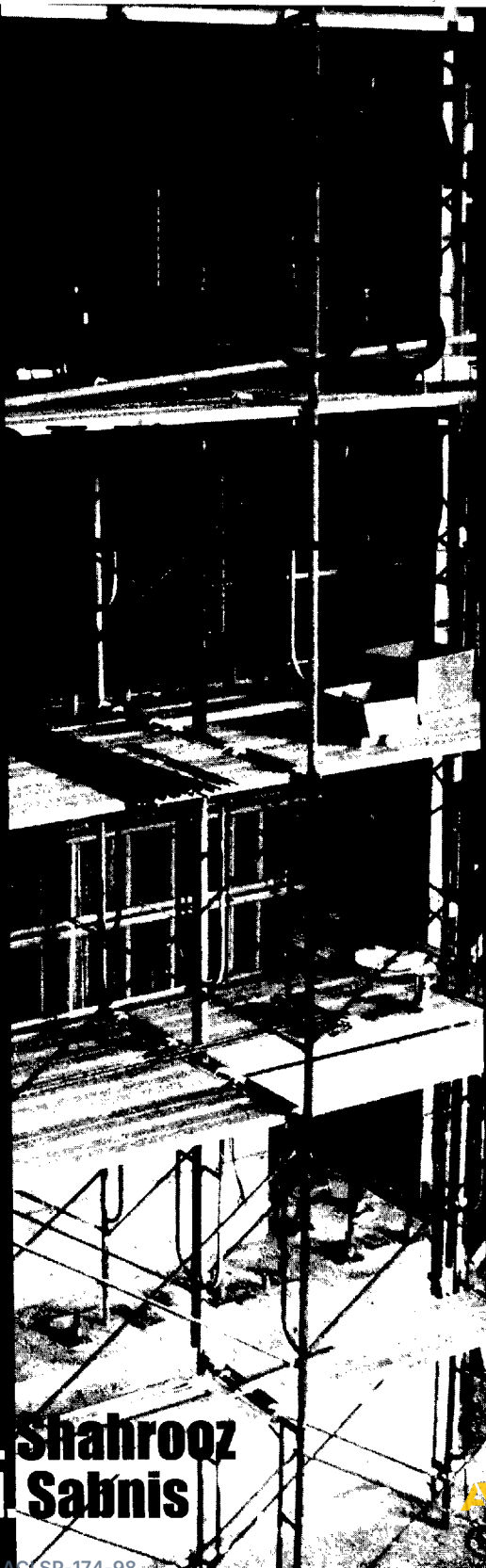




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Hybrid and Composite Structures

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PREFACE

Innovative design applications and advanced research has led to widespread use of steel and concrete composite and hybrid systems in the construction of buildings, bridges, and many other types of civil structures. The state of the art in this field continues to move forward today. Extensive research programs and field testing have yielded efficient, reliable, and safe procedures, incorporating these two dissimilar materials for overall improved construction. This publication presents an overview of the latest developments in behavior and design of composite and hybrid structures.

In 1995 ACI Committee 335 sponsored two technical sessions in Montreal, Quebec on the current practice of composite and hybrid construction and the state of the art in the field. Researchers and practicing engineers from the United States, Europe and Japan gave presentations encompassing topics related to design and construction of composite and hybrid systems and the advancement of research in three continents.

The twelve papers appearing in this volume include topics presented in Montreal, along with additional manuscripts. The breadth and depth of the material covered make this publication a useful resource to practicing engineers, educators and researchers.

Acknowledgments

The editors sincerely thank Professor Richard N. White of Cornell University for all of his efforts to make this publication possible; all of the members of ACI Committee 335 who volunteered their time and expertise to review the manuscripts; and finally, ACI staff for help in bringing the hard work of so many people together in this volume.

Bahram M. Shahrooz
Gajanan M. Sabnis

Editors

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

James M. Ricles, Le W. Lu and Took Kwong Sooi

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 subassembly 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; diaphragms; 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

$$\delta = C_d \delta_{xe} \quad (1)$$

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

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

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

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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 AII 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

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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 AII 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:

$$N_o = \frac{1}{3}A_c f'_c + \frac{2}{3}A_s \sigma_y \quad (3)$$

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.

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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 $\gamma=0.05$ 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:

$$V_{total} = V_c + V_s \quad (4)$$

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

$$V_s = \frac{1}{\sqrt{3}} \sigma_y A_w \quad (5)$$

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

$$V_c = S(D - t)f'_c \cos^2(\alpha) \quad (6a)$$

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),

$$V_c = 2S(D - t)f'_c \cos^2(\alpha) \quad (6b)$$

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

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

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

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

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

$$V_{\text{total}} = (b - t)(D - t)(j\beta f'_c) + 1.2 \frac{\sigma_y}{\sqrt{3}} 2bt \quad (8a)$$

In Eqn. (8a), $j\beta f'_c$ is the concrete shear strength, which is based on the mean value of several experimental tests:

$$j\beta f'_c = \min(0.12f'_c, 18 + 0.036f'_c) \quad (f'_c \text{ in kg/cm}^2) \quad (8b)$$

and,

$$j\beta = \frac{2.5b}{d_b} \leq 4 \quad (8c)$$

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

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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

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

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,

$$f_{cd} = \beta 0.85f'_c \quad (9b)$$

where

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

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

capacity and predicted strength is not as good, resulting in a range in value of 0.70 to 0.90 for the ratio V_{exp}/V_{total} , along with a mean and coefficient of variation of 0.80 and 11.4%, respectively. The reason for the greater discrepancy involving the specimens without interior diaphragms (e.g., Specimens 2, 3, and 4) is that Eqns. (8a) through (8c) are based on an experimental data base that is largely dominated by specimens having connections with interior diaphragms. The proposed model, where V_c is based on the capacity of a strut provides the best overall prediction for specimen shear strength, where the range for V_{exp}/V_{total} is 1.02 to 1.05, with a mean and coefficient of variation of 1.03 and 1.8%, respectively.

CONNECTION STRUCTURAL SUBASSEMBLY STUDY

Test matrix

In Phase II of the research program, three beam-to-column connection subassemblies were tested. These connections are referenced as Specimens C1, C3, and C3R. Each test specimen was comprised of a full moment connection between an interior CFT column and two wide flange steel beams. Schematics of each of the three specimens are shown in Figure 9 through 11. For each specimen the beams were W24x62 sections and the column a steel tube of 406 x 406 x 12.5 mm dimensions. Specimen C1 had a set of interior diaphragms in the connection, with Specimens C3 and C3R having extended structural tees. The proportioning of the member sizes of the specimens corresponded to a full-scale model of the members in the ninth floor of the prototype perimeter CFT moment resisting frame (see Figure 1). Measured dimensions and material properties for the beams and column of each specimen are given in Tables 4 and 5, where f'_c in Table 5 is the concrete compressive strength at the day of testing. Included in Tables 4 and 5 are the flexural capacities of the W24x62 beams (M_p) as well as the CFT column (M_n), where the latter is subjected to the axial force P which simulated gravity load. The beam flexural capacity M_p and column resistance M_n for each specimen was based on the measured geometry and material properties. The average axial load P applied to each specimen was 2050 kN, which corresponded to about 16% of the column axial load strength. The column flexural strength M_n was computed per ACI criteria without applying material reduction factors.

The connection elements, column, and panel zone were designed to resist the forces caused by a moment of $1.25M_p$ developing in both beams in order to ensure a weak beam-strong column system, whereby primary yielding and energy dissipation occurred in the beam. The panel zone's shear capacity for each specimen was checked using the AII provisions (e.g., Eqns. (8a) through (8c)) to ensure that a shear failure in the panel zone did not occur. The AII provisions were used in lieu of the other models because these provisions were based on tests in which the specimens had similar connection details involving shear tabs and exterior or interior diaphragms as in the current study.

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The interior set of diaphragms of Specimen C1 were located opposite the beam flanges, using full penetration welds in conjunction with the self shielded flux core arc welding (FCAW) process to attach them to the inside of the steel tube (see Figure 9). The beam flanges were also welded to the column, using full penetration flux cored arc welding. The backing bars were subsequently removed, and the exposed weld back gouged and then profiled with a 6 mm size fillet weld in order to improve the quality of the full penetration beam flange welds. A set of 89 mm long cover plates were subsequently welded to the beam flanges and column of Specimen C1, as shown in Figure 9. This was done in order that additional cover plates of greater length could be later added to strengthen the beams for future tests intended to fail the panel zone. By pre-attaching the cover plates in this manner the additional cover plates could be attached to the beam without heating and thereby affecting the concrete in the steel tube. In Specimen C1, as well as the other test specimens, the shear tab was bolted to the beam web using five-A325 25 mm diameter bolts and then partially fillet welded. The placement of fillet welds along the shear tab, as shown in Figure 9, was required in accordance with AISC LRFD seismic design provisions (5), since the plastic section modulus of the flanges accounted for less than 70% of the plastic moment capacity M_p of the beam. In addition, in all specimens six-16 mm diameter, 114 mm long shear studs were used to transfer each beam gravity load to the concrete and provide resistance to prying caused by the bending moment developed in the shear tab.

Specimen C3 had a pair of ST7.5x25 structural steel tees attached to the beam flange and column as shown in Figure 10 using full penetration welds. The tees were of 241 mm length and designed to transfer to the steel tube within the panel zone the beam tension flange force. Specimen C3R represented a retrofit of Specimen C3, following the testing of the latter, in order to enhance its cyclic performance. This was accomplished by welding tapered, vertical steel plates of 230 mm length to each beam flange to create a condition of a more gradual transition of cross-section from the beam to the face of the column, as shown in Figure 11. This was necessary in order to reduce the magnitude of strain that became concentrated at the end of the connection detail of Specimen C3, and which caused fracture of the beam flanges during testing.

Test Setup

In order that representative forces develop and boundary conditions exist in the vicinity of each connection, each specimen was tested as a structural subassembly that is shown in Figure 12. The ends of the beams and column of the subassembly corresponded to points of inflection at midspan and midheight in the prototype frame beams and column when subjected to lateral loading (see Figure 12(b)).

