



THE DESIGN AND CONSTRUCTION OF CONCRETE-FILLED STEEL TUBE COLUMN FRAMES

by

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SUMMARY

The construction of a moment-resisting joint in a steel frame must accommodate the acceptable tolerances that occur during the production of structural shapes and during fabrication and erection. Further, the construction should be economical; however, the joint must be of sufficient strength and stiffness to satisfy the intended structural integrity and seismic demands. This paper describes the design and construction of a type of moment-resisting joint recently used for two low-rise buildings in Vancouver, Washington U.S.A. The joint consists of a structural steel wide-flange shape penetrating continuously through a composite concrete-filled steel tube (CFT). Examples of two joint types are considered: One in which the girder was attached to the column in the field, and the second consisting of shop fabrication of the critical joint. Each joint configuration was detailed to accommodate the needed tolerances of fabrication and field erection, while minimizing welded joints in critical regions of the connection.

The design of frames with CFT columns is similar to frames using structural steel shapes for columns in which it is desirable to maintain a hinging girder joint. Thus, the ability to accurately compute the CFT column and joint strengths is imperative. Strength and stiffness estimates of the CFT column from both the *American Concrete Institute's ACI-318* and the *American Institute of Steel Construction 1994 AISC/LRFD* provisions are compared. Each specification produces somewhat different values for strength, but the estimate of stiffness varies most significantly between the two provisions. Finally, one method to estimate joint strength, which is shown to depend significantly on joint configuration, is provided.

INTRODUCTION

Concrete-filled steel tube (CFT) columns combine the advantages of ductility, generally associated with steel structures, with the stiffness of a concrete structural system. The advantages of the concrete-filled steel tube column over other composite systems include: The steel tube provides formwork for the concrete, the concrete prolongs local buckling of the steel tube wall, the tube prohibits excessive concrete spalling, and composite columns add significant stiffness to a frame compared to more traditional steel frame construction. While many advantages exist, the use of CFTs in building construction has been

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limited, in part, to a lack of construction experience, a lack of understanding of the design provisions and the complexity of connection detailing. Consequently, a joint was needed that could utilize the favorable strength and stiffness characteristics of the concrete-filled tube column yet be constructible.

This paper summarizes a steel girder to concrete-filled steel tube (CFT) connection detail that has been designed for several recent projects constructed in Vancouver, WA, U.S.A. Several variations of CFT joints have been investigated experimentally [1], and it appears that continuing the girder through the CFT column can induce full plastic hinging of the structural shape. The practical connection has been designed to accommodate the needed tolerances resulting from the rolling process, fabrication and construction. Circular CFTs were used because of their more favorable ductility characteristics relative to the square tube, and were more accommodating when the structural frames were not oriented along orthogonal axes. Two connection types will be discussed: One where the girder-to-column joint was produced in the field and the other in which the joint was shop fabricated.

The design of this connection is discussed, along with the relevant requirements from the applicable U.S. design codes. A design methodology, which consists of the flexural strength, stiffness and equilibrium at the joint, is presented. Results suggest that the flexural strength of the CFT section depends on the assumptions used in the prediction of strength. However, stiffness controls the design of many structural systems using moment-resisting connections. Flexural stiffness was shown to be highly variable, depending on the method of computation. Finally, a reasonable evaluation of joint equilibrium was needed to ensure proper inelastic behavior of the structural system. These issues are critical in obtaining a safe and economical CFT joint design.

INELASTIC CONNECTION BEHAVIOR

Figure 1 shows the results from two of the six circular CFT connections tested by Schneider, *et. al.* [1, 2]. In this study, only circular tubes were considered, since the connection of the girder to the tube wall tends to be more difficult when compared to the square tube counterpart. The Type I connection was a connection that was attached to the skin of the steel tube only. This connection was favored by many of the practitioners on the advisory panel for this research project since it appeared to be the easiest to construct. Effectively, the flanges and the web were welded to the skin of the tube, and the through-thickness shear of the tube wall controlled the distribution of flange force, or the flared geometry of the flange plate, to the tube wall. The Type II connection had the girder section continue through the concrete-filled steel tube. An opening was cut in the steel tube to allow the girder to pass through the core. Each connection tested consisted of a 356 mm (14 inch) diameter pipe with a 6.4 mm (1/4 inch) wall thickness and a W14x38 for the girder. The yield strength of the pipe and the girder was 320 MPa (46.0 ksi), with an approximate concrete strength of 35 MPa (5.0 ksi).

In all cases in this test program the connection was intended to be shop fabricated. This was primarily to control the quality of all welded joints. A stub-out of the connection was intended to be attached to the tube column and shipped to the construction site. The field splice would be made to the end of the connection stub-out of the connection. As the construction of the structural system progressed, the tube would be filled with concrete. Clearly, a connection like Type I provides the least amount of interference with the placement of the concrete infill. However, a connection like Type II may introduce significant difficulty in getting good consolidation of the concrete in the tube for lifts over several floors.

As demonstrated by the normalized moment-rotation behavior shown in Figure 1, the connection that continued through the CFT exhibited far superior behavior relative to the exterior-only Type I connection. For the Type I connection, the steel tube experienced high local distortions in the connected region. Fracture initiated in the connection stub at approximately 1.25% total rotation and propagated into the tube wall by 2.75% rotation. This tearing propagated from the tips of the flange toward the web.

