Behavior of concentrically loaded CFT braces connections

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ABSTRACT

Concrete filled tubes (CFTs) composite columns have many economical and esthetic advantages, but the behavior of their connections is complicated. Through this study, it is aimed to investigate the performance and behavior of different connection configurations between concrete filled steel tube columns and bracing diagonals through an experimental program. The study included 12 connection subassemblies consisting of a fixed length steel tube and gusset plate connected to the tube end with different details tested under half cyclic loading. A notable effect was observed on the behavior of the connections due to its detailing changes with respect to capacity, failure mode, ductility, and stress distribution.

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Introduction

Concrete filled steel tube (CFT) columns are widely used in composite steel–concrete braced steel frames due to the gained advantages from combining concrete and steel. They increase the frames stiffness compared to open steel sections and decrease construction costs as they preclude the use of shuttering. The use of braced frames becomes indispensable, especially in multi-story buildings to reach the drift limits recommended by the design codes. However, connections to CFT columns are complex, concerning both design and construction. Although there is a wide range of connections that can be used with composite construction, designers usually face difficulties and insufficient design data when dealing with brace-to-column connections of composite braced frames with (CFT) columns. This is clearly pronounced in the Egyptian Code of Practice for Steel Construction and Bridges ECP 205-2001 [1], which provides only one provision for the design of composite column connections.

During earthquake excitation, the brace forces' direction is reversed. Hence, the brace-to-column connection should ensure that the load is transferred to both steel and concrete in a manner that ensures satisfactory seismic performance of the system. The connection detailing is crucial to guarantee...
minimum ductility requirements. Brace-to-CFT column connections are usually detailed using a gusset plate welded directly to the steel tube shell or penetrating through the CFT column. Tests performed show that steel-to-steel connections possess higher bond stress demands than connections penetrating through the CFT column. Meanwhile, the main load transfer mechanism in connections with gusset plates penetrating through the column is direct bearing. This leads to the concentration of bearing stresses beneath the gusset plate at intersection locations [2,3].

Many design codes consider direct bearing, shear connection, or direct bond interaction between steel and concrete as the main load transfer mechanisms at the connection zone. However, most codes do not allow combining these mechanisms due to lack of data on the behavior of the connection [4,5].

In addition to the configuration of the connection, many factors are considered crucial to the behavior of brace-to-column connections such as thickness of tube and gusset plate, coefficient of friction between steel and concrete, and the level of axial force on column. MacRae et al. [3] performed experimental and numerical study on different configurations of brace-to-column connections in order to evaluate the effect of such factors on the ability of braces to transfer force into the concrete and steel of CFT columns. Cyclic compressive load was applied to the top of the gusset plate parallel to the column axis. However, the direction of brace loading was not considered.

Analytical studies [3,6] were conducted to analyze the behavior of gusset plate CFT-to-bracing connections while considering a wide range of parameters such as the load ratio on the CFT column, thickness of gusset plate, and introduction of cutouts. Generally, failure of the connection was observed under the connection area. It was also found that increasing the thickness of gusset plate or introducing cutouts has small effect on the ultimate strength of CFT column; yet, they would cause more local bulged shapes on the steel tube below the connection area.

Many researchers [7–10] have investigated the behavior of gusset plate connections in braced frames. It was found that the capacity of gusset plate connections is affected by thickness of gusset plate, brace inclination angle, and existence of stiffeners. Many studies focused on addressing the yield mechanisms and failure mode behaviors of such connections under seismic loading effects. Moreover, tests on full-scale composite braced frames with CFT columns under pseudo-dynamic loading were conducted in order to give insight into the global behavior of the system and the effect of connections as reported by Tsai and Hsiao [11]. However, experimental studies that address the transfer of force between the bracing member and the CFT column are rare and are usually conducted on subassemblies with the brace force applied in a manner that does not reflect the real situation.

The main goal of this paper is to present the results obtained from an experimental program carried out to investigate the behavior and strength of different configurations of the brace-to-column connection under the action of compressive cyclic loading between zero and a value increasing till the capacity of the connection. The data extracted from the tests will be used to perform a parametric study addressing the effect of various parameters. It is also aimed to study the effect of connection detailing on its behavior, load capacity, and failure mode which are considered as the key aspects for the development of design provisions for such connections.

