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Behavior of CFT Column-WF Beam Moment Connections Under Seismic Loading

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Synopsis: This paper describes research associated with the seismic behavior of moment connections for concrete filled tube (CFT) column-to-wide flange (WF) steel beam framing systems. The objective of this multiphase research program is to assess the force transfer mechanism in these connections, examining the effect various structural details have on this mechanism, as well as on the connection's strength, stiffness, and ductility. The first phase of the program was devoted to assessing the shear capacity of the panel zone in a CFT column-to-beam connection under simulated seismic lateral load conditions. The results from tests show that a CFT panel zone possesses exceptional ductility, including connections without interior diaphragms. In addition, a capacity equation based on the superposition of the shear strength contribution of the steel tube and concrete core within the panel zone provides a prediction that agrees reasonably well with specimen strength. The second phase of the research program involves full-scale structural connection subassemblage tests. Results from tests show that specimens in which the beams are designed to dissipate energy can have exceptional cyclic ductility. However, connections must be properly detailed to avoid strain concentrations which could lead to fracture. Measured deformations in the column show that a CFT column's initial stiffness is well estimated by transformed section theory. However, interstory drift deformations beyond 0.5% of the story height tend to reduce the stiffness after concrete cracking and debonding of the concrete from the steel tube.

Keywords: Beams; diaphrams; framing systems; lateral pressure; shear strength

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INTRODUCTION

Structural steel tubes filled with concrete are referred to as concrete filled tubes (CFT), and will be the type of composite column that will form the basis for this paper. The advantage of using a CFT composite column is that the steel tube provides permanent formwork for the concrete, thereby expediting construction. In addition, the steel provides confinement to the concrete inside the tube, whereas the concrete inhibits local buckling from occurring in the steel tube. In the event of a fire causing the expose of the CFT to extreme temperature, it has been shown (1) that the concrete core provides resistance that enables some limited amount of force redistribution to take place from the steel tube to the concrete, providing in the process some resistance to fire.

It has been reported by Griffis (2) that under axial loads, reinforced concrete columns are approximately 11 times more cost-effective than structural steel columns in terms of strength and stiffness, giving an indication of the economy of composite columns that make use of steel and concrete. In order to explore the feasibility of CFT columns, the perimeter moment resisting frame (MRF) shown in Figure 1 was designed. The frame consists of CFT columns and steel wide flange spandrel beams, and was designed for a seismic zone 4 using the NEHRP provisions (3) to establish the member forces under seismic and gravity loading conditions. The columns were sized on the basis of applying full composite action with strain compatibility, which is the recommended procedure per ACI 318-89 (4). The spandrel beams were sized using the seismic provisions in the AISC LRFD code (5), resulting in a weak beam-strong column system. In order to comply with the NEHRP provisions, it is required that the design meet interstory drift criteria. This criterion states that the interstory drift δ should not exceed 0.015h, where δ is based on the factored elastically computed drift δ_{xe} under the lateral seismic loads and h is the story height, with

$$\delta = C_d\, \delta_{xe} \tag{1}$$

In Eqn. (1) C_d is equal to 5.5 in accordance with NEHRP (3).

The design of the prototype perimeter MRF was found to be controlled by the interstory drift criterion. The factored interstory drift ratio δ/h for each floor and member sizes are summarized in Figure 1 (see CFT column). In the calculation of δ_{xe} the flexural stiffness property EI of the CFT column, where E is Young's modulus and I the moment of inertia, was based on uncracked transformed section theory, where

$$EI = E_s\left(I_s + \frac{I_c}{n}\right) \tag{2}$$

in which E_s, I_s, I_c, and n are Young's modulus of the steel tube, moment of inertia of the steel tube, moment of inertia of the infilled concrete's gross area, and the modular ratio between the steel and concrete (i.e., $n=E_s/E_c$), respectively. Assuming a 55 MPa compressive strength concrete and computing E_c on the basis of ACI criteria, the modular ratio is 5.9. Consequently EI is increased by an average of 59% compared to the EI for a 406 x 406 x 12.5 mm structural steel tube without concrete.

A similar frame using wide flange steel columns in place of CFT columns was also designed, and the drift and member sizes are given in Figure 1. This design was also controlled by drift. A comparison of the CFT and WF column systems indicates that the former requires 22% less weight of structural steel, representing an appreciable cost savings in material. Since drift controlled the CFT system, this comparison assumes that the elastic EI of the CFT column can be accurately predicted using transformed section modulus theory. In addition, the connections in these frames must be economical, otherwise the cost savings in this form of construction may be lost.