The column of the test setup was axially loaded by a constant gravity load P using a system of two tension rods in conjunction with two hollow core

actuators and a loading beam. A horizontally placed actuator was used to apply a lateral cyclic displacement history to the top of the column in order to develop overturning moments and forces in the specimen members and connection corresponding to earthquake loading conditions. The lateral loading history for each specimen corresponded closely to the recommendation by ATC-24 (23) for the cyclic seismic testing of structural steel components. A loading history basically consisted of initial sets of elastic cycles of progressively greater amplitude, followed by sets of inelastic cycles also of progressively greater amplitude. The amplitude of the inelastic displacement cycles were proportional to the displacement at first yielding of the specimen. Conditions of zero moment at the ends of the beams and the ability for the subassembly to move laterally in accordance with prototype motion, as shown in Figure 12(b), was accomplished by the use of cylindrical bearings and rigid links in the test setup as shown in Figure 12(a). Lateral bracing was used to prevent lateral, out-of-plane buckling of the beams and column.

The instrumentation plan for each specimen included: calibrated load cells to measure the applied lateral load H , gravity load P , and beam reactions developed in the rigid links; displacement transducers to measure the beam and column displacements at various locations along these members, as well as shear deformation in the connection's panel zone; rotation transducers to monitor the rotation at the ends of the beams and column, respectively; and, strain gages to measure the local strain in the beam flanges, connection diaphragm, and steel tube. The above measurements enabled lateral load-interstory drift ($H-\delta$) response at the top of the column, the panel zone shear-deformation ($V-\gamma$) relationship, beam moment-plastic rotation ($M-\theta_p$) behavior, as well as the contributions of the column, panel zone, and beam to interstory drift to be determined. Prior to testing, the steel members of a specimen were painted with whitewash in order to observe any signs of yielding by cracking and flaking of the applied coating.

Test Results

The lateral load-interstory drift ($H-\delta$) beam moment-plastic rotation ($M-\theta_p$) hysteretic response for Specimen C1 is shown in Figures 13 and 14, respectively. Before testing Specimen C1, an accidental lateral overloading occurred, causing the specimen to develop inelastic deformation in its beams, in the east (push) direction of lateral column displacement. This inelastic deformation amounted to a residual displacement of 51 mm in the push direction from the vertical plumb position at the top of the column. The displacement history of this specimen was therefore modified to include 6 initial elastic cycles before subjecting it to inelastic deformation in the west (pull) direction in order to straighten out the column to the vertical position. This required a total of about 100 mm of horizontal displacement to be imposed to the top of the column in the pull direction, in order to arrive at the plumb position (which, after elastic unloading corresponded to the origin in Figure 13). Unless otherwise noted, all response

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quantities, including lateral displacement ductility μ , panel zone deformation γ , and plastic beam rotation θ_p , are referenced from the position with the column plumb (i.e., from the origin $\delta = 0$ mm in Figure 13). Since no data was acquired during the accidental overloading, only the modified elastic cycles and subsequent inelastic cycles are shown in Figure 13.

The response of Specimen C1 resulted in a maximum displacement ductility μ equal to 3, corresponding to the interstory drift δ of 122 mm before testing was stopped. Prior to stopping the test, the response of the specimen was exceptional, with good hysteretic behavior and ductile performance. The beams of Specimen C1 developed yielding in their flanges at the end of the cover plates when the lateral displacement of the top of the column reached 41 mm (as attested by the flaking of white wash which was placed on the specimen). The corresponding displacement ductility μ was equal to 1.0. By the end of the test the flange yielding had attenuated away from the column over a length of one beam depth (e.g. 610 mm), and into the web adjacent to the flanges. Minor local beam flange buckling was observed to occur when the lateral displacement approached 82 mm, corresponding to $\mu = 2$.

During the displacement cycles corresponding to $\mu = 3$, the steel tube wall in the panel zone developed minor yielding due to shear, where the shear strain approached a deformation of $\gamma = 0.003$ radians. This amount of shear strain just exceeds the steel tube shear yield strain, and was found from the strain rosette data to be fairly uniform across the panel. Some prying and bulging of the column walls adjacent to, and between beam flanges, was also observed in the joint region when $\mu = 3$. The column outside the joint region was found to remain essentially elastic.

At a displacement corresponding to the second cycle with an amplitude of $\mu = 3$, fracture occurred in the beam flanges at the end of the 89 mm long cover plates. This fracture originated at the toe of the weld and attenuated across the beam flange. Necking of the flange material at the fracture indicated that a significant amount of high strain had developed in the base metal, which was attributed to a stress concentration. It was also observed that fractures had initiated in the steel tube, adjacent to where the beam flanges were welded to the column. These fractures indicated that the tube was locally over stressed in the wall by an outward prying force of the attached beam tension flange. A photograph of Specimen C1 corresponding to $\mu = 3$ is given in Figure 15, where yielding in the beam flanges and web is apparent from the flaking in the whitewash. The cracks in the beam flanges at the ends of the cover plates are seen in the left (east) beam. Prior to fracturing, Specimen C1 developed a cyclic plastic beam rotation of $\theta_p = 0.027$ radians (referenced from the position in which the column was straightened to the vertical, see Figure 14). This amount of plastic beam rotation is considered satisfactory, compared to prior and recent tests involving W-shape beams and hollow steel box columns (7) and W-shape

columns (24). Upon developing significant fracture, the test was stopped in order to evaluate and improve the connection detail such that a greater amount of inelastic deformation could be achieved in further tests of the specimen.

The lateral load-drift ($H-\delta$) and beam moment-plastic rotation ($M-\theta_p$) response for Specimen C3 is shown in Figures 16 and 17. Specimen C3 developed an interstory drift displacement of δ equal to 112 mm (where $\mu = 3$), before fracture also occurred in one of the beam flanges. This fracture originated at the end of the structural tee, near the toe of the weld which attached the structural tee to the edge of the beam flange. Strain gage readings indicated that the strain at the location of fracture was probably close to the material fracture strain. The test was then stopped in order to attempt to repair the specimen for a subsequent test.

Prior to fracturing, Specimen C3 also reflected good cyclic response, which included no deterioration in cyclic strength nor stiffness, as well as good energy dissipation. The plastic beam rotation θ_p achieved was 0.025 radians (see Figure 17). Furthermore, the steel tube of the panel zone developed a maximum deformation of $\gamma = 0.004$ radians, which was almost twice the shear yield strain. By the end of the test the beam flanges had yielded over a length of one beam depth (i.e., 610 mm) from the face of the column, with pronounced yielding occurring in the beam flanges outside the area of the structural tees, and over a distance of 100 mm from the end of the tees in the direction towards the column. Some yielding also was observed to occur in the flanges of the structural tees at 51 mm from the column face. This observed yielding and panel zone strain indicated that the structural tee connection detail was effective in developing the steel tube panel in shear to resist the beam tension flange forces. A photograph of Specimen C3 taken when μ was equal to 3 is shown in Figure 18, where yielding in the beam and structural tee is apparent from the flaked whitewash. It was determined that the high strain and subsequent fracture was due to a stress concentration at the origin of fracture. The detail for Specimen C3 therefore was modified in order to enhance the beam's ability to develop a larger amount of cyclic inelastic deformation.

To develop a detail that would improve the performance of Specimen C3, a nonlinear finite element analysis (25) of the connection detail was first performed using the computer program ABAQUS (26). The analysis results confirmed that a strain concentration existed in the beam tension flange at the end of the connection where the tee flange terminated. To reduce this concentration of strain, tapered vertical plates were attached to the beam flange in the finite element model (similar to the detail shown in Figure 11). This modified detail was shown by a subsequent analysis to significantly reduce the concentration of tensile strain in the beam flange.

Based on indications that this detail would enhance the performance, Specimen C3 was repaired and retrofitted with plates, becoming Specimen C3R.

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The lateral load-drift (H - δ), beam moment-plastic rotation (M - θ_p), and panel zone shear force-deformation (V - γ) hysteretic response for Specimen C3R is shown in Figures 19 through 21.

During the testing of Specimen C3R, at a displacement ductility μ equal to 2.5 corresponding to an interstory drift displacement δ of 91 mm, on the south side of the subassembly a fracture initiated in the west beam connection along the vertical weld between the bottom tee flange and the column. This occurred when the west beam bottom flange developed tension. Continued cyclic loading led to a crack growth and deterioration of the ability of the connection to transfer the west beam moment to the column, where at a displacement amplitude δ of 110 mm in the pull direction corresponding to μ equal to 3 a complete fracture of this weld occurred. This was followed by fracture through the north side structural tee of the west beam in the subsequent cycle of displacement having an amplitude of $\mu = 3.5$. This subsequent displacement corresponded to an interstory drift of δ equal to 128 mm in the pull direction. It was later determined that the cause of the initial fracture in the south side structural tee flange weld was associated with the lack of a fillet weld behind the flange, where a 6.4 mm size fillet was specified but inadvertently omitted during fabrication (see Detail I in Figures 10 and 11). The subsequent fracture of the corresponding north side tee, which had the required 6.4 mm fillet weld, was due to its inability to resist the entire west beam tension flange force, causing this tee to fracture through the base metal. The maximum plastic rotation θ_p that developed in the west beam prior to fracture was equal to 0.03 radians.

Since the east beam connection was intact, the test was continued by releasing the rigid link at the end of the west beam to deactivate it and cause only the east beam and column to provide resistance to the laterally applied load H . This resulted in a reduction in the resistance of the subassembly to about one-half its resistance prior to when fracture occurred, as evident in Figure 19 at a lateral displacement δ of 128 mm.

Prior to fracture, a significant amount of flexural yielding of the beam flanges and shear yielding of the steel tube within the panel zone was visible. A noticeable amount of shear distortion of the panel zone was also present compared to the other test specimens. The maximum shear in the panel zone was determined to be 3230 kN, with the shear deformation γ equal to 0.009 radians, which is equivalent to about four times the yield deformation. The shear-deformation (V - γ) relationship of Specimen C3R (see Figure 21) shows some energy dissipation to have occurred in the panel zone. Some yielding was also present in the web and flanges of the structural tees, however, most of the yielding was concentrated at the end of the connection in the beam flanges, where the onset of local flange buckling was also observed.