**Experimental**

A series of 12 tests designed to investigate the behavior, load capacity, and failure mode of brace-to-CFT connections were carried out to failure in the laboratory of American University in Cairo. The studied parameters were the connection configuration, the direction of load application as well as the thickness of gusset plate. All other parameters that may affect the connection behavior such as pipe size, pipe thickness, and plate edge distance were fixed. Fig. 1a shows a general layout of the different tested connections, which are summarized and listed in Table 1. The tests were divided to three groups A, B, and C depending on the loading and pipe size used. The connection details were divided into four types' No. 1, 2, 3, and 4. Type 1 refers to connections with gusset plate welded directly to the steel tube shell with provision of shear connectors welded within the connections zone and embedded inside...
concrete, type 2 to connections with gusset plate fitting within slots in the steel tube, type 3 to connections similar to type 2 with an additional gusset plate welded perpendicular to the penetrating gusset plate, and type 4 to connections with gusset plate welded directly to the steel tube shell which are considered as pilot connections.

The cost of connections is an important factor that determines the possibility of their application. Determining the exact cost of the studied connections is not a straightforward task as it depends on the size of the connection, diameter of the column, the external applied loads, and the needed workability. In addition, the cost of the connection varies from one country to another and within the same country. However, for comparative purposes, the cost of the studied connections is estimated based upon the relative weight of the used materials and the relative fabrication time consumed in each connection. Generally, the overall cost of the studied connection types can be sorted in a descending order as follows: 3, 1, 2, and 4. This order accounts for the required welding and slotting practices within the connection zone. Type 3 requires slotting on the four sides on the connection; in addition, welding is required outside and inside the steel tube. Type 1 requires a relatively large diameter of the steel tube to facilitate welding the shear studs inside the steel tube. Type 2 requires slotting and welding at two opposite sides of the connection. Type 4 requires the least workability as the gusset plate is welded to the steel tube directly.

The connections with different shapes and configuration are attached to the specimens' ends to avoid the effect of its deformation at mid-span on connection behavior. Table 1 also shows the pipe diameter, $D$, shell thickness, $t$, and the connection gusset plate thickness, $t_g$, which vary according to the connection details and load application.

Part of test specimens after concrete pouring is shown in Fig. 1b. The specimens consisted of a small part of circular pipe fabricated from 114 or 168 mm diameter, which represent a part of CFT column, connected to stiffened steel seats by bolting. The length of specimens was fixed to be 680 mm. The distance between the gusset plate edge and all specimens’ ends was 90 mm. This distance was chosen as the least probable distance in order to decrease the moment applied on the steel tube to the least value. Maximum allowable fillet weld sizes were used to join the gusset plate to the steel tube that performed using E70 electrodes. Typically, the strength capacity of the designed connection is higher than the connected members in order to ensure a ductile behavior at failure which is not exactly respected in the tested specimens. To study the adherence and compatibility of connection design to seismic specifications, specimens SP-H1 and SP-H2 only were designed such that the CFT was stronger than the connection, but the rest of the specimens’ connections were stronger than its CFT column. Type 3 connections were designed such that the bearing area on the concrete was equivalent to type 2 connections. This resulted in using a gusset plate with half of the thickness used in type 2 connections.

**Test configuration**

The basic test setup is composed of the tested subassemblies, supporting system, and loading system as shown in Fig. 1d. The supporting system was designed to be compatible with the lab facilities available (strong wall, strong floor, and rigid frame). The hydraulic actuator with capacity 1400 kN and ±400 mm stroke was supported by the strong wall from one side and attached to the specimen through a thick plate and two angles connected to the end of the connection gusset plate. The specimens were attached to the strong floor through two end seats composed of thick plates. Such seats were connected to specimens’ end plates by friction bolts and connected to the