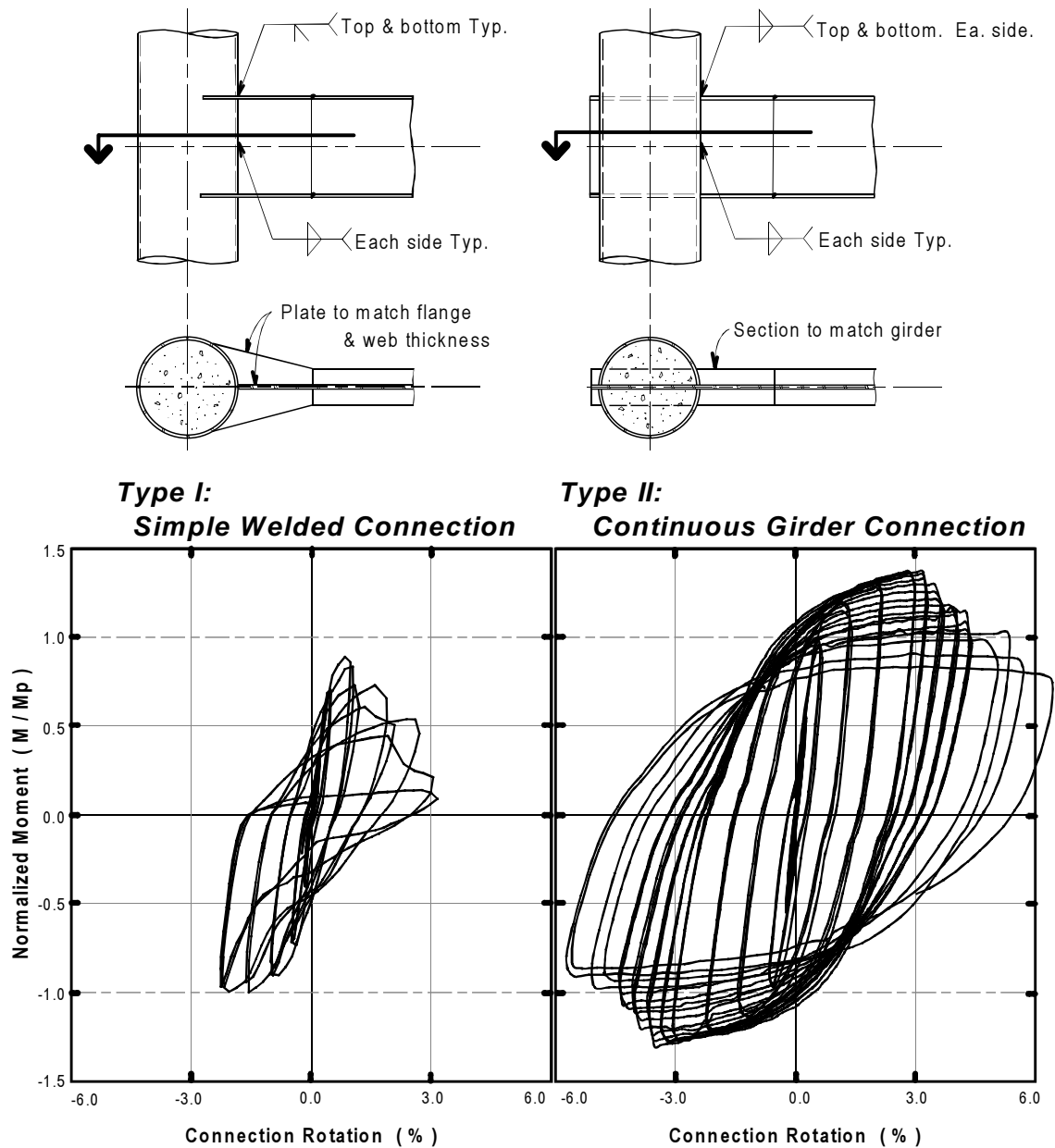


Figure 1. Inelastic Moment-rotation Behavior of Tested CFT Joints.

Only one flange fractured, resulting in an unsymmetric $M-\theta$ behavior and a pinching of the hysteretic curves. Results of connection Type II exhibited quite stable inelastic behavior. Effectively, this connection exhibited classic girder hinging type behavior. Local flange buckling was observed at approximately 3.0% total rotation, and the web buckling was observed at approximately 3.5% total rotation. The deterioration of the inelastic characteristics was observed after the onset of local web buckling. Failure of the connection was caused by fracture of the beam flange in the connection stub region. This flange tearing eventually propagated into the web. Although the flexural strength decreased approximately 30% from peak value, the hysteretic behavior remained stable even at large rotations. No concrete crushing was observed, and there was no apparent sign of local distress on the tube wall. These results clearly indicate that connections used in moment-resisting steel frames using CFT columns should utilize the continuous connection-type if the system might be subjected to large seismic demands.

PRACTICAL USE OF CFT COLUMN CONNECTIONS

The two CFT joint types constructed for different buildings are shown in Photos 1 and 2. In both cases, the wide-flange shape was continuous through the steel pipe column. Both buildings were two stories; with the first floor using a composite concrete slab on a steel girder framing system and the roof having a corrugated metal deck only. Both buildings also had an orthogonal layout for the lateral-load resisting frames, with most frames located along the perimeter. Circular CFTs were used for both buildings in lieu of the more traditional steel wide-flange columns because the preliminary cost estimates indicated that CFTs could be as much as 20% more economical.



Photo 1a: Field-fabricated CFT Joint.



