E_c = modulus of elasticity of concrete (ksi)
E_{ct} = tangent modulus of concrete (ksi)
E_s = modulus of elasticity of steel (ksi)
E_{st} = tangent modulus of steel (ksi)
E_t = effective tangent modulus of composite section (ksi)
EI = flexural stiffness of the composite section
EI_s = flexural stiffness of the steel tube
f_b = bond strength (ksi)
f_{bs} = bond stress (ksi)
f'_c = characteristic 28-day cylinder strength of concrete (ksi)
f_{cc} = modified confined concrete strength (ksi)
f_{cd} = design concrete strength (ksi)
f_{cr} = computed ultimate buckling stress of column (ksi)
f_{cu} = characteristic 28-day cube strength of concrete (ksi)
f_{my} = modified value of compostie column yield stress (SSRC) (ksi)
f_{su} = ultimate steel strength (ksi)
f_y = yield strength of steel tube (ksi)
f_{yb} = yield strength of girder (ksi)
G = effective shear modulus of elasticity of composite section (ksi)
G_c = shear modulus of elasticity of concrete (ksi)
G_s = shear modulus of elasticity of steel tube (ksi)
H = applied transverse load (k)
H_{max} = maximum applied transverse load (k)
H_{cm} = computed maximum transverse load (k)
HT = hollow steel tube
I = effective moment of inertia of composite section (in^4)
I_c = moment of inertia of concrete (in^4)

I_s = moment of inertia of steel (in^4)
k = empirical augmentation factor for concrete strength
K = effective length factor
K_o= initial shear stiffness
K_{co} = computed initial shear stiffness
L = length of member (in)
L_{eff} = effective length of member (length multiplied by K factor) (in)
L_s = short column length (in)
M = applied bending moment (k-in)
M_o = computed nominal bending moment capacity (k-in)
M_{pc} = computed ultimate bending moment in presence of axial force (k-in)
M_u = measured ultimate bending moment (k-in)
N.A. = test data not available, or category not applicable for given test
n_c= number of load cycles until fracture
n_b = number of load cycles until local buckling
P = applied axial load (k)
P_a = allowable axial load (as per a design specification) (k)
P_c = axial load acting on concrete (k)
P_{co} = computed nominal capacity of the concrete core (k)
P_{cr} = computed ultimate buckling load of column (k)
P_o = computed nominal axial load capacity of CFT (i.e., cross-section strength) (k)
P_{so} = computed nominal capacity of the steel tube (k)
P_s = axial load acting on steel tube (k)
P_u = measured ultimate axial load capacity (k)
P_{uo} = modified computed ultimate axial load capacity (k)
P_y = measured axial yield capacity (k)
q = simplification parameter
r = effective radius of gyration of composite section (in)
r_c = radius of gyration of concrete (in)
r_s = radius of gyration of steel (in)
R = member chord rotation (%)
R_{max} = maximum measured chord rotation (%)
R_{cm} = computed maximum chord rotation (%)
R/C = reinforced concrete
SRC = steel reinforced concrete (encased steel shape)
S_s = section modulus of steel (in^3)
t = steel tube thickness (in)
T = applied torsional moment (k-in)
T_o = computed ultimate torsional moment (k-in)
T_u = measured ultimate torsional moment (k-in)
V = shear force (k)
V_{max} = maximum shear force (k)
V_c = computed ultimate shear strength (k)
V_u = ultimate shear strength of composite section (k)
V_p = shear strength of panel zone (k)

V_s = shear strength of steel tube (k)
V_y = yield shear strength (k)
V_{yc} = computed yield shear strength (k)
W = energy absorbtion
w = weight of concrete (pcf)
α = ratio of steel area to concrete area
β = ratio of smaller end moment to larger end moment
γ = shear strain
δ = mid-height deflection (in)
δ_{axial} = axial deformation (in)
δ_y = yield axial deformation (in)
Δ = end deflection of column (in)
Δ_{cr} = end deflection of column causing local buckling of steel tube (in)
Δ_{max} = maximum end deflection of column (in)
ε = axial or longitudinal strain in steel (mid-height)
ε_{axial} = axial strain of composite section
ε_{circum} = circumferential or hoop strain in steel
ε_{concr} = axial strain in concrete (mid-height)
ε_o = concrete strain at f'_c
ε_s = axial strain in steel
ε_u = strain at ultimate stress of steel tube
ε_d = diagonal strain in the connection
ε_{vol} = concrete strain when volumetric expansion occurs
ε_y = strain at yield stress of steel tube
θ = torsional angle of twist; rotation (rad)
θ_{pc} = ultimate rotation (rad)
θ_c = connection rotation
θ_u = ultimate rotation
λ = slenderness ratio (KL/r)
μ = curvature ductility
μ' = empirical parameter
ν = Poisson's ratio
τ_c = shear stress in concrete
ξ = confinement factor
σ_c = stress in concrete including effect of confinement (ksi)
σ_r = radial confining pressure on concrete (ksi)
σ_{sl} = longitudinal stress in steel (ksi)
σ_{sc} = circumferential stress in steel (ksi)
φ_e = strength reduction factor due to the eccentricity ratio (e/r_c)
φ_l = strength reduction factor due to the slenderness ratio
ϕ = curvature

Appendix B. References

COLUMN, BEAM-COLUMN, AND FRAME TESTS AND ANALYSES

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Appendix C. Supplementary Bibliography

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