BEHAVIOR OF MOMENT CONNECTIONS BETWEEN CONCRETE-FILLED STEEL TUBE COLUMNS AND WIDE FLANGE STEEL BEAMS SUBJECTED TO SEISMIC LOADS

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SUMMARY

Split-tee, through-bolted connections between rectangular concrete-filled steel tubes and wide-flange steel beams were investigated to determine the ability of the concrete core within the panel-zone to transfer join shear forces. By depending on the confined concrete core within the joint for shear transfer, additional labor-intensive details such as exterior diaphragms and internal stiffeners could be avoided. The objective of this research program was to quantify the extent to which the confined concrete core, or compression strut, plays a role in the transfer of joint forces, and to ultimately develop design guidelines and criteria for use in building codes. The testing procedures used are described, and some preliminary results of full-scale moment connection tests are presented. Further analysis of the results will be used to develop design code recommendations in a future paper.

INTRODUCTION

Research was conducted at the Phil M. Ferguson Structural Engineering Laboratory at The University of Texas at Austin to study the behavior of the split-tee through-bolted moment connection between concrete-filled steel tubes (CFT) and wide flange beams. The aim of the program was to develop design recommendations for this type of joint, focusing on the role of the concrete core in transferring beam flange forces through the joint. The connection testing program was divided into two phases, which consisted of a total of 21 test specimens.

Phase I was an investigation into the shear behavior of 15 half-scale CFT panel-zone specimens, and is referred to as the panel-zone tests. Joint force transfer conditions were simulated without constructing the complete connection. A special loading frame was designed and used to subject the tubes to joint shear forces by applying simulated beam flange forces to the panel-zone region.

Phase II consisted of six full-scale, fully assembled moment connections. All specimens in Phase II were cruciform specimens with spit-tee through-bolted connection details.

Specimens and Properties

For the first 15 experiments (Phase I), test specimens and a reaction frame were designed to idealize the connection zone region on half-scale CFTs without utilizing actual connection details. Test results, which were presented in the SEWC 1998 [Koester, 1998], showed that changing the footprint of the simulated beam flange forces had negligible effect on the capacity of the joint. In addition, joint strength was not limited by the plastic moment capacity of the composite section. A companion FEM analysis [Uchida, 1998] showed satisfactory comparison between the test data and the analytical model. It was also shown by the same FEM analysis that the stress condition of the panel-zone test specimens is similar to that of a beam-column joint with a split-tee through-bolted connection.

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The remaining six tests (Phase II), the focus of this paper, were conducted on full-scale, through-bolted moment connections (Figure 1). The test setup for the cruciform-shaped beam-to-column moment connections is shown in Figure 2. Phase II specimens were fabricated from 305 mm (12 in.) and 406 mm (16 in.) square concrete filled tubes. All six specimens were based on the same general connection design, but connection component dimensions were varied.