Photo 1b: Bent Plate Closure on Girder Web.



Photo 2a: Shop-fabricated CFT Joint.



Photo 2b: Girder Connection for Shop Joint.

The primary objective of each joint-type shown in Photos 1 and 2 was to maintain continuity of the wide-flange shape through the full depth of the CFT column. To make the joint more constructible a rectangular notch, matching the width of the wide-flange shape, was cut in the steel pipe. A bent plate welded to the girder web, spanning full depth between girder flanges, must be used as a closure plate for the joint. Closure plates, as shown in Photo 1b, are angle-shaped with the 90° bend extending toward the core of the concrete-filled tube. This provides some tolerance in the final location of the girder through the column and provides a surface on which to weld the tube to the girder. Because of the need to develop the flexural strength at the CFT joint, this vertical weld between the steel pipe and the girder web is critical. The steel tube wall may be welded either directly to the 90°-bend surface of the closure angle, or

a filler plate may be welded between the steel tube and the closure plate. Closure plates may or may not be needed between the bottom girder flange and the tube; however, a closure will be needed above the top flange. Once welded, the joint is fully enclosed thereby ensuring complete confinement of the concrete core at the joint by the steel tube.

After the joint is complete, the steel pipe column is filled with concrete. To facilitate compaction of the concrete around the joint, holes in the girder flanges have been used to allow concrete consolidation around the girder flanges in the core of the steel tube. Several types of concrete mixes and delivery methods have been tried; however, it has been found that self-compacting concretes may be the most appropriate and, perhaps, economical considering the cost and labor involved in tube wall vibration. The design should also consider the ability to avoid long drop-lengths when placing concrete in the steel tube core.

The CFT joint shown in Photos 1a and 1b provides the most convenient method to accommodate fabrication and construction tolerances. The convenience of this method is that girders can span over at least two CFT columns, eliminating the need for temporary support during construction and minimizing the number of field connections for the girder. The field splice for the girder can be located away from the column-girder joint, eliminating the critical flange welds otherwise needed for moment-resisting steel frame connections. Further, the girder splice can be a bolted end plate connection, minimizing the number of complete-joint penetration field welds. This CFT joint type is a relatively simple post-and-beam solution to achieve a more sophisticated moment-resisting structural frame.

The slot in the tube provides other advantages in frame construction. Accommodating the tolerances needed for U.S. construction practices may preclude the use of some connections that otherwise exhibit favorable seismic performance. This may be due to a complex field construction, or a detail that requires a tighter tolerance than otherwise possible. With the slotted steel pipe connection the girders can be connected without significant racking of the frame. Further, the girders can be slightly out-of-alignment in plan, or in elevation, without introducing significant internal misalignment stresses in the girders or the columns. Finally, leveling of the girders can be made by leveling nuts applied to each girder at the bottom of the slot. This could be done by slightly over-cutting the slot base in the shop to ensure ease of construction.

Construction experience of the first low-rise building in Vancouver, WA utilizing the joint shown in Photo 1 suggests that the steel tube for the next level should be erected before the girder connection is fully welded. This is due to the out-of-round condition created for the tube once the girder notch is cut. Residual stresses within the steel pipe column cause the portion of pipe remaining to flatten, pinching the tube toward the notch. Consequently, the tube wall joint is out-of-alignment between the upper and lower columns when placing the next column lift. The portion of the pipe above the girder can more easily be brought back into alignment if the vertical joint between the pipe wall and the closure plate is welded after the column fit-up is complete.

One drawback of this particular connection is that each column lift extends over one floor only. Thus, a complete-joint penetration groove weld is needed for each pipe column at each level. This is reasonably acceptable for a building two-story or so, but splicing each column for each lift may be impractical for buildings with several floors. For multiple floors, it is considered more advantageous to the construction schedule if each column can extend over several floors per lift. For this type of construction, a slot could be cut in the tube to accept a girder stub-out at each floor level. This column could be assembled in the shop and erected in the field in a tree configuration. This type of connection is shown in Photos 2a and 2b. The girder could be only partially assembled so that girders and the frame could be aligned without introducing significant internal stresses or could be fully connected in the shop and the beam span between the column stubs could provide some of the erection tolerances. The second method is shown under construction in Photo 2. For this particular case, a welded flange plate connection, as shown in

Photo 2b, was used at the girder splice. The welded flange plate connection was designed to remain elastic and induce girder hinging between the beam splice and the face of the CFT column. The splice was designed to accommodate the full plastic hinging of the girder at each end.

Frame geometry and construction sequence would dictate whether a single- or multi-story lift would be more economical, since many more girder connections may be required in the multi-story lift compared to the number of column connections required in the single-story lift. Regardless, it was considered that the same basic CFT joint was similar in both conditions and the design methodologies were equivalent.

DESIGN ISSUES

Proper design of moment-resisting frames using CFT columns requires an ability to determine the strength and stiffness of the CFT element. Either the *American Concrete Institute ACI-318* [3] concrete specification or the *American Institute of Steel Construction AISC/LRFD* [4] steel specification can be used to determine structural characteristics. Because of the design assumptions, each code produces slightly different results. Unmodified flexural strength is relatively similar between the two specifications; however, the factored capacities used for design create more differences for highly loaded CFT columns. Flexural stiffness is substantially different between the two provisions. The difference in predicted stiffness can make a significant impact on the design and analysis of the structural system, since the element sizes in many moment-resisting frames are controlled by drift. The following is the methodology used for the design of the connections used in these buildings in Vancouver, WA, and a discussion of some of the limitations from the available research.