Continued cyclic loading with only the east beam active resulted in pronounced local flange buckling and additional yielding, where the local buckle had a half wave length of about 300 mm. As the displacement amplitude was further increased, the effect of cyclic beam flange buckling resulted in a gradual reduction in beam flexural capacity, as shown in Figure 20. The hysteretic response involved exceptional cyclic energy dissipation. A photograph of Specimen C3R corresponding to a displacement of $\mu = 4.5$, showing panel zone and beam yielding in addition to flange buckling, is given in Figure 22. After imposing three cycles of lateral displacement δ of 183 mm, corresponding to a ductility μ of 5 and an interstory drift ratio δ/h of 7.2%, the test was stopped. The maximum amount of plastic rotation θ_p developed in the east beam was equal to 0.065 radians.

The response of the connection involving the east beam was outstanding, illustrating that this type of detail can supply exceptional cyclic ductility. However, it also illustrates that satisfactory welds must be placed in the connection in order to avoid the type of fracture observed in the lower flange connection detail of the west beam.

Assessment of Test Results

From the specimen measured response it was possible to determine the contributions of the column deformation δ_c , beam and connector element deformations δ_b , and panel zone deformation δ_p , to the subassembly's interstory drift. The relationship between the deformations δ_c , δ_b , and δ_p to the interstory drift are plotted in Figure 23 for Specimens C3 and C3R. The response of Specimen C1 is not shown, but is similar to that of Specimen C3. The results shown in Figure 23 correspond to the peak displacement of the first half cycle of each displacement amplitude of the lateral loading history. It is seen in Figure 23 that the relationship between δ_p and H , as well as δ_c and H becomes nonlinear as the peak lateral load H is reached. The non-linearity in the H - δ_p relationship is associated with yielding of the steel tube and cracking of the concrete within the panel zone. In the H - δ_c relationship it is associated with cracking of the concrete, debonding of the steel tube from the concrete, and flexural yielding of the steel tube near the connection in the column under cyclic loading.

A more significant amount of non-linearity is seen in the H - δ_b relationship, and is associated with the beams developing inelastic deformation in the specimens. This is expected since the design was based on a weak beam-strong column concept. As noted previously, Specimens C3 and C3R developed a plastic beam rotation θ_p of 0.025 and 0.065 radians, respectively. The abrupt decrease in lateral load shown in Figure 23(b) that is accompanied by a reduction in panel zone and column deformation after achieving the peak load is associated with the deactivation of the west side beam of Specimen C3R. Upon deactivating this

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beam, the panel zone shear in Specimen C3R was reduced by almost a factor of two, leading to a reduction in the panel zone deformation δ_p .

The H - δ_c relationship was studied in an attempt to evaluate the flexural stiffness EI of the column. Shown in Figure 24(a) and (b) are the envelopes of the H - δ_c relationship corresponding to the first cycle of each amplitude of the displacement history for Specimens C3 and C3R. Included in these figures are elastic predictions of the column's deformation $\delta_{c,th}$, for a specified lateral load H , where

$$\delta_{c,th} = \frac{H(h - d_b)^3}{12EI} \quad (10)$$

For each specimen three predictions for $\delta_{c,th}$ were made by using EI for the column based on: (1) uncracked transformed section modulus (EI_u) per Eqn. (2); (2) ACI provisions for column stiffness (EI_{ACI}); and, (3) the column stiffness (EI_s) considering only the steel tube to resist the deflections. The value for EI_{ACI} was based on

$$EI_{ACI} = \frac{E_c I_g}{5} + E_s I_s \quad (11)$$

where E_c , I_g , E_s , I_s are the modulus of elasticity of the concrete, gross moment of inertia of the concrete core, modulus of elasticity of steel, and moment of inertia of the steel tube, respectively. Note that Eqn. (11) is intended to be used for design involving stability and second order effects and not necessarily for the lateral stiffness of CFT columns due to flexural bending. An examination of Figure 24(a) indicates that the initial stiffness of the column of Specimen C3 is well predicted up to an interstory drift corresponding to $\delta = 0.005h$ using the uncracked transformed section modulus. This was found to be also the case for Specimen C1. As the displacements increase, however, the column stiffness decreases due to concrete cracking and debonding of the steel tube from the concrete. In Specimen C3R it is apparent that the previous cyclic loading history, imposed on the specimen during testing as Specimen C3, had an effect on the initial elastic stiffness of Specimen C3R. The lateral stiffness of Specimen C3R is much closer to that of a specimen having a steel tube column with no filled concrete (i.e., EI_s).

The proportioning of the connection for each specimen was based on the weak beam-strong column design (which also satisfied drift requirements in the prototype frame), where as noted previously the design moment in the beams was equal to 1.25 times their flexural capacity (i.e. $1.25M_p$). This assumption was based on test results of prior researcher (27). The maximum beam moments, $M_{max,conn}$, developed in the specimens at the end of the connection are shown in the M - θ_p beam relationship (see Figures 14, 17, and 20) to exceed their flexural

capacity M_p that is based on the beam's measured dimensions and yield strength. The ratio of $(M_{\max}/M_p)_{\text{conn}}$ for each of the beams is given in Table 6, where the value for $M_{\max, \text{conn}}$ corresponds to the maximum beam moment at the end of the connection, and is shown to range from $1.13M_p$ (Specimen C3) to $1.23M_p$ (Specimens C1 and C3R). Hence the assumption of designing for a maximum beam moment of $1.25M_p$ appears reasonable due to the fact that under significant cyclic loading strain hardening develops in the severely yielded beams. Specimen C3 had a smaller maximum beam moment of $1.13M_p$ compared to that of Specimens C1 and C3R, due to the fact that Specimen C3 was not subjected to as large an amplitude of inelastic deformation.

The plastic rotation θ_p of the beams of each specimen was based on the following expression

$$\theta_p = (\theta - \theta_{el}) \frac{0.5L^*}{L_{bm}} \quad (12)$$

where θ , θ_{el} , L^* , and L_{bm} are the measured beam rotation at the face the column, corresponding measured elastic beam rotation, distance between the beam end reaction and the face of the column, and the distance from the end of the beam to its plastic hinge location, respectively. The maximum plastic beam rotations $\theta_{p, \max}$ of 0.027, 0.025, and 0.065 radians that developed in Specimens C1, C3, and C3R, respectively, are of significant amount, particularly that which developed in Specimen C3R. In the aftermath of the recent 1994 Northridge earthquake, in which several welded steel wide flange beam-to-wide flange column moment connections fractured, it appears (24) that for these connections a required ductility (i.e., $\theta_{p, \max}$) of 0.02 to 0.04 radians may be stipulated in future steel codes.

Due to the moment gradient the beam moments at the face of the column ($M_{\max, \text{face}}$) were larger than those at the end of the connection $M_{\max, \text{conn}}$. However, with the addition of the cover plates or structural tees to form the connection the beam moment capacity at the column face was also larger. The ratios of maximum beam moment-to-flexural capacity $(M_{\max}/M_p)_{\text{face}}$ for the beams at the column face were 0.67 for Specimen C1, and 0.61 and 0.73 for Specimens C3 and C3R, respectively (see Table 6). The corresponding maximum column moments $M_{\max, \text{col}}$ developed at the face of the joint are summarized also in Table 6, where they are also compared with the flexural resistance $M_{\text{cap, ACI}}$ per ACI 318-89 (4) and $M_{\text{cap, AISC}}$ per AISC plastic design provisions (28), respectively. The AISC plastic analysis ignores the contribution of the concrete to the column resistance. All capacities were computed using the measured dimensions and material properties. The results shown in Table 6 indicate that the maximum column moment in Specimens C1 and C3 are close to that of the bare steel tube, whereas in Specimen C3R the maximum column moment $M_{\max, \text{col}}$ is greater than the steel tube capacity. However, all of the maximum column moments in the specimens were

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considerably less than the capacity based on the ACI method for analyzing a composite column.

Measured longitudinal strains in the connection region of Specimen C3 are plotted in Figure 25, corresponding to the peak lateral displacements of the first half cycle for each level of displacement ductility μ when displacing the column of the specimen in the west (pull) lateral direction. These results show the effect that the stiffness of the steel tube wall has on the force transfer mechanism associated with the steel beam flange force. When a beam flange is in tension, the flexibility of the steel tube wall results in a majority of the flange's tension force being transferred through the flanges of the structural tees that are welded to the corner of the steel tube. When this occurred in Specimen C3, the maximum tensile strains in the flanges of the structural tees are approximately nine times the yield strain. On the contrary, for the compression beam flange, the flange force bears against the column face and the infilled concrete. Under such a condition the connection is locally more stiff in the steel tube, and a more uniform strain develops across the beam compression flanges and the adjacent structural tee.

The above phenomenon indicates that the core of concrete within the panel zone provides some resistance to the panel zone shear, by developing a compression strut as illustrated in Figure 2. Evidence of this is also provided in Figure 21, where the shear-deformation (V - γ) relationship shows the maximum panel zone shear (V_{exp}) exceeding the shear resistance V_s of Specimen C3R per Eqn. (5) that is provided by the steel tube. A summary of the values for the V_{exp} for each specimen, and a comparison with the resistance V_s and predicted total shear capacity V_{total} (i.e., combined steel and concrete strengths) based on Eqn. (4) are given in Table 7. The specimen maximum shear V_{exp} is shown to exceed V_s by 26% (Specimen C1) to 53% (Specimen C3R). Only Specimen C3R was found to approach its panel zone shear capacity, as shown in Figure 21 where the V - γ relationship shows energy dissipation and a stiffness that is almost zero at the maximum shear of +3230 kN. An examination of the ratios for maximum experimental shear-to-predicted shear capacity in Table 7 indicates that the Kanatani model underestimates specimen panel zone shear capacity ($V_{Kanatani}$), for in Specimen C3 the ratio of $V_{exp}/V_{Kanatani}$ exceeds 1.0 and it did not develop a panel zone failure. Furthermore, Specimen C3R was closer to achieving a panel zone shear failure, yet the value for $V_{exp}/V_{Kanatani}$ is 1.36 for this specimen. The panel zone shear capacity V_{ACI} and V_{AIJ} based on the ACI and AIJ provisions, respectively, for the concrete resistance provided results that were almost similar. Compared to the other capacity models, these analytical results agree better with the trend in the experimental results. The new formulation for the contribution of the concrete using the strut capacity per Schlaich et al. (22) shows a similar trend as the Kanatani model in the comparison of V_{exp} with the predicted capacity (V_{strut}) where it over predicts the strength of Specimens C3 and C3R. In both of these formulations the concrete contribution is dependent on the compression strut width S and strength. The effect of the compression forces acting against the tube from the shear tab developing moment is likely to have an influence on the strut's

width within the panel zone. This effect is ignored in the current models. Since the panel zone shear capacity based on these formulations did not agree well with the structural connection subassembly test results, it appears that further studies are needed in order to improve the prediction of the concrete's contribution to the shear capacity. Such studies should consider the effects of the shear tab and beam flange details.