Under lateral seismic loading, the connections in CFT column MRF systems are subjected to large forces, resulting in the formulation of a large shear in the panel zone of a joint, as illustrated in Figure 2. The panel shear is the result of large beam flange forces that are transferred to the connection. To deal with the large beam flange forces, CFT construction in Japan has traditionally involved the use of interior or exterior steel plate diaphragms in the connection, as shown in Figure 3, to provide stiffening of the tube and a force path for the beam flange forces. While the use of diaphragms may improve performance and avoid premature connection failure, their incorporation into CFT moment connections increases significantly the fabrication costs.

The behavior of a connection can differ, depending on the proportioning of the joint's beams and columns, as well as the detailing of the structural connection elements, including diaphragms, weld details and bolts. The most suitable connection design involves plastic hinging in the beam, where energy dissipation and ductile behavior can occur. A ductile response involving panel zone yielding may also be acceptable, however, repair after an earthquake can be costly since column repair is involved.

While an assortment of research has been conducted on connections for steel construction involving box columns not filled with concrete (6,7) which U.S. designers have utilized for guidance, there is a lack of research and knowledge in the U.S. on the subject of the seismic behavior of connections for CFT column composite systems having steel beams. Previous research on CFT column systems in the U.S. has mainly included the experimental test programs by Furlong (8,9), which involved assessing the strength of circular and square CFT columns and beam-columns. The results of these tests demonstrated the benefits of filling a steel tube with concrete. The infilled concrete helped prevent local buckling of the tube's wall from occurring, in addition to causing an increase in strength relative to that of the bare steel tube to develop. In spite of these and other tests and design guidelines that appear in ACI 318-89 (4) and AISC (5), the use of composite construction in the U.S. appears to have gone beyond current information related to structural behavior and research. The newly published NEHRP Provisions (10) have seismic resistant guidelines for connections involving SRC construction, based mostly on the research by Deierlein et al. (11,12), however, no guidance is in existence related to the seismic resistant design of moment connections for CFT construction.

On the contrary, in Japan a vast amount of research has been conducted on composite construction, including connections for CFT construction, with seismic resistant design recommendations appearing in the AIJ provisions (13). Tomii et al. (14) conducted axial load tests on 268 CFT column specimens, including square and circular steel tubes. He found that although the confinement of the concrete by the steel tube is limited due to the Poisson effect causing the steel tube to expand radially at a greater rate than the concrete, specimens of thin wall thickness reached an axial load capacity corresponding to their nominal squash load by having the local buckling of the steel tube inhibited by the presence of the concrete. Research by Kato (15,16), Yokoyama et al. (17), Morino et al. (18), and Matsui (19) on CFT connections having interior and exterior diaphragms, respectively, have shown that these elements develop a complex stress state and are susceptible to fracture or local buckling. Cyclic tests on CFT moment connections were conducted by Kanatani et al. (20), as well as Morino et al. (18), in which shear yielding of the steel tube within the connection's panel zone occurred. Their test results demonstrated that a ductile hysteretic behavior in the specimens can be achieved, as long as shear buckling and fracture of the steel tube within the panel zone is inhibited. Design guidelines have resulted from the above investigations, with some appearing in the AIJ Standards for Composite Construction (13).

As noted already, in the U.S. there currently is a lack of seismic resistant design standards and general knowledge of the seismic performance of connections in CFT column-wide steel flange beam MRF systems. While the research conducted in Japan has resulted in seismic design provisions, the differences in construction practices does not lend itself well to directly using these guidelines in the U.S. Further research based on large scale tests is therefore required that would provide information and seismic performance of details that the U.S. may utilize.

A research program was therefore planned and conducted at Lehigh University associated with the seismic performance of full-scale moment connection subassemblies in CFT column-wide flange steel beam systems. The objective of this research was to examine the force transfer mechanism in the connection, and to evaluate the effects that connection details have on the cyclic strength, stiffness, and ductility. This information is intended to provide data and a basis for developing future recommendations for seismic resistant design. The research program consisted of several phases, and included both analytical and experimental studies. Reported herein are the first two phases, where Phase I has involved an experimental investigation concerned with the shear-deformation response of CFT panel zones under monotonically applied shear. Included is an analytical study involving the development of a new capacity model, and an evaluation of this as well as existing models' accuracy in predicting panel zone shear capacity. Phase II of the research program involved the testing of full-scale connection subassemblies under seismic loading conditions. The cyclic performance of connections having interior and exterior diaphragm details, respectively, was evaluated. The perimeter moment resisting frame previously shown in Figure 1 was adopted as a prototype for the test specimens of the research program.