FLEXURAL STRENGTH

Since the current structural design philosophy in the U.S. is strength-based, the flexural strength of the structural elements is a primary consideration. The strength prediction of a wide-flange girder has evolved over many years and is fairly well prescribed. However, the two available material specifications differ slightly in the assumptions used to compute the axial-load, moment interaction surface of the CFT column.

In general, the *ACI -318* concrete specification applies the same basic assumptions used for reinforced concrete sections to the composite element. The interaction surface was obtained by varying the curvature through the cross-section from full compression to full tension. Compatibility was enforced by the plane section assumption, and the compressive strain was maintained at 0.003 mm/mm . The Whitney rectangular stress block was used to obtain the compressive strength of the concrete core. Tension on the concrete section was ignored. The strength provided by the steel tube depended on the strain through the cross-section. Tube stress was computed as the strain times the elastic modulus, but was limited by the yield stress. This stress-strain condition is shown in Figure 2. Axial capacity of the cross-section for the imposed curvature was computed by the summation of stresses through the cross-section. The flexural strength of the cross-section was computed by the summation of the first moment of stresses.

The major difference between the concrete code and the steel code is the assumption of curvature at full strength. Since the *AISC/LRFD* steel specification is strength based, the axial-load, moment interaction surface is obtained by assuming a fully plastic cross-section. This is equivalent to an infinite curvature about the plastic neutral axis. A schematic of a stress distribution acceptable to the *AISC/LRFD* is also shown in Figure 2. This indicates that the tube wall on either side of the plastic neutral axis was at yield. The portion of the concrete in compression was then determined by the rectangular stress block assumptions.

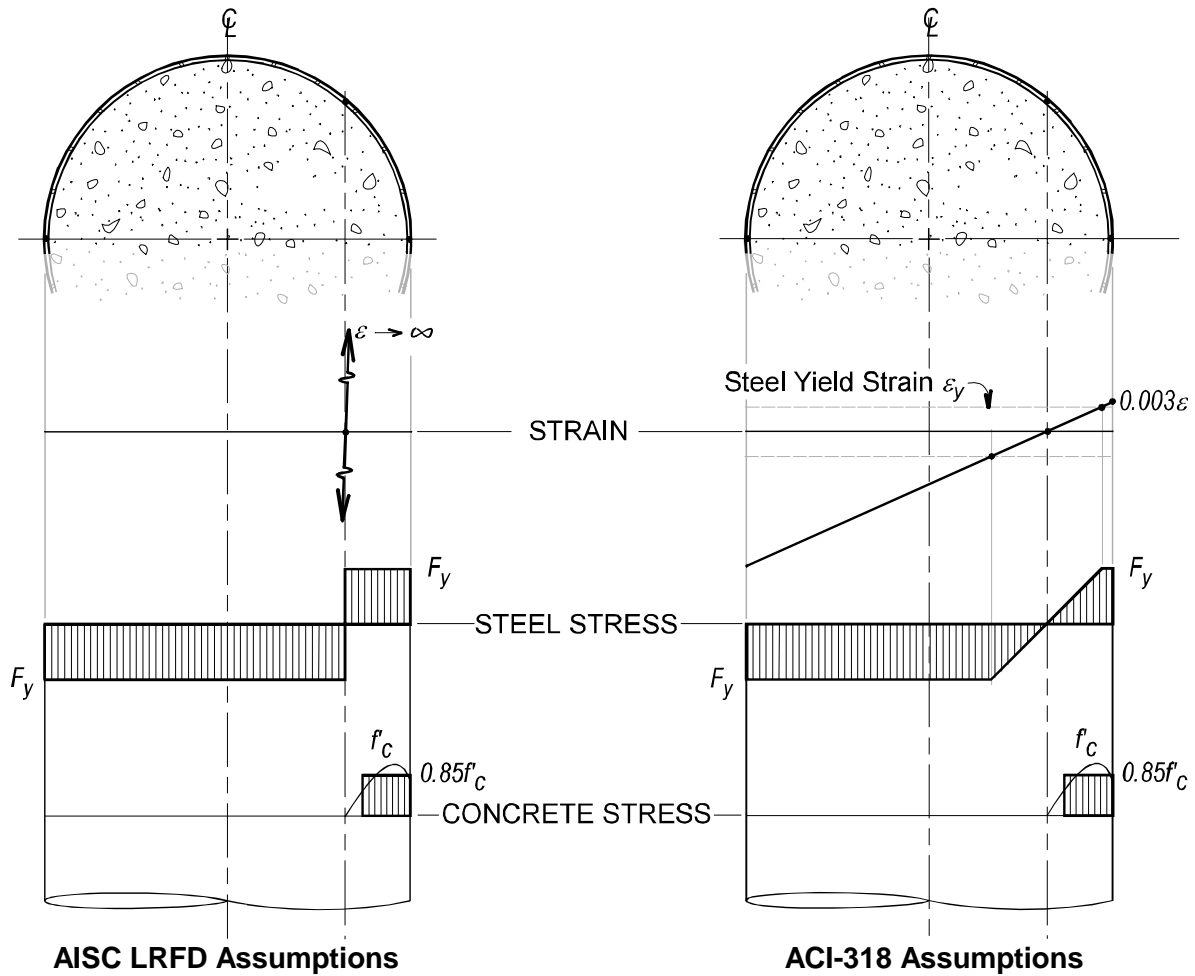


Figure 2: Stress-Strain Assumptions for the U.S. Specifications.