SUMMARY AND CONCLUSIONS

An ongoing research program on the subject of the seismic performance of moment connections for CFT column-wide flange steel beam MRF systems has been presented. This study includes panel zone tests of one half-scale specimens, as well as full-scale structural joint subassembly specimens. The results of the tests on the CFT panel zone specimens indicate that while interior diaphragms may enhance a panel zone's shear capacity by increasing the contribution of the concrete, panel zone specimens without interior diaphragms also performed well under monotonic load and had exceptional ductility. The panel zone shear capacity of the specimens was found to correlate well with joint capacity strength equations that were based on superimposing the shear strength of the steel tube and the concrete core in the joint region, where the latter is based on a strut model.

The test results of the structural connection subassemblies indicates that connections with interior diaphragms or extended structural tees have exceptional cyclic strength, stiffness, and ductility, where the beam is designed to yield in flexure and dissipate energy. The best performance is obtained when the details minimize the possibility of strain concentrations developing in the connection area, whereby beam flange fracture is inhibited. Under such conditions the maximum beam moment at the end of the connection (i.e., cover plates or extended structural tees) can approach 1.25 times its plastic flexural capacity. It was found that the initial elastic lateral stiffness of the CFT column was reasonably well estimated by using the uncracked transformed section. This holds, however, only for cyclic displacements that correspond to an interstory drift less than 0.5% of the story height. More severe deformation caused cracking and debonding of the concrete from inside the tube, leading to a reduction in the elastic stiffness of the CFT column. A comparison of the predicted and experimental shear capacity of the connection panel zone of the subassembly specimens indicates that the analysis methods using a concrete compression strut to evaluate the contribution of the concrete could use improvement. Further analytical and experimental studies are currently underway to determine the influence of the connection details on the strut capacity in order to obtain a better prediction of the panel zone's shear capacity.

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TABLE 1—CFT PANEL ZONE TEST SPECIMEN MATRIX

Specimen	Diaphragm	t (mm)	b/t	d (mm)	α (deg)
1	Yes	5.8	35	203	45
2	No	5.8	35.1	203	45
3	No	4.4	46	203	45
4	No	4.3	45.7	305	56

TABLE 2—CFT PANEL ZONE SPECIMENT MATERIAL PROPERTIES

Specimen	Nominal Dimensions (mm x mm x mm)	σ_y (MPa)	σ_u (MPa)	f'_c (MPa)
1,2	203 x 203 x 6.3	385	520	39
3,4	203 x 203 x 4.5	377	462	39
Diaphragm Plate	190.5x190.5x10 mm with 76 mm Dia. Hole	276	448	-

TABLE 3—CFT PANEL ZONE SPECIMEN CAPACITY

Specimen	V_{exp} (kN)	V_s/V_{exp}	$\frac{V_{exp}-V_s}{V_{exp}}$	V_{exp}/V_{total}			
				Kanatani	ACI	AIJ	Strut Model
1	900	0.68	0.32	0.93	0.98	0.90	1.01
2	702	0.80	0.20	0.92	0.85	0.70	1.02
3	589	0.73	0.27	0.99	0.87	0.75	1.05
4	578	0.74	0.26	1.11	0.89	0.85	1.04

TABLE 4—CFT SUBASSEMBLY TESTS - BEAM AVERAGE MEASURED DIMENSIONS AND MATERIAL PROPERTIES

Specimen	b_f (mm)	d (mm)	t_f (mm)	t_w (mm)	Flange		Web		M_p (kN-m)
					σ_y (MPa)	σ_u (MPa)	σ_y (MPa)	σ_u (MPa)	
C1	179	603	14.9	10.9	293	456	342	475	777
C3	170	603	14.9	10.9	298	475	348	495	805
C3R	179	603	14.9	10.9	298	475	348	495	805

TABLE 5—CFT SUBASSEMBLY TESTS-COLUMN AVERAGE MEASURED DIMENSIONS AND MATERIAL PROPERTIES

Specimen	b (mm)	D (mm)	t (mm)	b/t	σ_y (MPa)	σ_u (MPa)	f'_c (MPa)	M_n (kN-m)
C1	408	408	11.7	34.8	379	448	43.4	1242
C3	409	409	11.8	34.7	379	448	43.4	1245
C3R	409	409	11.8	34.7	379	448	53.4	1365

Note: σ_y and σ_u are mill test reported values.

TABLE 6—CFT SUBASSEMBLY TESTS - MAXIMUM BEAM MOMENT AT END OF CONNECTION $M_{MAX,CONN}$ AT COLUMN FACE $M_{MAX,FACE}$ AND MAXIMUM COLUMN MOMENT AT FACE OF JOINT $M_{MAX,COL}$

Specimen	$M_{max,conn}$ (kN-m)	$(\frac{M_{max}}{M_p})_{conn}$	$M_{max,face}$ (kN-m)	$(\frac{M_{max}}{M_p})_{face}$	$\theta_{p,max}$ (rad)	$M_{max,col}$ (kN-m)	M_{max} $M_{cap, AISC}$	M_{max} $M_{cap, ACI}$
C1	954	1.23	984	0.67	0.027	869	0.94	0.69
C3	906	1.13	992	0.61	0.025	893	0.99	0.72
C3R	992	1.23	1182	0.73	0.065	1058	1.14	0.78

TABLE 7— CFT SUBASSEMBLY TESTS-MAXIMUM PANEL ZONE SHEAR FROM EXPERIMENTS V_{EXP} AND COMPARISON WITH THEORETICAL STRENGTHS

Specimen	V_{exp} (kN)	$\frac{V_{exp}}{V_s}$	$\frac{V_{exp}}{V_{Kana}}$	$\frac{V_{exp}}{V_{ACI}}$	$\frac{V_{exp}}{V_{ALL}}$	$\frac{V_{exp}}{V_{Strut}}$
C1	2660	1.26	0.96	0.72	0.79	0.88
C3	2728	1.29	1.17	0.82	0.81	1.13
C3R	3230	1.53	1.36	0.94	0.94	1.35

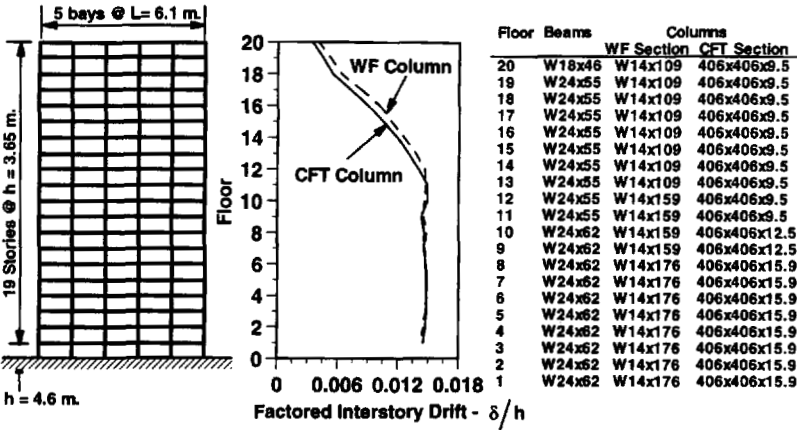


Fig. 1—Prototype Perimeter Moment Resisting Frame for CFT and WF Column Systems.

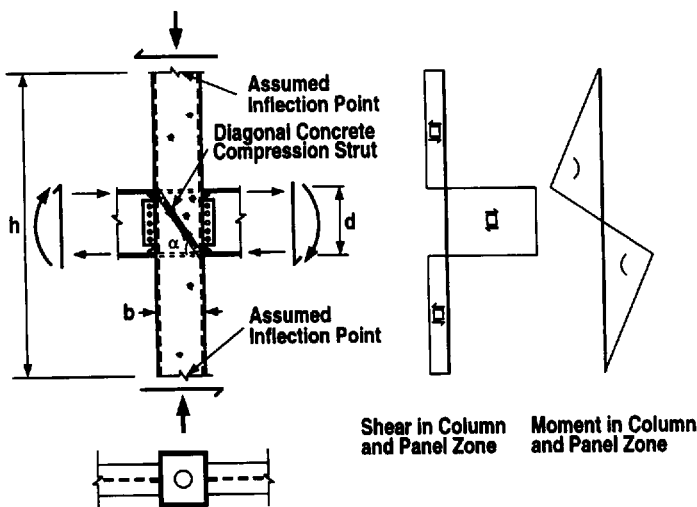


Fig. 2—Shear and Moment in Panel Zone Under Seismic Lateral Loading.

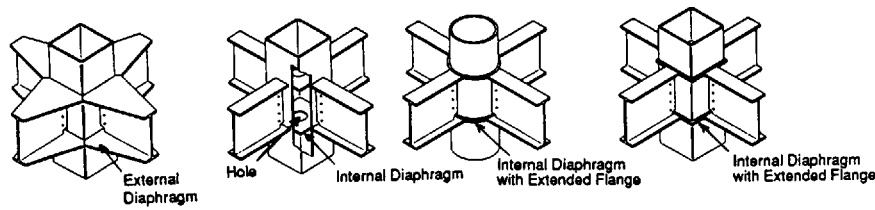


Fig. 3—Typical Japanese CFT Column-to-Beam Moment Connections.