CFT PANEL ZONE STUDY

Test Matrix

Under lateral loading, the shear developed in the panel zone of a CFT-to-beam moment connection is resisted by both the steel tube and concrete. The concrete's resistance to panel zone shear is by the development of a diagonal compression strut, as shown in Figure 2, having the inclination angle (or strut angle) of α. One purpose of the panel zone study was to assess the effect the interior diaphragm has on the compression strut and the overall performance of the panel zone. In addition to the diaphragm, other details which can affect the panel zone behavior include: the width-to-thickness (b/t) ratio of the steel tube in the panel zone; compression strut angle α; concrete compressive strength; and steel tube yield stress. To study some of these effects, four CFT panel zone specimens were designed and tested. The dimensions of the test specimens correspond closely to one-half scale of the columns in the lower floors of the prototype structure. The experimental study included a comparison of the specimen capacity with that predicted by analytical models.

A summary of the test matrix containing the four test specimens is given in Table 1. A photograph showing all four test specimens prior to placing concrete inside them is given in Figure 4. Square steel tubes of nominal width b equal to 203 mm and thickness t of 6.3 and 4.5 mm, respectively, were used. The details considered as parameters among the specimens were: interior diaphragms; b/t ratio; and compression strut angle α. The effects of these parameters on shear capacity were investigated by changing the details of the panel zone in accordance with Table 1. Specimen 1 of the test matrix was the only specimen which had interior diaphragms. Each diaphragm was placed inside the steel tube where simulated beam flange forces were applied to the outside wall of the specimen. The steel diaphragms were fitted inside the tube of Specimen 1, having an average measured thickness of 10 mm. A 76 mm diameter hole was placed in the center of each diaphragm plate, similar to that shown in the cross-section of the column in Figure 2, in order to enable concrete to be placed. Specimen 2 was identical to Specimen 1, except the former did not have any diaphragms. The b/t ratios of Specimens 3 and 4 were both increased relative to that of Specimens 1 and 2. While the nominal b/t ratio was the same for Specimens 3 and 4, Specimen 4 had a larger strut angle, which was accomplished by fabricating Specimen 4 to a greater length. The overall length of Specimen 4 was 610 mm, which was equivalent to twice the shear span d. The remaining specimens each had an overall length of 406 mm. The measured material properties for the steel tube and diaphragm yield stress (σ_y) and ultimate stress (σ_u), as well as the concrete compressive strength f'_c, are given in Table 2.

Test Setup

The testing scheme shown in Figure 5 was developed to simulate local force conditions applied to a panel zone, including compression beam flange forces and column axial force. The setup for the panel zone test shown in Figure 5 has two end reactions to resist a monotonically applied transverse load H at midspan (at a distance of d from each support), thus creating two panel zones in each test specimen. The magnitude of the end reactions was equal to the shear V developed in the panel zones. Rollers, in conjunction with 50 mm and 100 mm wide bearing plates placed across the specimen's width, were provided at the ends and midspan of a specimen. The bearing plates were each 12 mm thick, and had a length of 203 mm in order to span the width of the steel tube. A constant axial compressive load P was applied in the horizontal direction to both ends of the specimen to simulate column axial force effects. The axial load was applied by a hydraulic ram, using a 50 mm thick steel plate at each end of the specimen as bearing plates in order to distribute the load to the steel tube and concrete. Hydrostone was placed between each of these plates and the specimen ends to ensure a uniform bearing contact between the surfaces. Both ends of the test specimen were restrained from rotating. The axial force P was equal to 890 kN in Specimens 1 and 2, and 712 kN in Specimens 3 and 4. The magnitude of P was equivalent to one-half the nominal squash load N_o (i.e., $P=0.5N_o$), defined in accordance with AIJ provisions (13), and which corresponded closely to the estimated magnitude of the combined gravity and seismic axial load effect in a ninth floor column of the prototype structure, where:

$$N_o = \tfrac{1}{3} A_c f_c' + \tfrac{2}{3} A_s \sigma_y \qquad (3)$$

In Eqn. (3) A_c and A_s are the cross sectional area of the concrete core and the steel tube, respectively. Prior to testing, each specimen was instrumented in order to measure strain in the panel zone of the steel tube, displacement under the applied transverse load H, and end rotation. The applied axial and transverse loads were monitored using calibrated load cells.