| Test group | Detail number | Test unit | $D$ (mm) | $t$ (mm) | $t_g$ (mm) | concrete infill | $f'_c$ (MPa) | Loading direction |
|------------|---------------|-----------|----------|-----------|-------------|----------------|---------------|------------------|
| A          | 1             | SP-H1     | 168.3    | 5         | 12          | Filled         | 37.8          | Perpendicular [horizontal] |
|            | 2             | SP-H2     | 168.3    | 12        |             |                | 40.6          |                  |
| B          | 1             | H-1       | 114.3    | 3         | 12          | Filled         | 17.5          | Perpendicular [horizontal] |
|            | 2             | H-2       | 114.3    | 3         | 12          |                | 17.5          |                  |
|            | 3             | H-3       | 114.3    | 6         | 6           |                | 17.7          |                  |
|            | 4             | H-4       | 12        | 12        |             |                | 17.7          |                  |
|            | 4             | H-5       | 12        | 12        | None        |                | 17.5          |                  |
| C          | 1             | V-1       | 114.3    | 3         | 12          | Filled         | 18.1          | Parallel [vertical] |
|            | 2             | V-2       | 114.3    | 3         | 12          |                | 18.1          |                  |
|            | 3             | V-3       | 114.3    | 6         | 6           |                | 18.1          |                  |
|            | 4             | V-4       | 12        | 12        |             |                | 18.1          |                  |
|            | 4             | V-5       | 12        | 12        | None        |                | 18.1          |                  |
floor by 80 mm diameter anchor bolts. The load is applied on the specimens in two different directions, parallel and perpendicular to the specimen that simulates the vertical and horizontal components of the brace forces. It can be seen that specimens were supported at one central point at each end which resulted in rotating of the whole specimen around these points. Therefore, the additional rotation and displacement are subtracted from the results. Great care was given to leveling of the specimens to ensure that it was correctly aligned during testing. Fig. 1e shows the general setup for test unit (SP-H1) of group A.

**Material properties**

The tested specimens included steel columns pipes, steel plates, and concrete infill. Four test coupons were cut out from the steel pipes ends composing the columns after the completion of the experiments to evaluate the mechanical properties
through tensile tests. The principle material properties measured in these tests are reported in Table 2. Connecting angles were made of steel 37, which have yield strength and ultimate strength equal to 240 and 360 MPa, respectively. The used shear studs were of grade (8.8) which stands for yield strength and ultimate strength equal to 640 and 800 MPa, respectively. Besides, quality control during the mixing of concrete was made by the determination of mechanical properties of concrete by testing of 150×150×150 mm standard cubes and 150 mm diameter and 300 mm height standard cylinder. The control specimens are casted in the same time of pouring concrete in columns. The mean compressive strength of concrete cylinders ($f'_c$) after 28 days is equal to 38.0 MPa for test group A and equal to 16.7 MPa for test groups B and C. Table 1 shows the concrete compressive strength that was estimated at the testing day.

**Table 2** Mechanical properties of steel shapes.

| Specimen no. | Yield stress (MPa) | Ultimate stress (MPa) | Elongation (%) |
|--------------|--------------------|-----------------------|----------------|
| Specimen-1   | 278.8              | 347.4                 | 18.8           |
| Specimen-2   | 293.8              | 356.6                 | 30             |
| Specimen-3   | 267.1              | 347.4                 | 33.75          |
| Specimen-4   | 300.6              | 362.7                 | 25             |

**Instrumentation**

The instrumentation of the specimens was designed to determine the applied loads, measure deformations along the specimen, and quantify the internal stresses of the specimens. Electrical strain gages, with 6 mm gauge length with 120 Ohms resistance, were glued to the steel tube at the midpoint of the gusset plate to measure strains at the connection zone during different stages of loading. Three strain gages were employed in test group A, while two strain gauges were used for the rest of the specimens (Fig. 1c). Linear variable displacement transducers (LVDTs) with strokes equal to 100 and 200 mm were used to measure displacements at different locations of specimens as illustrated in Fig. 1d. The LVDTs were mounted at the required location by means of special posts attached to a rigid beam. Small holes were made along the specimens for the LVDT needle to rest in. This was meant to minimize the movement of the LVDT from the desired point during testing. The value of the applied load was measured directly by the load cell attached to the actuator.

**Loading procedure**

At the beginning of the test, a small force was applied and increased gradually; meanwhile, the data acquisition system was observed to ensure that the readings are reasonable. The applied compression load was 0.5 kN/s which was chosen as the slowest possible loading rate in order to preclude any impact or dynamic effects and guarantee better observation of the testing procedure while preserving a reasonable testing period. Initially, the forces were increased by 2.5 kN per each successive cycle then drops to zero till reaching 25 kN. Then, the load was increased accordingly with 5 kN per each cycle until the specimens showed large deformations or failure of one of the structural elements. Fig. 1f shows the general loading pattern of different tests.

**Results and discussion**

The experimental results that show the specimens’ behavior, load–displacement curves, failure patterns, and strain measurements of the different specimens are outlined and discussed in the following sections.