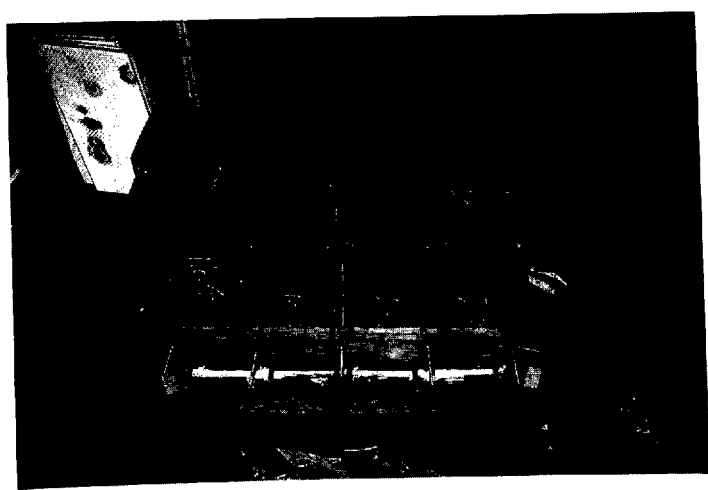


Fig. 4—Panel Zone Specimens Prior to Placing Concrete.

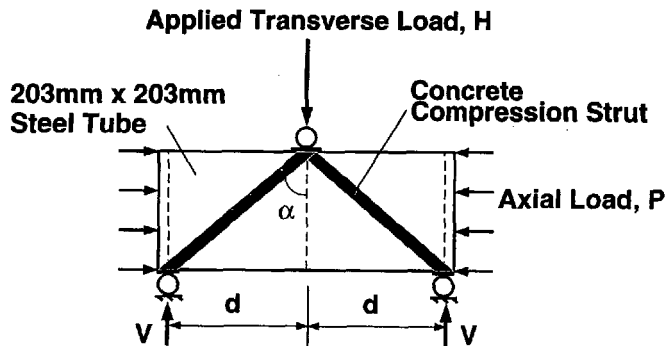


Fig. 5—CFT Panel Zone Test Setup.

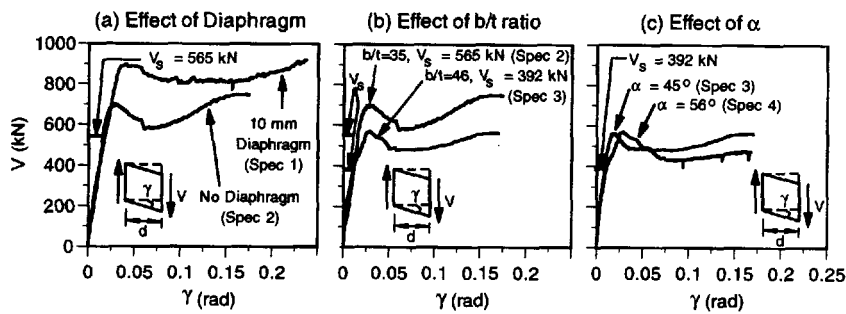


Fig. 6— Shear - Deformation (V - γ) Response of Panel Zone Specimens.

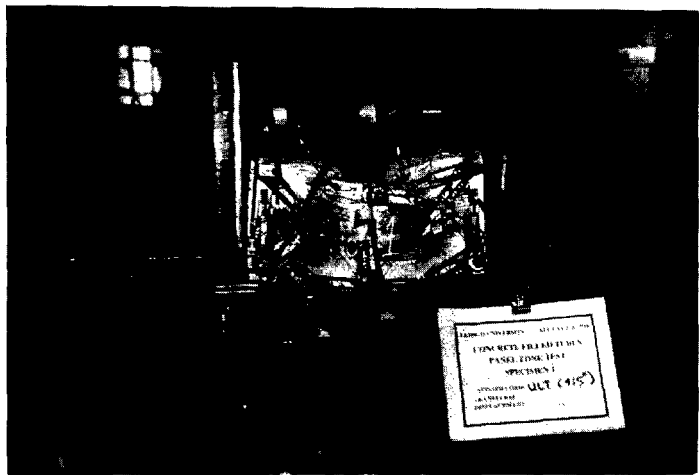


Fig. 7—Specimen 1 After Completion of Test.

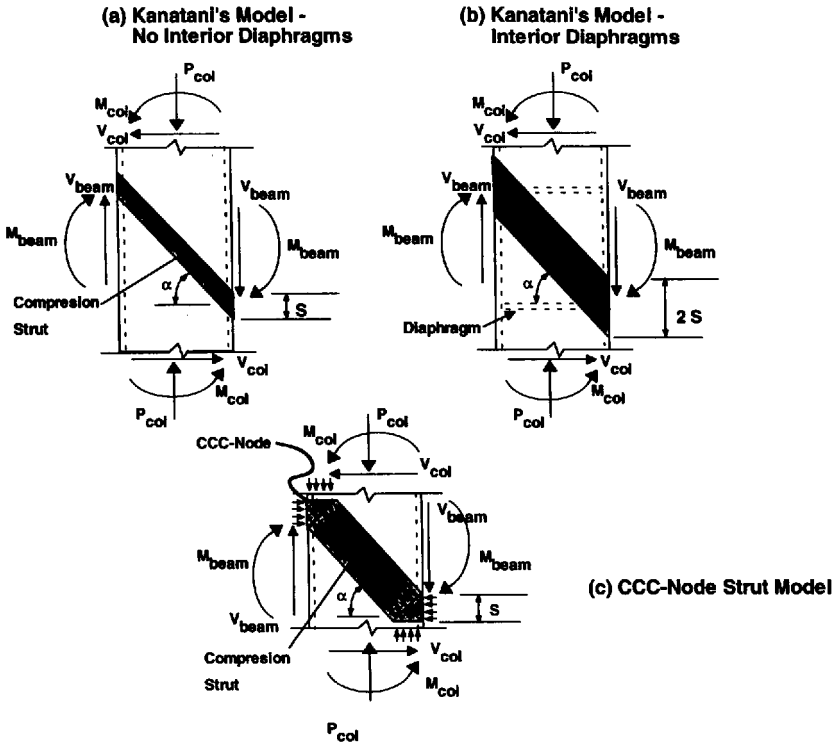


Fig. 8—Models for Determining Concrete Joint Shear Capacity V_c .

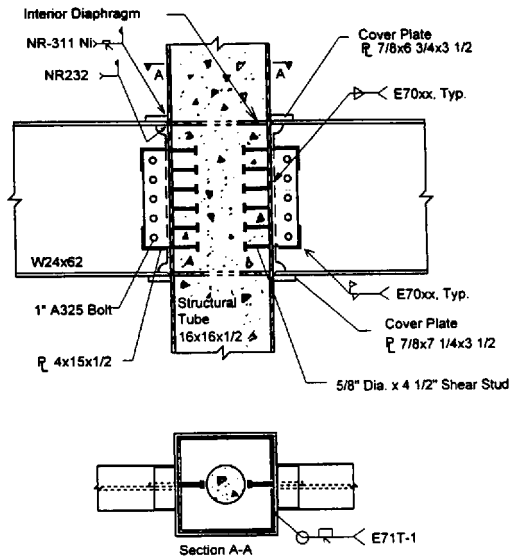


Fig. 9—Connection Details - Specimen C1 (1 inch = 25.4 mm).

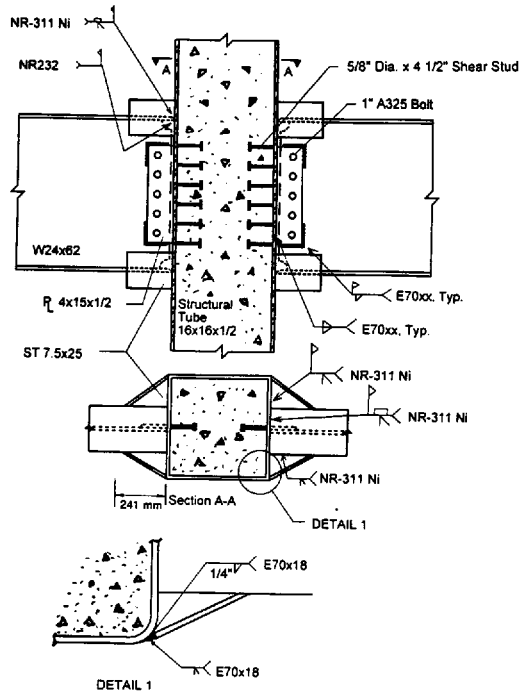


Fig. 10—Connection Details - Specimen C3 (1 inch = 25.4 mm).

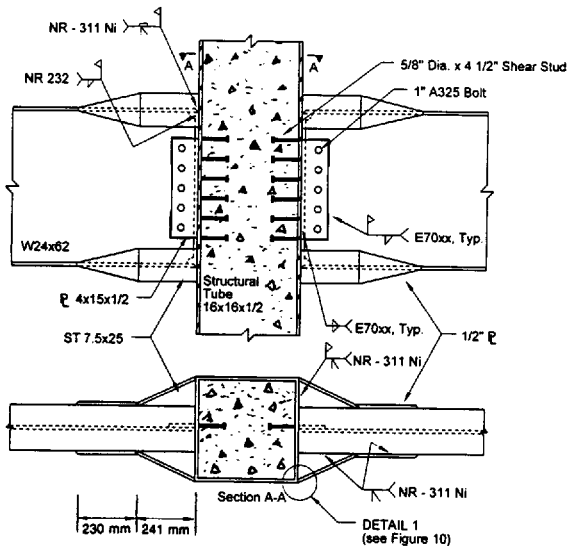
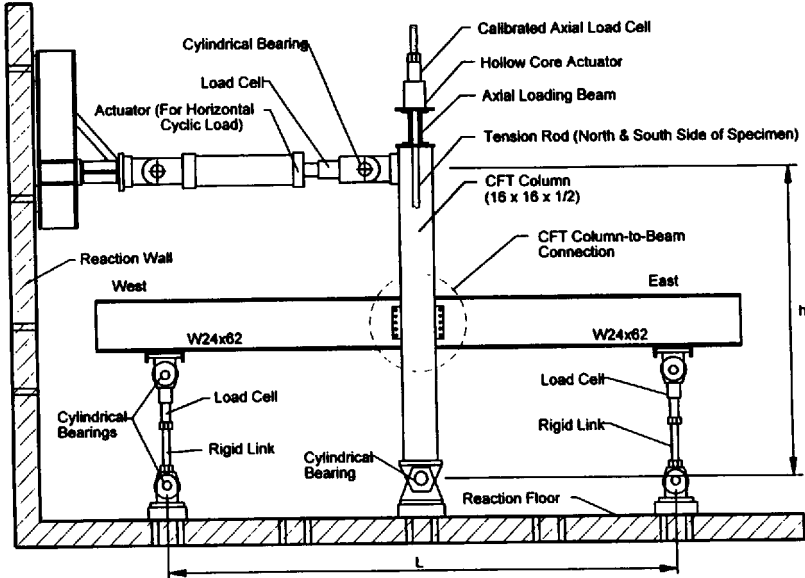


Fig. 11—Connection Details - Specimen C3R (1 inch = 25.4 mm).