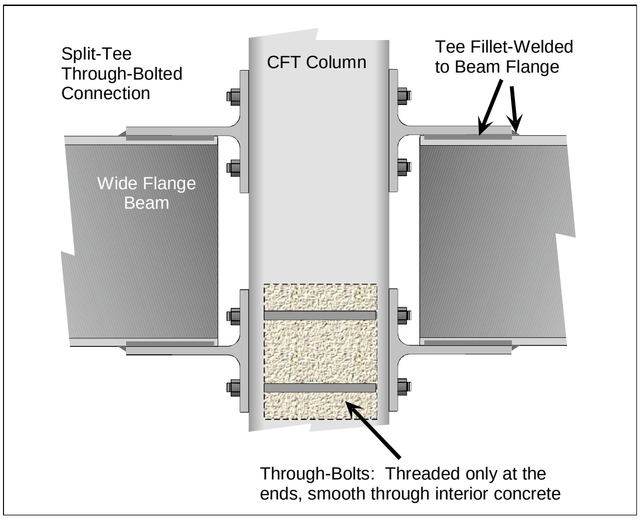

Figure 1 – Connection Detail

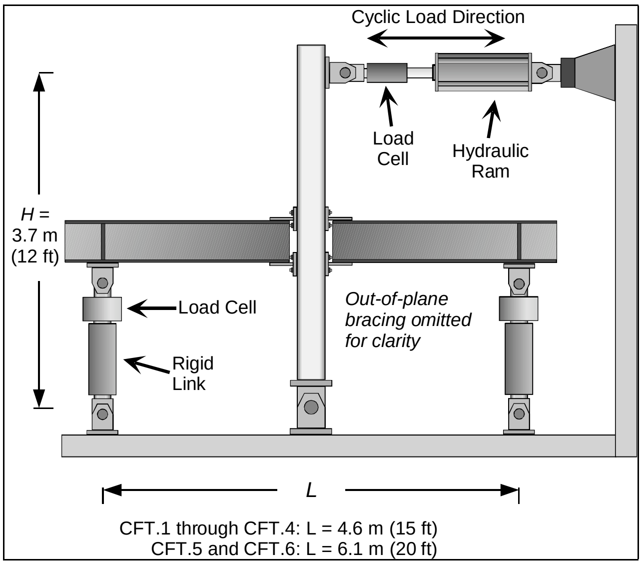

Figure 2 – Test Set-up for Full-Scale Connections

The first four specimens in this series, CFT.1 through CFT.4, were 305 mm (12 in.) square CFT specimens, and were designed by scaling the 406 mm (16 in.) CFT specimens by 3/4, and adjusting the beam sizes accordingly. The remaining two full-scale 406 mm (16 in.) CFT moment connections, CFT.5 and CFT.6, were based on the designe and construction of similar specimens tested at Lehigh University. Specimen CFT.5 is based on Lehigh [Ricles, 1997] specimen number 6, and has similar dimensions for the CFT and all of the connection components. Specimen CFT.6 was designed with a smaller tee width bearing against the face of the column. CFT.6 was designed to examine the effect of narrowing the tee width, while preserving the same beam and column properties as those used in CFT.5.

The column height for all six specimens was 3.7 m (12 ft.) to permit using the same test set-up for all six tests. The beam spans for CFT.1 through CFT.4 were based on their size relative to specimens CFT.5 and CFT.6. Thus the beam span *(L)* from inflection point to inflection point was 4.6 m (15 ft.) for CFT.1 through CFT.4, and

6.1 m (20 ft.) for CFT.5 and CFT.6. The dimensions and material properties of the CFT specimens used in Phase II are shown in Table 1 and Table 2

Table 1 – Material Properties of Full-scale CFT Specimens

Specimen	Square CFT mm (in.)	Concrete f'_c MPa (ksi)	Steel Tube Properties		
			f_y MPa (ksi)	f_u MPa (ksi)	% Elongation
CFT.1	305 (12)	50 (7.2)	366 (53)	416 (60)	26
CFT.2		50 (7.2)			
CFT.3		50 (7.3)			
CFT.4		51 (7.4)			
CFT.5	406 (16)	49 (7.14)	397 (58)	485 (70)	41
CFT.6		49 (7.14)			

Table 2 – Wide-Flange Beam Properties

Specimen	Beam Size (AISC)	Beam Properties		
		f_y MPa (ksi)	f_u MPa (ksi)	% Elongation
CFT.1, 2	W 18 x 46	379 (55)	513 (74)	25
CFT.3, 4	W 18 x 65	367 (53)	516 (75)	21
CFT.5, 6	W 24 x 62	389 (56)	503 (73)	25

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Beam Sizes

Beam sizes used for all six specimens are shown in Table 2. W18 structural shapes were used for the beams framing into the 305 mm (12 in.) CFT columns. Two different W18 shapes were used to develop different failure modes at the connection. The W18 x 46 beams of specimens CFT.1 and CFT.2 were used to allow the beams to form plastic hinges. W18 x 65 beams were chosen to produce panel-zone failures in specimens CFT.3 and CFT.4. The amount of over-strength in the W18 x 65 beams assured that minimal inelastic deformation would occur in the beams, and that the connection would have to dissipate energy through the connection or panel-zone. Deformation of the panel-zone and connecting elements was the intended mechanism of energy dissipation, although typical U.S. design practice requires a strong-column/weak-beam failure mechanism.