The strength of the CFT column was determined by both methods. In general the strength computed using the *ACI* assumptions produced less flexural capacity than the strength predicted by the ultimate strength calculations. As the interaction surface approached full tension or compression, the two methods converged on the same solution. The strength must be scaled by the appropriate resistance factors to produce the factored capacity to be used in design. A comparison between the flexural strength estimates using each design method is shown in Figure 3. For these interaction surfaces, compression is shown as positive. These values were for 406 mm (16 inch) diameter steel tube pipes with 6.35 mm (1/4 inch) wall thickness. This size is typical for the CFT columns in the two-story buildings shown in Photos 1 and 2.

The *AISC/LRFD* describes each quadrant of the strength interaction surface with a bilinear relationship. The transition for composite behavior occurs at 30% of the axial capacity. To demonstrate the approximation of the *AISC/LRFD* method, the strength surface was computed for the location of the plastic neutral axis at very small increments through the depth of the cross-section. As shown by Figure 3, the bilinear relation provided by the *AISC/LRFD* is not descriptive of the entire interaction surface, and results in a very conservative estimate in strength for much of the failure surface. This would significantly impact the evaluation of highly loaded CFT columns.

Also shown in Figure 3 is the axial load moment demand for the CFT columns for the building design shown in Photo 2. This comparison can be made since the factored capacities from both the *ACI* and *AISC/LRFD* specifications have been adjusted for CFT column slenderness as determined by the geometry

of the specific building design. Further, the bending moment demand has been adjusted for the P- Δ effect. For these buildings, the P- Δ effect was found to increase the primary bending moment by as much as 16% even though the axial column load for a two-story building is relatively small. A comparison between the two specifications suggested that several regions within the interaction surface produce a large difference in strength estimates. However, since elements are typically sized for stiffness, the CFT column capacity exceeded demand.

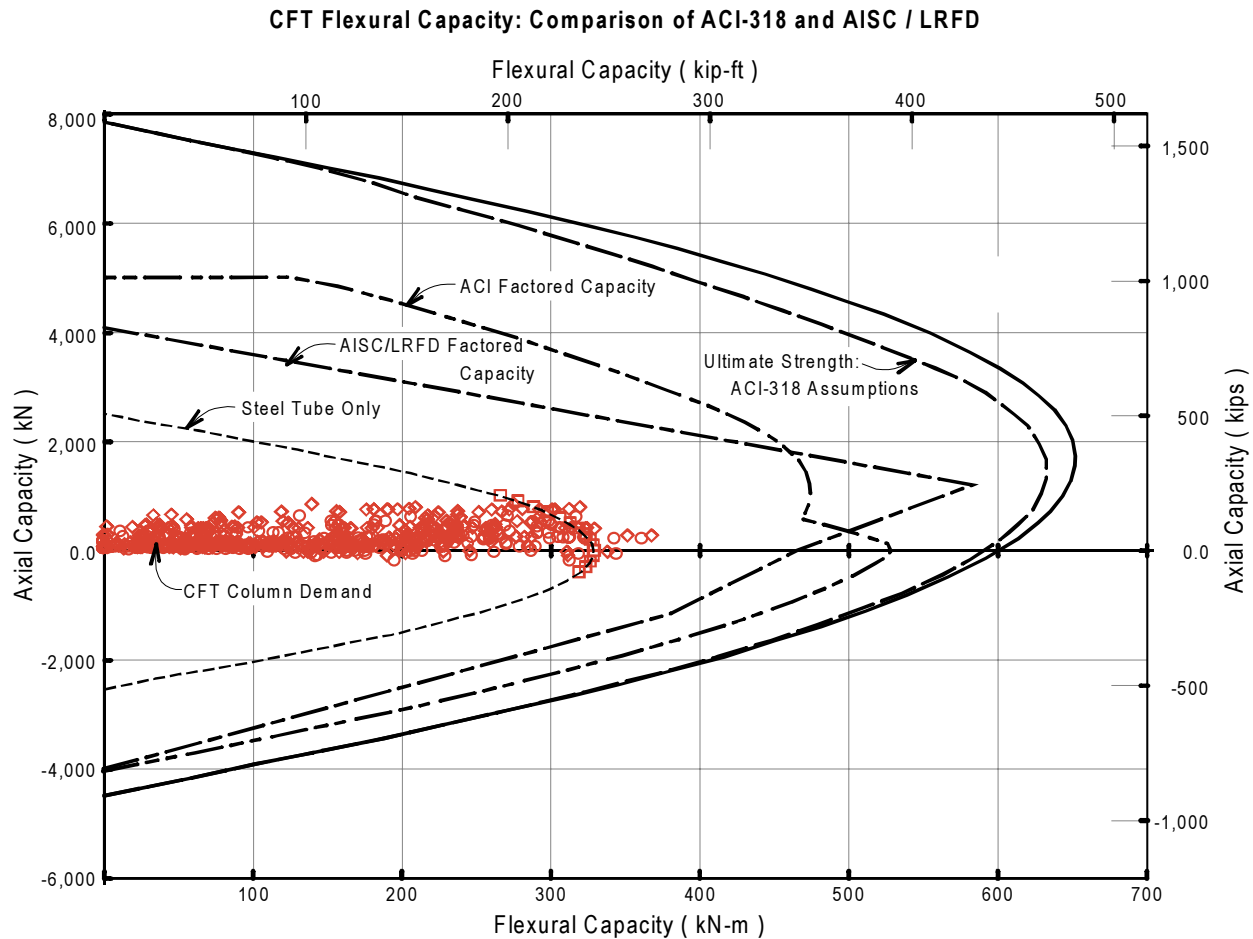


Figure 3. Flexural Capacity of CFT Column.