(a)



(b)

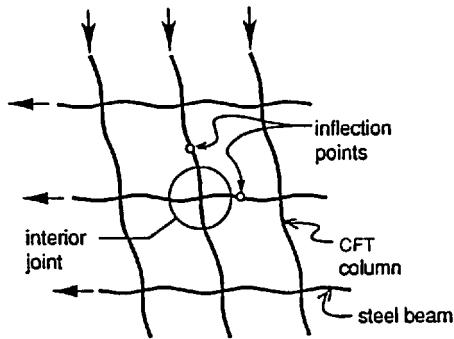


Fig. 12a—Experimental Structural Connection Subassembly Test Setup, and
Fig. 12b —Prototype Composite Interior CFT Joint.

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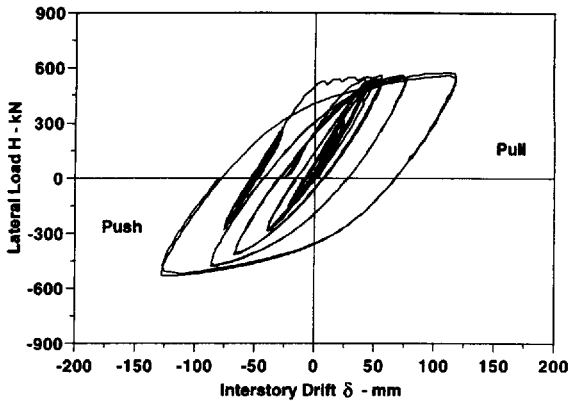


Fig. 13—Lateral Load-Interstory Drift Response, Specimen C1.

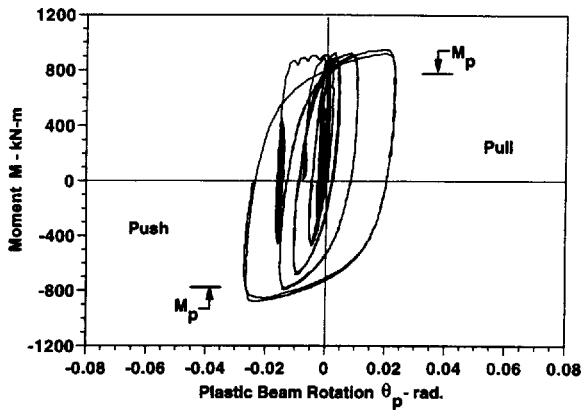


Fig. 14—Beam Moment-Plastic Rotation Response, Specimen C1.

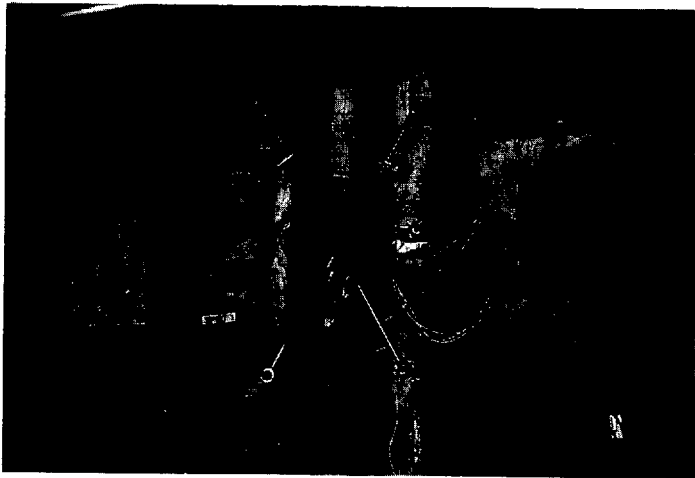


Fig. 15—Connection Region of Specimen C1 at $\mu = 3$.

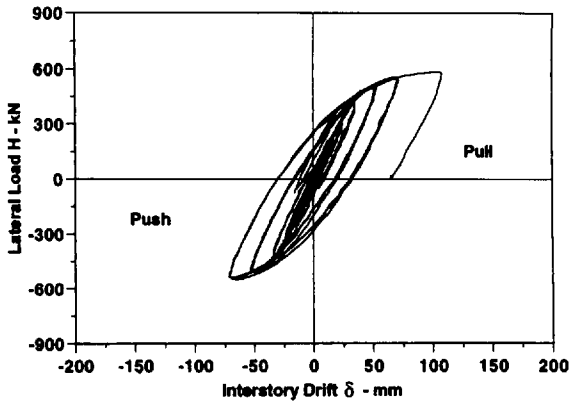


Fig. 16—Lateral Load-Interstory Drift Response, Specimen C3.

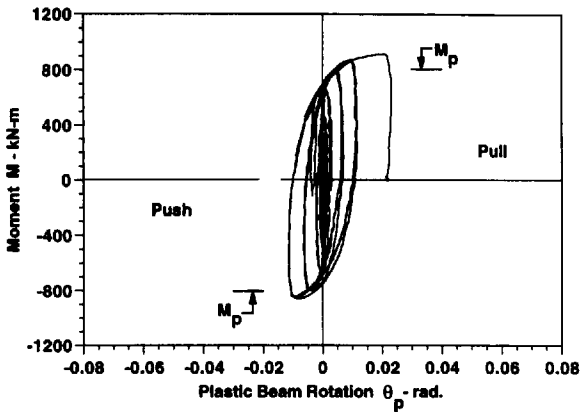


Fig. 17—Beam Moment-Plastic Rotation Response, Specimen C3.

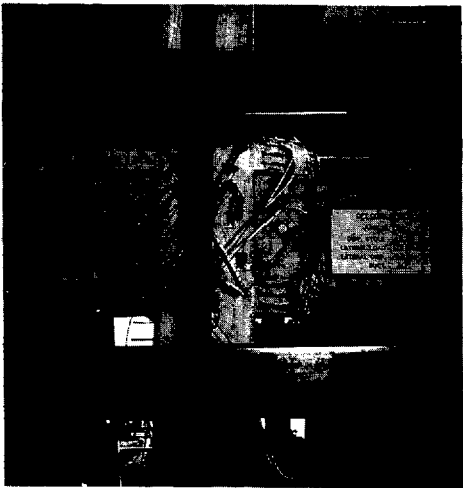


Fig. 18—Connection Region of Specimen C3 at $\mu = 3$.

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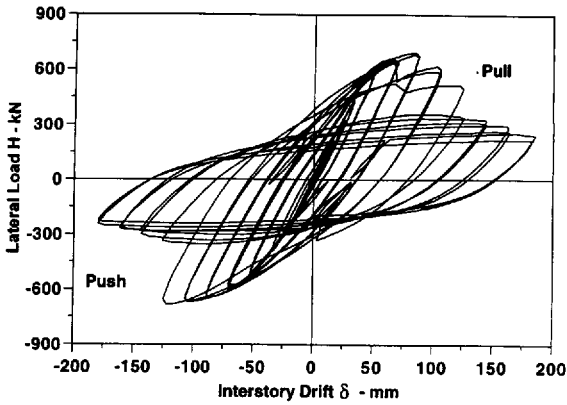


Fig. 19—Lateral Load-Interstory Drift Response, Specimen C3R.

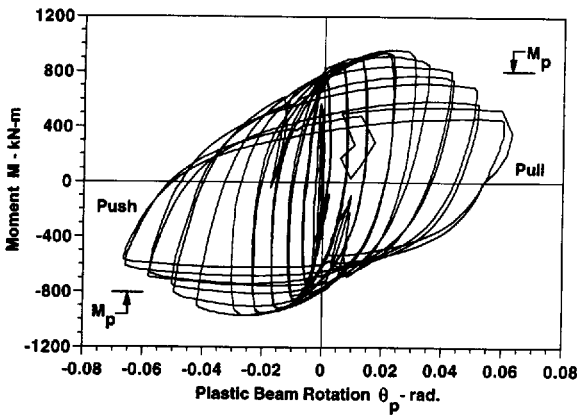


Fig. 20—Beam Moment-Plastic Rotation Response, Specimen C3R.

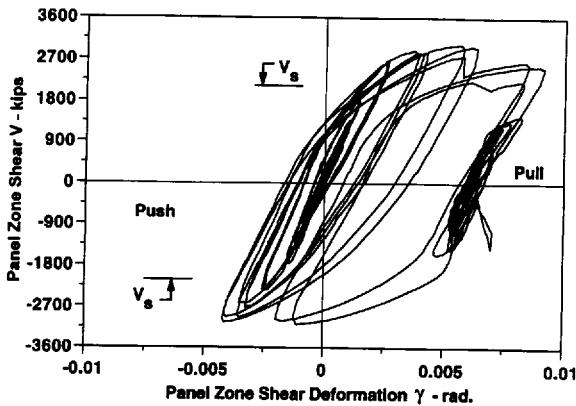


Fig. 21—Panel Zone Shear Force-Deformation Response, Specimen C3R.

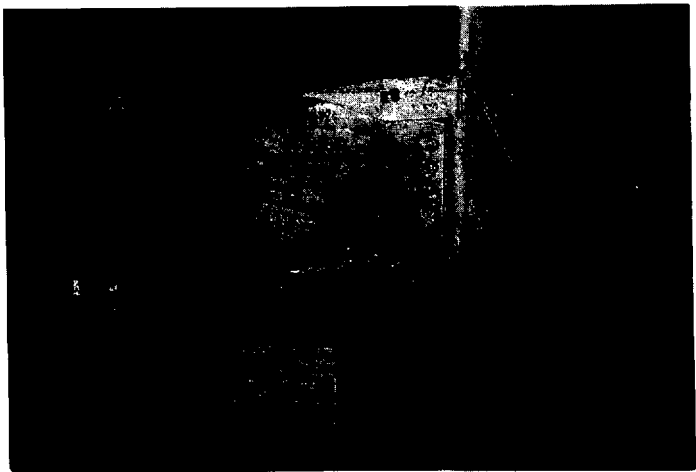


Fig. 22—Connection Region of Specimen C3R at $\mu = 4.5$.