Test Results

The shear-deformation (V-γ) response of each of the specimens is shown in Figure 6. The deformation γ was based on the midspan transverse displacement divided by the shear span *d*. Typical response of a specimen during testing involved an initial linear relationship between V and γ. As yielding developed in the steel tube of the panel zone under shear, a softening of the panel zone occurred. This phenomenon became more noticeable in specimens without diaphragms as the applied shear V surpassed the shear capacity V_s, the plastic shear capacity of the steel tube. With continued loading, the stiffness continued to decrease until it became zero, and the specimen capacity V_{exp} had been reached. The capacity V_{exp} of each specimen is summarized in Table 3. Specimen 1 had about a 30% greater capacity than corresponding Specimen 2, which is apparent in Figure 6(a) as well as Table 3. The additional strength achieved in Specimen 1 is attributed to the presence of interior diaphragms, which enhanced the concrete compression strut strength V_c by increasing the concrete strength and compression strut width. In addition, a higher local crushing strength of the concrete existed at the location of load application due to the added confinement provided by the diaphragms. Specimen 2 was found to have a 19% greater capacity than Specimen 3 (see Figure 6(b)). This increase in strength is mainly attributed to the larger wall thickness t of Specimen 2 which resulted in a greater shear strength V_s. Specimens 3 and 4 had a capacity V_{exp} of 589 kN and 578 kN, respectively, implying that the difference in the strut angle α of 45 and 56 degrees did not significantly affect specimen capacity.

Continued imposed deformation during testing showed that all specimens develop exceptional ductility. An increase in strength occurred under large displacements, as tension membrane action developed in the steel tube. All specimens eventually developed outward local buckling in the panel zones. For Specimens 2, 3, and 4 this occurred at a shear deformation of approximately γ=0.05 to 0.06 radians, taking the form of an outward uniform bulging of the panels. The interior diaphragms in Specimen 1 enabled a greater shear deformation γ of approximately 0.15 radians to develop before local buckling occurred along a diagonal of the panel zone. This diagonal local buckling resembled that which has been found to occur in stiffened panels of plate girders subjected to a high shear force. The presence of the concrete inside the steel tube enabled all of the specimens to develop significant post buckling strength and ductility. A photograph of Specimen 1 upon completion of the test is shown in Figure 7. The local crushing under the load point seen in Figure 7 occurred at a large γ, and was typical of all specimens.

Analytical Study

The results for specimen shear capacity V_{exp} were compared to that predicted by selected existing analytical models, as well as a newly developed formulation. All of these models were based on superimposing the shear strength V_c of the infilled concrete with that of the steel V_s, where the total shear capacity, V_{total}, is equal to their sum:

$$V_{total} = V_c + V_s \tag{4}$$

Since shear yielding occurred in the steel tube prior to local buckling, the steel contribution was based on considering the von Mises yield criterion for metals, where

$$V_s = \frac{1}{\sqrt{3}}\sigma_y A_w \tag{5}$$

in which A_w is the web area of the steel tube that creates the steel panel zone (i.e., A_w=2bt). The differences among the models was in the determination of the concrete strength V_c. The three existing models that were selected included: (1) Kanatani (20); (2) ACI 318-89 (4); and, (3) Architectural Institute of Japan Composite Construction Standards (13). The formulation by Kanatani bases the concrete shear capacity V_c on the strength of a diagonal compression strut that forms within the concrete (see Figure 2), where for a joint without interior diaphragms

$$V_c = S(D - t)f_c'\cos^2(\alpha) \tag{6a}$$

where S, D, and α are the bearing width of the beam flange on the column face, width of the column normal to the panel zone, and the strut's angle of inclination (see Figure 8(a) and (b)). The strut width S is associated with the bearing width developed by the beam flanges acting on the outside column face. For joints having interior diaphragms it is postulated by Vermaas et al. (21), because the diaphragms tend to increase the strut's width, that V_c is twice that of Eqn. (6a),

$$V_c = 2S(D - t)f_c' \cos^2(\alpha) \tag{6b}$$

The shear strength contribution of the concrete V_c based on ACI 318-89 (4) involved referencing Section 21.6.3 - Shear Strength, where for joints without interior diaphragms the analogy of a reinforced concrete joint confined on two opposite faces was used, hence

$$V_c = \frac{(b - t)(D - t)1.246\sqrt{f_c'}}{1000} \qquad (f'_c \text{ in MPa}) \tag{7a}$$