**Fig. 1e** General view of test setup of test unit SP-H1.

**Fig. 1f** Cyclic loading pattern.

**Fig. 2a** Fracture of weld in test SP-H2.
Specimens SP-H1 and SP-H2 were not loaded till failure due to their higher strength and the lower capacity of the setup. However, permanent deformations were observed in the gusset plate of test unit SP-H1. Moreover, cracks at the bottom of the fillet weld were observed in test unit SP-H2 as shown in Fig. 2a. Nevertheless, the steel tube itself preserved its integrity. The rest of specimens were loaded till failure. The observed failure mode for specimens H-1 and H-4 was bulging of the pipe shell underneath the gusset plate as shown in Fig. 2b. Specimens H-2 and H-3 failed due to tearing of the steel pipe as shown in Fig. 2c. The location of tearing was at the end of the gusset plate and at 5 cm away from the end of the gusset plate for the two specimens, respectively. This may be attributed to the fact that for connections with gusset plate penetrating through the steel tube, increased portion of the load is imposed on the steel shape, and the distribution of the load on the whole steel section is achieved. As shown in Fig. 2d, for tests V-1 and V-4, the observed failure mode was bulging of the pipe shell underneath the gusset plate in addition to the tearing of the steel shell at the top of the gusset plate. In these two specimens, the load is mainly transferred by the fillet weld which imposes moment on the pipe shell due to the eccentricity of loading. For test V-2 and V-3 as shown in Fig. 2e, the observed failure mode was bulging of the pipe shell underneath the gusset plate and tearing of the steel shell at the other side of the pipe. A significant and sudden loss of resistance was observed for the test units without concrete infill due to large plastic deformations observed at the gusset plate zone and beneath the applied load for specimens H-5 and V-5 as shown in Fig. 2f. These results conform to the failure modes reported by Hu et al. [6].

Load–displacement curves

The displacement for the specimens loaded in the perpendicular direction is calculated at the middle of the gusset plate after removing the deformation resulting from the rotation of the
test unit ends around the fixation point. On the other hand, the displacement for the specimens loaded in the parallel direction is calculated at the middle of the gusset plate as the average of the readings of LVDT-1 and LVDT-2 shown in Fig. 1c (ii).

Figs. 3a–3c show the load–displacement curves for each group of test units. Table 3 summarizes the main results inferred from the load–displacement curves. Figs. 4a and 4b show comparison between different details of group B and C with respect to the relative increase in weight, fabrication time, and strength compared to detail 1 (base detail). The following results can be outlined:

– The 12 specimens showed a linear behavior followed by a nonlinear behavior as the loading increased. The nonlinear behavior developed at different loading levels according to the direction of loading and the details of the specimens. For test group A, the nonlinear behavior started at load level around 250 kN. For test group B, the nonlinear behavior of specimens starts at load values ranged from 60 to 80 kN. While for test group C, the nonlinear behavior of specimens starts at load values ranged from 80 to 100 kN.

– The strength of the connections increased for concrete filled columns. The percentage of increase in connection capacity was equal to 55% and 74% for test groups B and C, respectively. This conforms to the results reported by MacRae et al. [3].

– Specimens of test group A attained the highest load capacity due to the increased dimensions of the CFT column and the higher concrete strength properties. Comparing the load–displacement curve for test units SP-H1 and SP-H2, it was observed that test unit SP-H2 reached higher strength capacity; however, the slope of the curve in the nonlinear zone is slightly higher for test unit SP-H1, which indicated that this connection type had a higher strength reserve after attaining permanent deformations.

– For specimens of group B, connection details 3 and 4 had the lowest and highest load capacities with values 96 and 115 kN, respectively. For specimens of group C, connection details 1, 2, and 3 have the highest load capacity with average value 128 kN, while the lowest load capacity was for detail 4 with a value 114 kN.

– The stiffness of test group B was almost identical; however, specimens with gusset plate penetrating through the CFT had a slightly higher capacity which is most pronounced