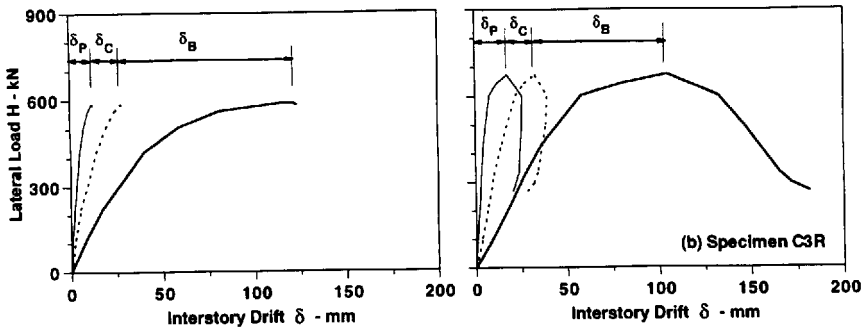


Fig. 23—Contribution of Panel Zone (δ_p), Column (δ_c), and Beam (δ_B) Deformations to Interstory Drift.

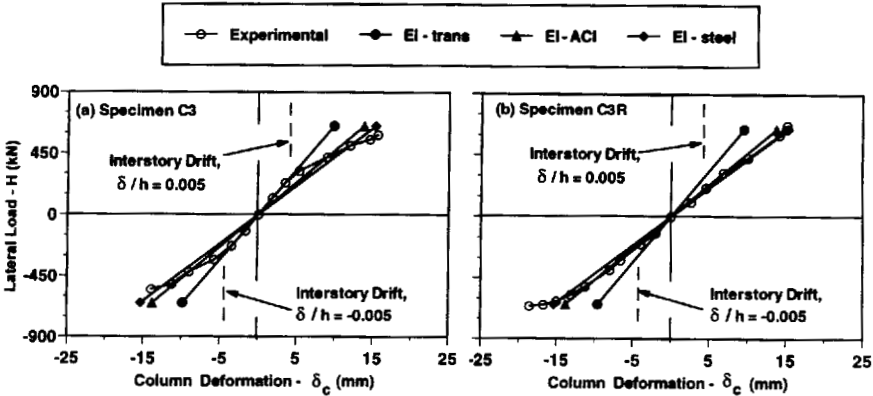


Fig. 24—Lateral Load - Column Deformation Response Envelope.

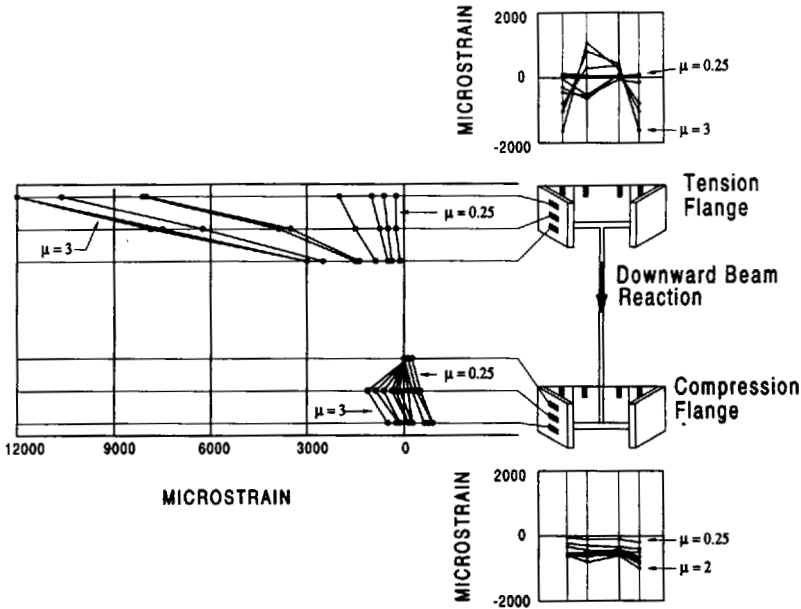


Fig. 25—Longitudinal Strain in Structural Tees and West Beam Flanges at Various Displacement Ductility Levels μ , Specimen C3.

SP 174-2

A Major Research Program on Steel-Concrete-Steel Sandwich Elements

George C. Hoff

Synopsis: This paper describes a large research project which was conducted to develop experimental data on composite structural elements consisting of a steel-concrete-steel (S-C-S) sandwich using headed studs to transfer shear within the composite element. This form of construction may be used as an alternative to either stiffened steel plate construction or reinforced concrete construction. The principal focus of the work was on marine structures such as arctic offshore drilling structures, tidal barrages, floating structures and submerged tube tunnels.

Three distinct categories of structural elements were evaluated: cylinders, flat panels and curved panels/junctions. A total of 59 tests were conducted on large structural elements. The tests included composite cylinders under static axial, static radial and impact loads, flat panels under static in-plane axial, static out-of-plane bending, fatigue out-of-plane bending, and static in-plane shear loads, and curved and junction panels under combined axial and bending loads. A 1:10 scale was chosen for the cylinder tests and a 1:4 scale was used for all other flat and curved panels. To aid researchers and designers doing work on similar types of elements, descriptions of the test specimens, method of specimen preparation and test procedures are given in the paper. The experimental results will be available for release in 1998.

Keywords: Concrete sandwich panels; cost optimization; cylinders; fatigue tests; marine structures; panels; shear; structures; studs

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INTRODUCTION

This paper describes a large research project (1) which was conducted to develop experimental data on composite structural elements consisting of a steel-concrete-steel (S-C-S) sandwich using headed studs to transfer shear within the composite element. The experimental results from that research have a confidentiality period associated with them. To aid researchers and designers who are considering these types of structural elements in their work, this paper describes the test program, details of the test specimens, and methods of testing. The actual test results will be described in subsequent papers beginning in 1998.

S-C-S consists of two steel plates or shells between which concrete is placed with sufficient connection between the steel and concrete to prevent slip so that plane sections remain plane when subjected to bending moments. This form of construction may be used as an alternative to either stiffened steel plate construction or reinforced concrete construction and has many advantages over either of those methods by themselves. These advantages include but are not limited to the following:

- The steel outer skins which enclose the concrete help increase the strength and ductility of the concrete. The concrete resists compression loads and the steel resists tension loads.
- The concrete stabilizes the steel skins thus increasing their buckling capacity. This reduces or eliminates the need for stiffeners as the concrete performs this function.
- The panel boundary condition restraints reduce or eliminate the need for shear connection or shear reinforcement. The steel outer skins reduce or eliminate the need for vertical or horizontal reinforcement and thus eliminate the materials, fabrication and installation costs of reinforcing bars.
- The steel outer skins are permanent formwork thus eliminating the need for conventional formwork for the concrete.

- The strength of S-C-S elements is not sensitive to construction imperfections. The concrete can readily absorb the fabrication tolerances of the steel skins, and the steel bridges over small discontinuities in the concrete.
- There is more uniform panel strength for localized punching shear/impact loads when compared to stiffened steel plating.
- The steel outer skins provide better abrasion resistance than a concrete surface and are also impervious to the ingress of the environment.
- The S-C-S has better fire resistant properties than steel construction.
- It is easier to make attachments to the steel outer skins than to a concrete surface.

The project to study S-C-S elements was conducted between 1989 and 1994. At the time the study began, there already existed a significant body of test information on S-C-S, but this information had not been studied as a single body of data to determine its usefulness in developing a design methodology. A review of this data indicated much duplication of testing and a lack of information on some typical types of S-C-S elements, types of loading and failure modes for these elements. The test program described herein was formulated to fill in the gaps such that the design of complete structures might be possible at the completion of the test program. The physical test program was complemented by analytical studies to develop a design guidance document (1). That guidance was used to develop several theoretical structures to which cost optimization was applied (1).

Many types of shear connectors have been considered for S-C-S sandwich panels. These include bulkheads, angle bars, studs or combinations of these connectors. In this test program, all specimens were designed with headed studs. From the review of previous work, it was found that the most cost efficient form of shear connector (in terms of shear capacity per unit cost) was the headed stud, hence the selection of headed studs was based solely on a perceived potential savings during fabrication of the sandwich panel. Studs can be rapidly welded on to individual steel plates either to singular plates or two plates simultaneously in a quick and simple operation. Other types of shear connectors require substantial lengths of welds and may have limited access to perform these welds. With current manufacturing and installation technology, the cost of drawn arc studs increases rapidly if the length exceeds 150mm and the diameter exceeds 22mm. However, by welding studs to the heads of those already installed, longer studs can be built up economically.

TEST PROGRAM

A comprehensive review of the large data base of existing laboratory tests on S-C-S

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indicated that there were three distinct categories of structural elements to be considered: cylinders, flat panels and curved panels/junctions. Table 1 shows these categories and the types of loading that was used in the test program for each category of structural element. A more detailed explanation for each type of test is contained in the following sections. A total of 59 tests were conducted.

TESTS OF CYLINDERS

As shown in Table 1, there has been a large number of tests performed previously of S-C-S cylinders (2,3,4,5,6). This test program tested an additional 8 cylindrical specimens; 6 under static axial conditions, 1 under static radial conditions and 1 under a quasi-impact load.

All of the previous 28 tests of cylinders under axial static loads were conducted at Manchester University, UK (4). Test variables included length of the cylinder, varying thickness of the outer skin and concentric and eccentric end compressive loads. Six tests to complement this previous work and fill in some of the data gaps were performed in this program and are described as Series A Tests in Table 2. Variables included the effects of concrete strength, thinner walls with constant proportions, effects of thinner steel skins and varying skins, and eccentric loading. All Series A specimens had an outer diameter of 1800mm, a height of 2700mm, and a wall thickness (including the steel skins) of 60mm with the exception of Specimen A3 where the wall thickness was reduced to 40mm. Both the target and actual concrete cube (100x100x100mm) strengths are shown in Tables 2 through 8.