The confinement is assumed to be supplied by the walls of the tube on the two opposite faces of the panel zone. For joints with interior diaphragms, a well confined reinforced concrete joint condition was assumed, where

$$V_c = \frac{(b - t)(D - t)1.661\sqrt{f_c'}}{1000} \qquad (f'_c \text{ in MPa}) \tag{7b}$$

The AIJ provisions for the ultimate strength of the joint consists of an empirical expression for V_c that is based on experimental data, where

$$V_{total} = (b - t)(D - t)({}_jf_c)({}_j\beta) + 1.2\frac{\sigma_y}{\sqrt{3}}2bt \tag{8a}$$

In Eqn. (8a), ${}_jf_c$ is the concrete shear strength, which is based on the mean value of several experimental tests:

$${}_jf_c = \min\left(0.12f_c', \; 18 + 0.036f_c'\right) \qquad (f'_c \text{ in kg/cm}^2) \tag{8b}$$

and,

$${}_j\beta = \frac{2.5b}{d_b} \leq 4 \tag{8c}$$

in which d_b is the beam's depth. As noted, Eqn. (8a) represents the mean of experimental data for specimen strength; the second term is the von Mises yield criteria multiplied by a factor of 1.2.

A formulation that was developed was based on superimposing the shear capacities of the concrete (V_c) and steel (V_s). In this formulation V_s is based on the von Mises yield criteria for the steel tube within the panel zone, and V_c the contribution from a diagonal compression strut force C (see Figure 8(c)). Under the actions of the bending moments in the column (M_{col}) and beams (M_{beam}), combined with the column axial force (P_{col}), a node is formed at the ends of the diagonal compression strut that is classified as a CCC-node per Schlaich et al. (22) along the lines of a strut and tie model. Within the nodal region the zone of concrete opposite the compression beam flanges is in a state of planar hydrostatic compression stress. Assuming the borderline of the nodal region to be perpendicular to the resultant of the stress field along the three edges per Schlaich et al. (22), and by consideration of the geometry of Figure 8(c) where the beam flange bearing width is S, the contribution of the strut force C within the strut to V_c is equal to

$$V_c = S(D - t)f_{cd} \tag{9a}$$

where f_{cd} is the compressive strength of the strut C. The resistance f_{cd} is based on that recommended by Schlaich et al. (22) for determining the capacity of concrete compressive struts in strut and tie models for concrete design. On this basis,

$$f_{cd} = \beta\, 0.85 f_c' \tag{9b}$$

where

$$\beta = \begin{cases} 0.4\,, & \text{panel zones without interior diaphragms} \\ 1.0\,, & \text{panel zones with interior diaphragms} \end{cases} \tag{9c}$$

A value of 0.4 for β is chosen for a panel zone having no interior diaphragms, for cracks of noticeable crack width were observed in the concrete of Specimens 2, 3, and 4, which were skewed with respect to the angle α of the panel zone. In Specimen 1, the concrete within the area of the strut had essentially no tension cracking, and therefore the analogy of a uniaxial compressive stress state was adopted, for which Schlaich et al. (22) recommends a value of β equal to 1.0.

A comparison of the predicted panel zone shear capacity V_{total} for each specimen with their corresponding experimental capacity V_{exp} is given in Table 3, where the ratio V_{exp}/V_{total} is summarized for each analysis model. The comparison shows that the model proposed by Kanatani provides results that agree reasonably well with the test results, where the ratio V_{exp}/V_{total} has a range in value from 0.92 to 1.11, a mean of 0.99, and a coefficient of variation of 8.8%. The assumption of doubling the strut width within the panel zone of specimens having an interior diaphragm resulted in a prediction of the experimental capacity that is within 7%. The use of the ACI provisions to compute V_c resulted in a range in value of 0.85 to 0.98 for the ratio V_{exp}/V_{total}, with a mean and coefficient of variation of 0.90 and 6.4%, respectively. Assuming a well confined joint, and applying Eqn (7b) for panel zones with interior diaphragms led to a precise prediction of the capacity of Specimen 1. While the AIJ provisions are shown to predict reasonable well the capacity of Specimen 1, the agreement between the other specimen's experimental