| Test group | Test unit | Yield load (kN) | Ultimate load (kN) | Initial stiffness (kN/mm) | Post-yield stiffness (kN/mm) | % Increase in strength |
|------------|-----------|-----------------|-------------------|--------------------------|-----------------------------|------------------------|
| A          | SP-H1     | 281.0           | –                 | 108.0                    | 94.7                        | –                      |
|            | SP-H2     | 312.1           | –                 | 115.0                    | 95.8                        | –                      |
| B          | H-1       | 74.0            | 105.0             | 29.1                     | 11.9                        | 103.9                  |
|            | H-2       | 78.1            | 103.7             | 31.4                     | 17.8                        | 101.4                  |
|            | H-3       | 72.0            | 96.0              | 41.4                     | 17.5                        | 86.4                   |
|            | H-4       | 76.0            | 115.0             | 31.0                     | 11.1                        | 123.3                  |
|            | H-5       | 42.9            | 51.5              | 17.7                     | 9.3                         | –                      |
| C          | V-1       | 102.0           | 127.7             | 18.3                     | 12.8                        | 280.1                  |
|            | V-2       | 103.9           | 126.0             | 27.6                     | 17.2                        | 275.0                  |
|            | V-3       | 97.2            | 120.8             | 22.2                     | 19.1                        | 259.5                  |
|            | V-4       | 99.0            | 114.3             | 19.8                     | 19.1                        | 240.2                  |
|            | V-5       | 32.5            | 33.6              | 9.7                      | 7.8                         | –                      |
in test H-3. This behavior is expected due to the addition of a gusset plate that decreased the deformations corresponding to an applied force. Consequently, the lowest stiffness is observed in tests H-1 and H-4, in which the gusset plate was directly connected to the steel tube. In test group 3, the stiffness of tests V-1, V-3, and V-4 was almost identical except for V-2, which exhibits slightly higher stiffness due to the presence of the gusset plate penetrating through the connection. The stiffness of test unit V-5 was less than that exhibited by the other test units in test group C that may be attributed to the change in the eccentricity resulting from the loading procedure which resulted in concentration of stresses at the end of the gusset plate. This zone was weak due to the lack of the concrete infill which resulted in fast deterioration of the specimen.

Axial and hoop strains in pipe

The hoop strain response for test group B is illustrated in Fig. 5a. In addition, Table 4 summarizes the main results inferred from the axial and hoop strain curves. Although tensile hoop strains are expected in CFT sections, tested specimens showed negative values that denote compressive stresses rather than tensile. This can be attributed to the absence of axial load on the CFT column and the direction of loading which imposes compression forces within the zone of connection. It was observed that the bare steel column exhibited fast deterioration. The lowest hoop strain response was exhibited by test V-3. Tests V-4 and V-3 showed ductility values less than the ductility shown by tests V-1 and V-2 by an average value of 32% and 42%, respectively.

The behavior of different connection details must be related to the cost and workability of the connection in order to determine the most economic application of the connection. The load–displacement curves showed that using connections with gusset plates penetrating through the steel tube increases the stiffness and loading capacity under loads parallel to the column’s axis. However, the ratio of increase in loading capacity depends on the thickness and diameter of the steel tube. Moreover, using shear studs at the connection zone increased the load capacity of the connection when loaded parallel to the column’s axis. Meanwhile, the connection detail 4 showed the highest loading capacity under loading conditions perpendicular to the column’s axis along with thin steel tube. Accordingly, connection detail 4 is more economic under small levels of loading; while, using connection details 1, 2, or 3 is more convenient under increased loading conditions.

- The ductility of different connections was compared based upon the ratio between the displacement at failure and displacement at the end of the linear zone. For test group A, the test was stopped before failure, so evaluating the ductility was not possible. For test group B, the ductility of tests H-1 and H-4 was identical. This was expected as the use of shear studs did not restrain the movement under the effect of loading in the perpendicular direction. When comparing the ductility of tests H-2 and H-3, it was found that H-3 is higher by 14% due to the fact that a thinner gusset plate was used in this specimen. Generally, for tests units H-2 and H-3, the ductility decreased by 45% compared to the ductility of test H-1 and H-4. This could be due to the restraining effect of the gusset plate penetrating through the CFT, restraining movement under the applied loads. For test group C, the ductility of tests V-1 and V-2 was almost the same. The least ductility was exhibited by test V-3. Tests V-4 and V-3 showed ductility values less than the ductility shown by tests V-1 and V-2 by an average value of 32% and 42%, respectively.

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Fig. 5b shows the axial strains measured at both sides of specimens for test group C. The neutral axis of the section was almost at the center for tests V-1, V-2, and V-4. It was observed that the strain response for test V-4 is lower than the other units which explains the lower load capacity carried by this joint compared to the other joints. For test V-3, the positive and negative strain responses were not identical which indicated that adding gusset plate in the connection changed...
the distribution of stresses on the section. For test V-5, the compressive strains were larger than the corresponding tensile ones due to deformation of the pipe under the gusset plate.