Of the 109 tests under static radial loading (Table 1), 106 were also conducted at Manchester University (5) with the remaining 3 being done in Japan (6). The Manchester work was part of a program on deep sea habitats. These tests included variations in geometry, relative proportions of steel and filler, types of filler (e.g., aggregates, concrete, etc.), steel to concrete bond (studs, bolts, adhesives), non-circularity, local damage effects, sustained pressure, hemispherical end closure, and wall penetrations. To complement these results, only one test was performed in this program in an effort to tie the previous results to the current program (C1 in Table 2). The specimen had an outer diameter of 2920mm and was 1200mm tall. The overall thickness was 60mm which included both the 3mm inner and outer skins.

The remaining cylindrical specimen test is described as the D-series test in Table 2. It was identical to Test Specimen C1 but was subjected to a quasi-impact load (two-point radial load) at mid-length. No previous data could be located for this type of loading on an S-C-S cylinder.

Cylinder Specimen Selection

The cylindrical test specimens (Fig. 1) are based on sections of the legs of large

floating facilities such as semi-submersibles and of turbine draft tubes of tidal barrages. The diameters of these elements in actual structures tend to range between 15 and 20m. Based on typical loads carried by the legs of a floating facility, a double-skin sandwich wall thickness would be around 500mm. For the size of testing equipment available, a 1:10 scale was chosen. This was a realistic compromise between technical and economic conditions.

It has been shown that shear studs offer no benefit to cylinders under radial load (2) or to axially loaded cylinders (5). Although some correlation has been found between stud spacing and the steel plate thickness with respect to the onset of buckling of the steel skins in cylinders subjected to in-plane loading, the spacing was sufficiently large so it was effectively a no-stud condition (6). Hence, no studs were included in the cylindrical specimens of this program.

The axial test rig for the work could accept cylindrical specimens up to 1.8 m diameter and the specimens were designed to that size. The test rig was capable of applying a total load of 30600 kN. Similarly, the available radial test rig could accommodate specimens up to 3m diameter and those specimens were designed accordingly. That test rig could provide a maximum load of 88000 kN which corresponded to a pressure of 9.6 MPa over 1m of wall height. The two point radial load for specimen D1 was to represent the point loads which might be associated with water-borne ice impacts.

Preparation and Testing of the Axially Loaded Specimens (A1 to A6)

The form of construction consisted of two unstiffened cylindrical tubulars with a concrete infill (Fig. 1). The steel skins did not utilize shear connections of any type. The thickness of the concrete infill was chosen at 54mm for all specimens except A3 which had an infill thickness of 36mm. On an assumed scale of 1:10, a section composed of two 3mm skins with a 54mm infill would correspond to a prototype section of 30mm skins with a 540mm infill and 18m in diameter.

The original test program had two target concrete strengths; 20 and 40 MPa. Specimen A2, which used 20 MPa concrete, was fabricated and tested first because its lower design strength would minimize any unnecessary difficulties in achieving failure. That specimen failed at 89% of the test rig capacity when the concrete cube strength at the time of test was only 25.3 MPa thus indicating that a concrete infill of 40 MPa would exceed the capacity of the test rig. The concrete strength in all remaining specimens was then reduced to a target value of 15 MPa.

Each skin of the specimens was instrumented (Fig. 2) before being placed in a circular formwork called a loading shoe. The shoe was then filled with grout which was to provide restraint to the free ends of the cylinder during testing. Initially, epoxy grout was used but this was later changed to a high alumina cement grout due to operational difficulties. Once the grout had hardened, the skins and the lower shoe were placed

in the test rig. The selected concrete mixture was then placed in the annulus of the two skins, lightly vibrated by internal vibrators, and finished with a trowel at the top surface.

The inner steel skin had to be heavily supported during the concrete placement to avoid a compressive failure (buckling) and minimize stresses caused by the hydrostatic pressure of the unhardened concrete. These stresses would be locked in once the concrete had hardened. Instrumentation on the initially cast specimens indicated that the internal supports provided had minimized these stresses.

Once the infill concrete had obtained sufficient strength, the top loading shoe was placed on spacer blocks on the top concrete surface. The space between the shoe and each skin was sealed and a grout poured into the remaining void. Once the grout had reached an acceptable strength, the top section of the testing rig was placed. The two sections of the testing rig were then joined together with 40mm stressing rods (Fig. 3). An array of 36 jacks were evenly spaced around the top loading shoe. Each jack provided the tension to the stressing rod which was dead anchored in the lower steel loading rig (Fig. 3). The 36 jacks were manifolded to a single hydraulic source. An initial bedding load was applied to each specimen and then released so zero instrumentation readings could be taken.

Load was applied in increments not exceeding 3000 kN and was maintained for two minutes at each increment. These increments were reduced when non-linear behavior was observed or any cracking sound or other unusual behavior was noted. Deflection plots were produced throughout the test. Instrument readings were taken to correspond to each load increment. After test completion, a visual inspection was performed to note any unusual features of the skins and/or the concrete. The outer skin was cut vertically and carefully removed from each specimen. Ultrasonic readings of the concrete were taken within the axial specimens to see if any zones of weak material had influenced the failure.

Preparation and Testing of the Radially Loaded Specimens (C1 and D1)

The two skins of the radial specimen were placed into the loading rig onto a prepared surface of two polyethylene sheets with a layer of grease between them. The polyethylene sheets were on a level plywood surface. This arrangement was designed to minimize any end restraint by the floor during the radial loads. The annulus was then concreted and, after hardening, the steel skin were instrumented. For the C1 specimen, a total of 44-200 tonne jacks were then uniformly spaced around the periphery of the specimen (Figs. 4 and 5). For the D1 specimen, two pair of jacks, diametrically opposed, were used (Fig. 4). All other jacks were either removed or disengaged from the hydraulic system.

A similar test procedure to that of the axial specimens was used for both radial specimens. The skins of the radial specimens were also cut open to inspect the

concrete inside.

TESTS OF FLAT PANELS

A typical S-C-S flat panel is shown in Fig. 6. As shown in Table 1, a large number of tests (224) on out-of-plane bending have been performed on flat panels. Of those tests, detailed information on the specimens and test procedures was only available in References 7 to 16. Only 34 of these tests (9.a, 10,14) involved the use of studs, however. No previous data was found for in-plane tests of flat panels. To supplement this information, this test program tested an additional 9 flat panels (Series G) under static out-of-plane bending (flexural) plus 21 flat panels (Series E, F and M) under static in-plane axial loading, 6 flat panels (Series H and P) under fatigue out-of-plane bending (flexural) and 2 flat panels (Series N) under static in-plane shear (racking loads). All 38 flat panels used studs for shear transfer (Tables 3 and 4).

Flat Panels Under In-Plane (Axial) Load

Because vertical walls will exist in almost all practical applications of S-C-S panels, it is necessary to consider flat panels under in-plane (axial) load. A scale of 1:4 was selected for the test specimens in this program. A prototype S-C-S wall may typically be 500-700mm thick which then leads to a flat panel specimen thickness of 150mm (average). For the cylinder tests noted above, the wall thickness to total steel skin thickness ratio was 10 to 15. For the flat panels, this would give steel skin thicknesses of 5mm to 8mm. It was decided to use skin thicknesses of 4mm and 8mm as these were at the extremes of thickness based on scale. Evaluation of the thinner 4mm skins would give an indication of whether thinner skins were detrimental to the panel performance. Other variables evaluated were the use of lightweight concrete as the infill material and the effect of confinement on the free edges of the test specimen. As the test specimens represent only a slice out of a continuous wall, the edge confinement simulates the effects of having a continuous wall rather than a free edge section. Fig. 7 shows the details of a typical flat panel specimen along with instrumentation locations on the panels. These tests were conducted in four separate test series: Series E, F, M and N.

E-Test Series: The E-Test Series (Table 3) examined the ultimate static strength behavior of flat S-C-S panels under concentric and eccentric axial load, with headed stud connectors providing shear connection between the steel skins and normal density infill concrete. Variables included overlapping and non-overlapping studs, stud density, skin thickness and varying eccentricities of load. The objectives of that test series were to:

1. Establish the number of headed stud shear connectors required to maintain the composite nature of the specimens up to flexural failure.

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2. Establish the stud density required to prevent buckling of the skins prior to failure.
3. Investigate the effect of eccentric loads on specimen behavior.
4. Study the effect of overlapping and non-overlapping studs.
5. Relate the behavior of S-C-S elements under axial loading to the flexural behavior of reinforced concrete beams and the buckling behavior of steel member design.

Specimens E1 to E10 (Table 3) were 2100mm in length and 600mm in width. The concrete infill was 150mm thick. The thickness of the steel skins was varied at 4mm and 8mm. The headed studs were either 150mm long with a 10mm diameter or 65mm long with a 6mm diameter. The 150mm studs were always fastened to the skin that would take the least compressive load or the largest tensile load. In this manner, these tests are linked to the G-Series tests (Table 5) which did the same thing. The ends of these specimens were bolted to heavy end plates prior to casting to ensure an even distribution of load between the skins and the concrete.

Specimens E11 to E13 had a reduced width of 200mm. The ends of those specimens included a 300x170x120mm rectangular reinforced cage made of 6mm diameter high yield reinforcing bars to reduce localized crushing and spalling of the concrete.

Specimens E1 to E9 had a constant stud density with the studs attached at 100mm centers. The purpose of these tests was to first compare the effect of overlapping and non-overlapping studs under increasing eccentricity of load, and secondly to evaluate the effect of plate thickness on both the ultimate capacity and failure mode. All these tests were done at City University in London.

Specimen E10 was also tested at City University and was identical to Specimens E1, E4 and E7 with overlapping studs and 8mm skins. However, this specimen was loaded at a small eccentricity of $e/h=0.20$ in order to link these previous tests to the more slender panels, E11, E12, and E13, which were tested at Cardiff University and with some previous work at Cardiff on specimens with an e/h in the range of 0 to 0.35.

Specimen E11 had 8mm skins on both faces, overlapping studs at 100mm centers, and an eccentric load of $e/h = 0.20$, thus linking it to Specimen E10. The effect of the reduced width of 200mm for E11 then could be evaluated against the 600mm width of E10.

Specimens E12 and E13 had reduced numbers of studs on the steel skins and different skin thicknesses as shown in Table 3. The skin with more compressive load was 4mm thick while the skin with least compressive load or tensile load was 8mm thick. The objective of these variations was to develop a better understanding of the plate buckling mechanism in these S-C-S elements.