The general design philosophy for the flexural strength of CFT columns is that the steel tube provides the longitudinal reinforcement and confines the concrete core. This differs from a conventional spirally-confined concrete column in which the longitudinal reinforcement resists stresses independently of the spiral confinement. In the case of the steel tube, it is possible the steel tube may be unable to resist both the strain demand of longitudinal compression and confinement of the concrete core simultaneously. Confinement of the core induces hoop (or tensile) stresses in the tube, while trying to resist compression due to flexure. Thus, it is possible the tube resists primarily the confinement forces. This is clearly evident in test results discussed by Schneider [5] for axially-loaded CFT columns in which the tube shows significant confinement strain as the concrete core approaches the cylinder strength.

Figure 4 illustrates the potential impact of losing the compressive strength of the steel tube. As shown, this clearly affects the compression and flexural capacities of the CFT column. In comparison to the CFT demand from the building design, it is possible the factored capacity of such a column may not be adequate. However, while there may be a loss of compressive strength in the steel tube, the added

confinement of the steel tube may increase the concrete strength. Experimental data and the supporting analytical results given by Schneider [5] suggest that concrete strength in circular tubes can increase by at least 50% more than the unconfined cylinder strength f'_c . Consequently, Figure 4 also shows the flexural strength failure surfaces of CFT columns having an increased concrete compressive strength of $1.25f'_c$ and $1.50f'_c$. In both of these cases, the compressive strength of the steel tube was considered to be negligible. The results indicate that some of the flexural strength of the CFT column is recovered compared to the estimates based on the *ACI* and *AISC/LRFD* assumptions; however, recovery is not near the estimates computed using the basic design assumptions.

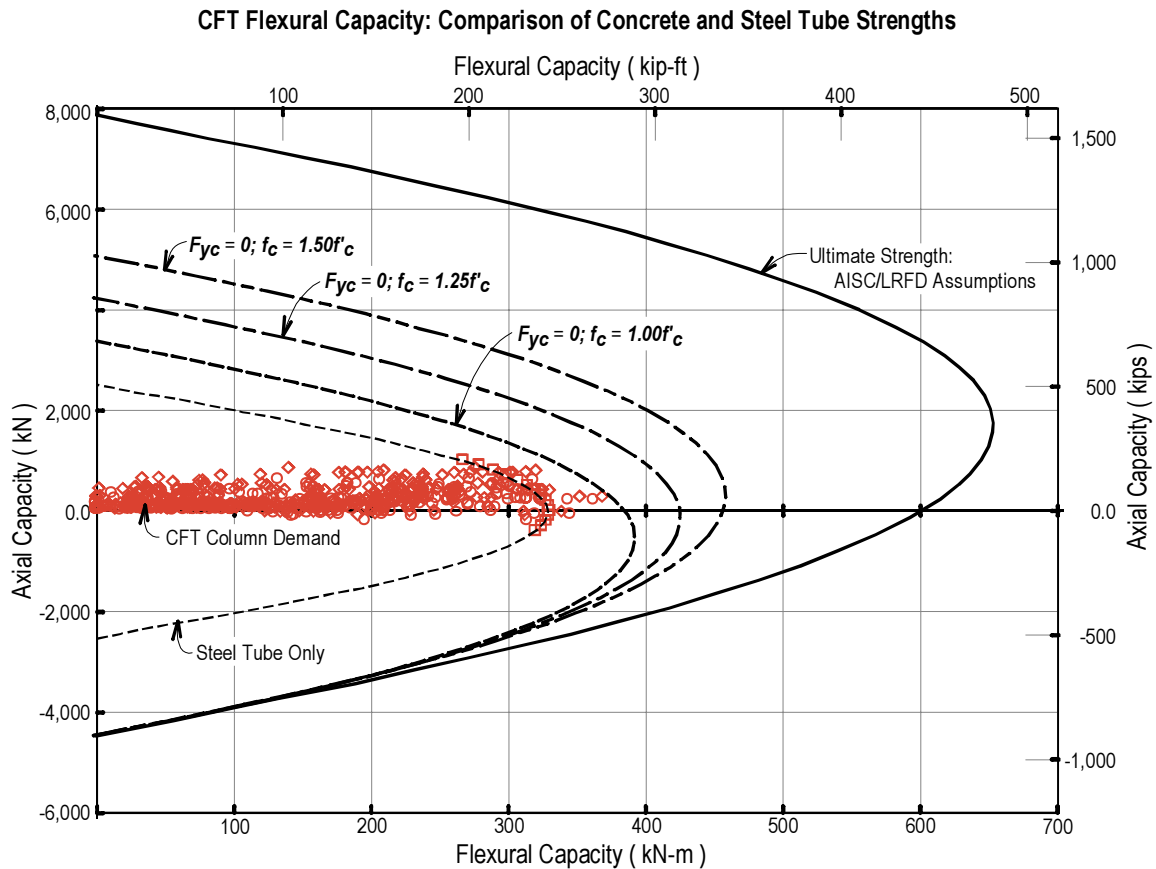


Figure 4. Influence of Steel Tube and Concrete Strengths on the CFT Column.