Conclusions

The effect of different detailing configurations on the behavior of beam–column connections in composite concentrically braced frames with CFT columns was investigated through an experimental program. The tested subassemblies consisted of a part of a CFT column with the studied connection welded at its end. The experiments showed variability in behavior of connections depending on configuration, column dimensions, or loading direction. The main conclusions drawn from the tests are summarized below:

- Local bulging of steel shell under the gusset plate is found to be a dominant failure mode for connections where the gusset plate is directly welded to the steel tube. Meanwhile, the tearing of the steel tube at the tension side is the main failure mode for connections with gusset plate penetrating through the CFT column.
- For connections with strength lower than the adjacent members, brittle failure modes are most likely to occur.
- When using a bare steel column, the strength and stiffness values are dramatically reduced. Moreover, failure is caused by large plastic deformations of the column wall. It is found that the addition of concrete infill increased the capacity of the joints by 55% and 74% for connections loaded parallel and perpendicular to the specimen axis, respectively.

- The load carrying capacity of the connections loaded perpendicular to the specimen axis exhibit higher values for test units with gusset plate welded directly to the steel tube. Meanwhile, the lowest load capacity is observed in the test unit imposing higher loads on the steel thin shell. On the other hand, the load carrying capacity of the connections loaded parallel to the specimen axis show higher values for test units with gusset plate penetrating through the steel tube. This indicates that the behavior of connections depends to some extent on the inclination of the brace force.
- The stiffness of connections is almost identical when loaded perpendicular to column axis; however, connections with gusset plate penetrating through the CFT column showed higher stiffness values when loaded parallel to the column axis.

Conflict of interest

The authors have declared no conflict of interest.

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References

[1] The Egyptian code of practice for steel construction and bridges. Research Center for Housing, Building and Physical Planning, Giza, Egypt; 2005.
[2] Roeder CW, MacRae GA, Gunderson CA, Lehman DE. Seismic design criteria for CFT braced frame connections. In: Proceedings of the international workshop on steel and concrete composite construction. Taipei, Taiwan; 2003. p. 97–106.
[3] MacRae GA, Roeder CW, Gunderson CA, Kimura Y. Brace–beam–column connections for concentrically braced frames with concrete filled tube columns. J Struct Eng (ASCE) 2004:233–43.

**Table 4** Summary of test results for different test units for strain curves.

| Test group | Test unit | Yield hoop strain (µε) | Ultimate hoop strain (µε) | Yield axial strain (µε) | Ultimate axial strain (µε) |
|------------|-----------|------------------------|---------------------------|------------------------|---------------------------|
| B          | H-1       | −91.9                  | −310.0                    | −                      | −                        |
|            | H-2       | −80.6                  | −260.4                    | −                      | −                        |
|            | H-3       | −42.0                  | −594.1                    | −                      | −                        |
|            | H-4       | −217.7                 | −694.4                    | −                      | −                        |
|            | H-5       | −447.0                 | −785.8                    | −                      | −                        |
| C          | V-1       | −                      | −                        | 2961.0, −3061.0        | 8913.2, −8850.0           |
|            | V-2       | −                      | −                        | 2370.2, −3424.0        | 9394.2, −8898.0           |
|            | V-3       | −                      | −                        | 1441.0, −4228.9        | 1985.8, −9390.9           |
|            | V-4       | −                      | −                        | 2384.8, −4011.2        | 6465.8, −8801.2           |
|            | V-5       | −                      | −                        | 225.9, −1756.9         | 538.5, −5594.9            |

**Fig. 5b** Axial strains of group C tests.

![Graph](image-url)
[4] AISC (2010). Draft specification for structural steel buildings. ANSI/AISC 360-10. Chicago (IL): American Institute of Steel Construction, Inc.; 2009.

[5] Jacobs WP, Hajjar JF. Load transfer in composite construction. Proceedings of the ASCE/SEI structures congress. Orlando, Florida; 2010.

[6] Hu HT, Chen CW, Huang MY. Nonlinear finite element analysis of CFT-to-bracing connections subjected to axial compressive forces. J Eng Struct 2011:1479–90.

[7] Whitmore RE. Experimental investigation of stresses in gusset plates. Engineering Experiment Station, University of Tennessee. Report No.: 16; 1952.

[8] Yam MCH, Cheng JJR. Behavior and design of gusset plate connections in compression. J Constr Steel Res 2002.

[9] Roeder CW, Lehman DE, Yoo JH. Performance-based seismic design of braced-frame gusset plate connections. J Connect Steel Struct 2004.

[10] Gross JL. Experimental study of gusseted connections. J Eng (AISC) 1990.

[11] Tsai KC, Hsiao PC. Pseudo-dynamic test of a full-scale CFT/BRB frame – Part II: Seismic performance of buckling-restrained braces and connections. J Earthq Eng Struct Dyn 2008;37:1099–115.