For CFT.5 and CFT.6, W24 x 68 beams were used. Both specimens were designed to fail through inelastic beam rotation near the face of the column, a failure mode consistent with current design practice. The main difference between CFT.5 and CFT.6 was the width of the tee acting on the face of the column. The intent was to determine the influence of the tee width on the transfer of shear forces through the connection even though the panel-zone itself was not designed to fail.

Testing Procedure

Testing of the CFT specimens was displacement controlled. The displacement was based on values of inter-story drift (% ISD), and was used consistently throughout the testing of all six CFT specimens. An identical displacement history was used at Lehigh University during the testing of similar connections. The displacement measurement at the top of the column was used to determine inter-story drift. The load actuator (ram) was controlled by hydraulic pressure, and load was held at each increment momentarily prior to data collection to reduce the influence of dynamic effects. Interstory drift levels ranged between 0.25 and 5.0 percent. To evaluate strength and stiffness degradation, cycles were repeated in the load history.

Preliminary Results and Trends

Based on the analyses completed to date, a few observations have been made. Results of shear panel distortion versus panel shear are presented in Figures 3 through 8. The values for panel shear are non-dimensionalized by comparing the measured values of panel shear with Equation 1, a hybrid equation for shear strength.

$$V_n = V_c + V_s \qquad \textit{where} \qquad V_c = C\sqrt{f_c'}\,b_e h \qquad \textit{and} \qquad V_s = 0.6 A_s F_y \tag{1}$$

$C = 0.63$ when units are: kips, ksi, and in^2

$C = 52.5$ when units are: kN, MPa (N/mm^2), and mm^2

Equation 1 is only used as a reference, and represents the superposition of the shear strength of a well confined concrete core within a reinforced concrete column joint, and the shear strength of the steel outer panels. The term $b_e h$ represents the cross-sectional area of the concrete core within the shear plane.

Specimens 2, 3, and 4 (Figures 3 through 5) were designed to fail in the panel-zone. In each case, peak shear was reached at 3.0% drift at values between 95% and 97% of the theoretical peak shear load calculated using Equation 1. Specimens 5 and 6 (Figures 6 and 7) were designed to develop beam-hinge mechanisms before the panel-zone failed. Panels zones in those two cases, which remained primarily elastic during the tests, reached values of about 60% of the theoretical peak shear capacity. Specimens 1 through 4 were taken to 5% drift before the tests were stopped. Specimens 5 and 6 were halted at 3.5% drift for technical reasons not related to the panel-zone. Figure 8 shows the results typical of the earlier half-scale panel zone experiments non-dimsionalized using the simplified model (Equation 1).

The concrete term b_e, the width of the concrete core, used in the comparison for Figures 3 through 8 is assumed to be the full width of the core of the actual specimens, although the load is applied in some cases to only a portion of that width. When the concrete term is scaled according to the actual width of the load application through the tees, all results become conservative, thus predicting values of shear strength well below that obtained in the lab.