The discrepancy in the amount of decrease due to the steel tube compressive capacity and the increased concrete strength due to confinement varies depending on pipe diameter, the diameter-to-tube wall thickness ratio, the ratio of steel to concrete area and other factors. In some cases, it has been found that the increase in concrete can offset all strength reductions due to loss of compressive strength of the steel tube. Consequently, this impact is highly dependent on the CFT column geometry.

JOINT EQUILIBRIUM

To achieve an adequate seismic design, the strength of the joint must also be considered. For the frames designed in Vancouver, WA a hinging girder structural system was desirable. While it may be easy to proportion the size of the CFT column and girder at a joint to provide the proper strength ratio, the girder-column joint must also sustain this load. Equilibrium of the joint is shown in Figure 5.

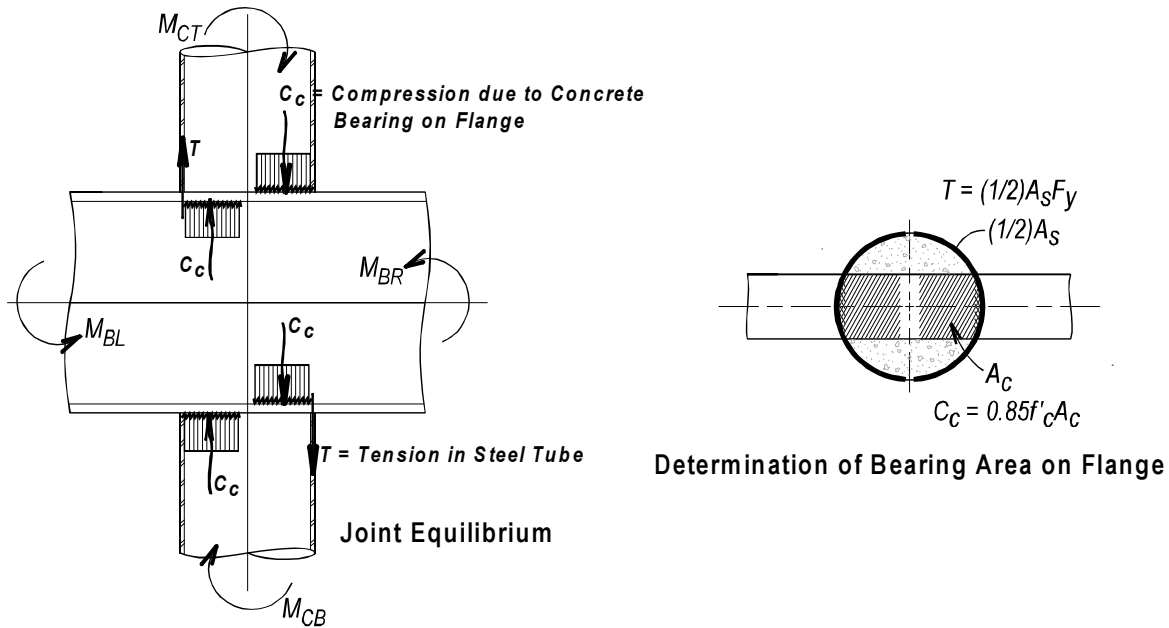


Figure 5. Joint Equilibrium and Internal Forces of the CFT Joint.

The moment in the joint must balance the moment induced by the girders, which in turn must be transferred to the CFT column. Two components are needed to resist the joint moment: The bearing strength between the steel girder flange and the concrete core, and the tension in the steel tube. The contact area between the steel girder and the compressive strength of the concrete core determine bearing strength. It is assumed that girder rotation occurs about the geometric center of the CFT column. Locally, the strength of the concrete will be higher than the cylinder strength because of the core confinement provided by the steel tube which may be considered in design. Depending on the diameter of the concrete-filled steel tube and the wide-flange girder width, both flanges could be considered for bearing in estimating the strength of the joint. Finally, the vertical welded joint between the steel tube and the bent closure plate must transfer the remaining moment. This dictates the amount of shear transfer needed between the closure plate and the steel tube wall. It has been found that joint equilibrium may govern the desired plastic hinge location and may limit the size of the girder that may be connected to the CFT column.

This equilibrium is consistent with the need to induce the moment distribution in the cross-section, as shown in Figure 2, and is the only joint equilibrium consistent with *ACI-318* specification. Provisions 10.16.3 and 10.16.4 of the *ACI* require the *axial load assigned to concrete shall be transferred in direct bearing and the load not assigned to concrete shall be developed by direct connection to the structural pipe* [3].

FLEXURAL STIFFNESS

While the ability to predict the flexural strength of the CFT column is critical, element sizes in a moment-resisting frame are generally controlled by flexural stiffness. Although both building systems considered in this evaluation were only two-stories, element sizes in both cases were selected to control drift. It was found that the flexural stiffness of the CFT columns was responsible for approximately 40% of the drift, while girder stiffness accounted for the remaining portion.

The flexural stiffness computed using *ACI-318* and *AISC/LRFD* specifications is shown in Figure 6. As shown, flexural stiffness varies significantly between the two specifications and is consistently different for increasing concrete strength. For the concrete strength used in the building shown in Photo 2, the

flexural stiffness, computed using the *ACI-318* provision, was $EI_{CFT} = 1.223 EI_S$, where I_S is the moment of inertia of the steel tube only. The stiffness predicted by the *AISC/LRFD* was $EI_{CFT} = 1.903 EI_S$. This resulted in a 56% difference in the estimate of stiffness.

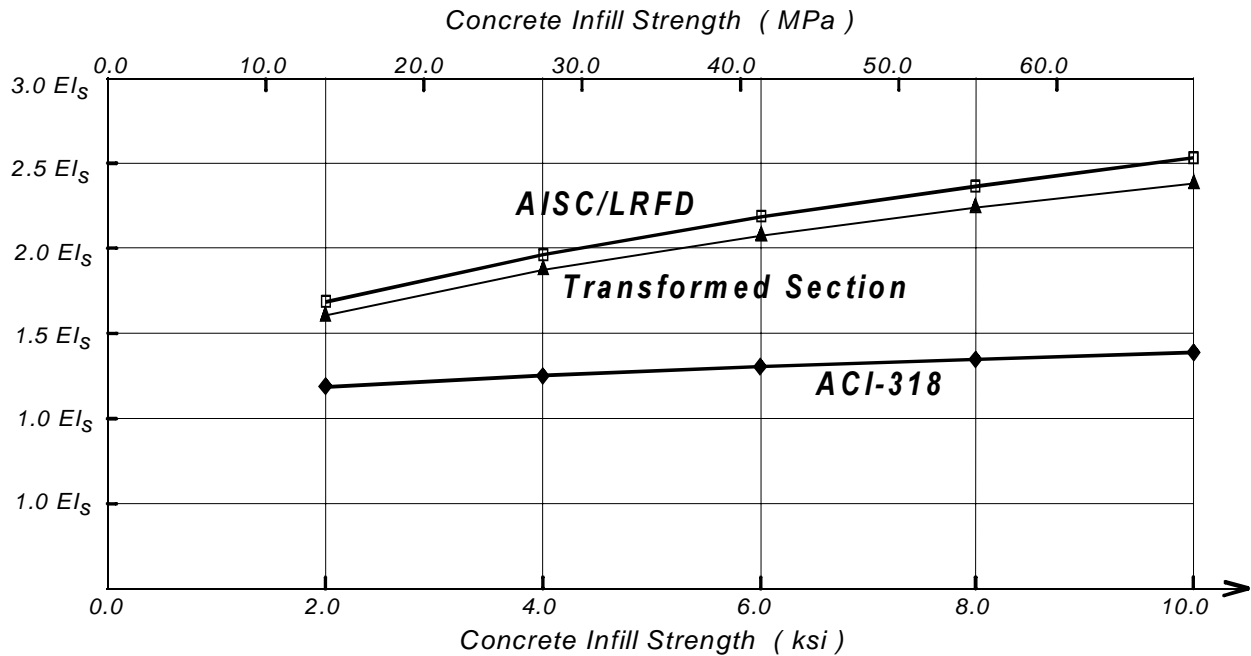


Figure 6. Computed Flexural Stiffness of ACI-318 and AISC/LRFD Provisions.

For comparison, the flexural stiffness was computed using a transformed section analysis from basic mechanics. This stiffness was computed as $EI_{CFT} = 1.933 EI_S$. However, this assumed an uncracked concrete cross-section. A flexural stiffness of $1.696 EI_S$ was computed when using a cracked moment of inertia of 70% of the gross moment of inertia, as suggested by *ACI-318* for columns. Because of the differences between the two predicted values, the transformed section method, assuming a cracked moment of inertia of the concrete core, seems to be a reasonable compromise between the two predicted values. Further, this does not excessively penalize the stiffness prediction as might otherwise be considered from the *ACI-318* provision.

CONCLUDING REMARKS

This paper discusses an economical connection for a steel girder to a concrete-filled steel tube (CFT) column that has been designed for two recent building projects near Vancouver, WA. The anticipated behavior during a seismic event, and some critical issues in the design of the connection, were also presented. Some comments regarding this joint behavior and design are worth noting:

1. Test results indicate that a girder connection continuous through the CFT column core will clearly have the potential to sustain a large seismic event. The continuous connection detail shown in this paper has proven to be reasonably economical and constructible relative to the wide-flange column frame.
2. The flexural capacity computed by the ultimate strength method, as allowed by the *AISC/LRFD* steel specification, resulted in more capacity than provided by the *ACI-318* concrete specification. This strength increase was due to utilizing the full capacity of the CFT cross-section at strength allowed by the *AISC/LRFD* provision. However, once each strength failure surface is factored for

use in design, the strength predicted by *ACI* provides more capacity for most of the axial-load / moment behavior compared to the *AISC/LRFD* provisions.

3. The computed flexural stiffness was shown to vary as much as 60% for the column sizes investigated for the building designs discussed in this paper. Stiffness, estimated using the transformed properties of composite materials and considering a cracked moment of inertia, was found to provide a reasonable compromise between the *ACI* and *AISC* provisions.
4. Joint equilibrium requires a distribution of load between the steel tube and the concrete core. Joint equilibrium is highly dependent on the geometry of the CFT column and the wide-flange girder and must be considered in design, since it has been shown to control the size of the girder that can be combined with particular CFT column sizes.
5. Research is needed to identify true behavior of the CFT column regarding the strain demand on the compression side of the column in flexure, the compression capacity of the concrete core, the distribution of stresses and equilibrium of the joint and the flexural stiffness. Verification on the accuracy of the axial-load, moment interaction surface is also needed. While reasonable assumptions can be made, these assumptions should be verified with test data.

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