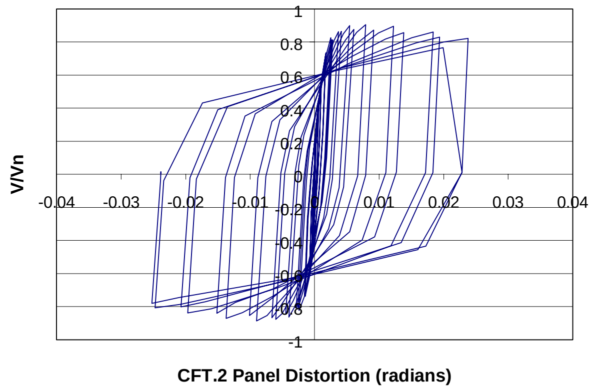

Figure 3 – Combination Panel/Beam Failure, Tee: 67% Width of Column

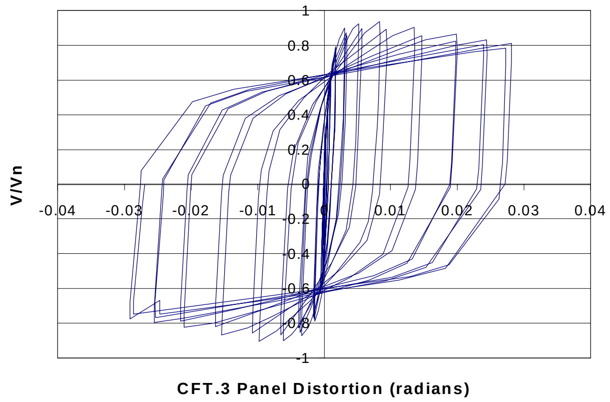

Figure 4 – Forced Panel-Zone Failure, Tee: 67% Width of Column

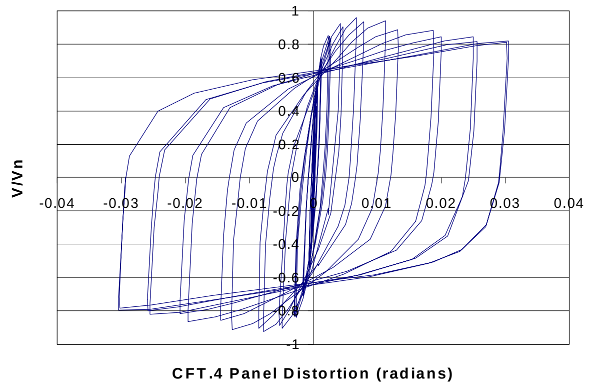

Figure 5 – Forced Panel-Zone Failure, Tee: 75% Width of Column

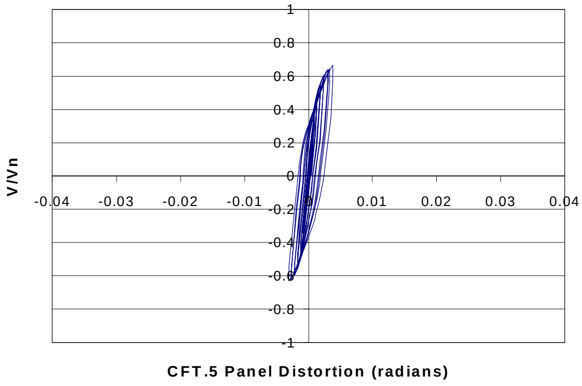

Figure 6 – Beam Plastic Hinge Mechanism, Tee: Full Width of Column

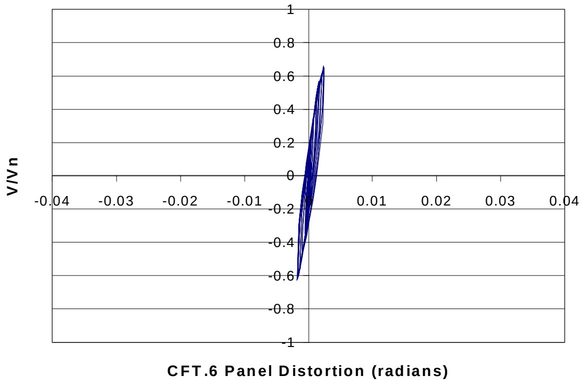

Figure 7 – Beam Plastic Hinge Mechanism, Tee: 56% Width of Column

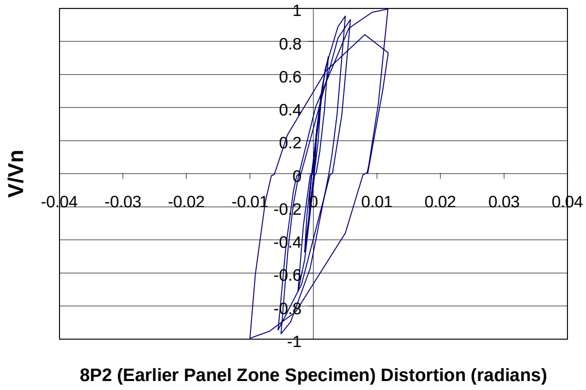

Figure 8 – Forced Panel Failure in Earlier Half-scale Panel Zone Specimen

Design Criteria

Work is continuing on the development of models for designing the panel-zone in the split-tee through-bolted connections. The aim of this project is to assist in the development of building code design recommendations for composite joints. Emphasis will be placed on the role of the concrete core in transferring joint shear forces through the connection. If such a joint is to be viable in US practice, it is imperative that the joints not require additional internal stiffening or complicated exterior details for force transfer between the beamts and the concrete filled tube column. A better understanding of the role of the concrete core will allow for practical and more economical designs with minimal additional detailing.

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