Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte
Band: 999 (1997)

Artikel: Detailing requirements for concrete-filled steel tubes connections
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DOI: https://doi.org/10.5169/seals-1036

Detailing Requirements for Concrete-Filled Steel Tubes Connections

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After receiving his M.S.C.E., Stephen P. Schneider worked several years at Skilling Ward Magnusson Barkshire, Inc a structural engineering firm in Seattle, WA. Among other notable buildings, this firm designed Two Union Square which is well known for its use of concrete-filled steel tubes and high performance concrete. In 1991, Stephen P. Schneider received his Ph.D. in Civil Engineering at which time he joined the faculty at the University of Illinois.

Summary

This paper summarizes experimental research on connections to concrete-filled steel tubes. Results suggested that transferring the girder force exclusively to the face of the tube wall led to fracture of the tube wall, flange and weld. External diaphragms alleviated extreme deformation demand on the tube, and enabled the connection to develop the flexural strength. Connections with components that penetrated the tube wall initiated a plastic hinge in the connected girder. However, inelastic cyclic behavior depended on the type of elements embedded.

1. Introduction

Concrete-filled tube [CFT] columns provide many advantages over the more conventional reinforced concrete or structural steel columns. Some of these advantages include: the steel tube provides formwork for the concrete core, the concrete prolongs local buckling of the steel tube wall, the tube prohibits excessive concrete spalling, and the composite column adds significant stiffness compared to traditional steel frame construction. While many advantages exist, the use of CFTs in building construction has been limited. This lack of use is due, in part, to limited construction experience and to the complexity of connection detailing. This paper summarizes part of an experimental program to study a variety of details to connect a steel girder to a circular CFT column.

Test data exists on a variety of connection details. Details that connect the girder to the steel tube only include: welding the girder directly to the tube skin (Valbert, 1968), using web angles or shear tabs to connect the girder to the tube (Shakir, 1992; Bridge, 1992), providing external or internal diaphragms (Kato et. al., 1992; Morino et.al., 1992), and variations on the above details (Ansourian, 1976). Conclusions from these studies suggest that connections loading the steel tube exclusively can cause excessive deformation demand on the tube wall and connection components. Connections that attempted to improve this behavior include: through bolting girder end plates (Prion et. al., 1992; Kanatani et. al., 1987), and continuing structural steel shapes through the column (Azizinamini et. al., 1992). Comparison of test data suggests that embedding connection components into the concrete core alleviates high shear demand on the tube wall, which may improve the seismic performance of the connection. The object of this research was to investigate the inelastic flexural behavior of a wide range of connection details.

Six connections tested in this research program are shown in Fig. 1. The primary interest of this research program was to investigate connections that develop the flexural strength of the

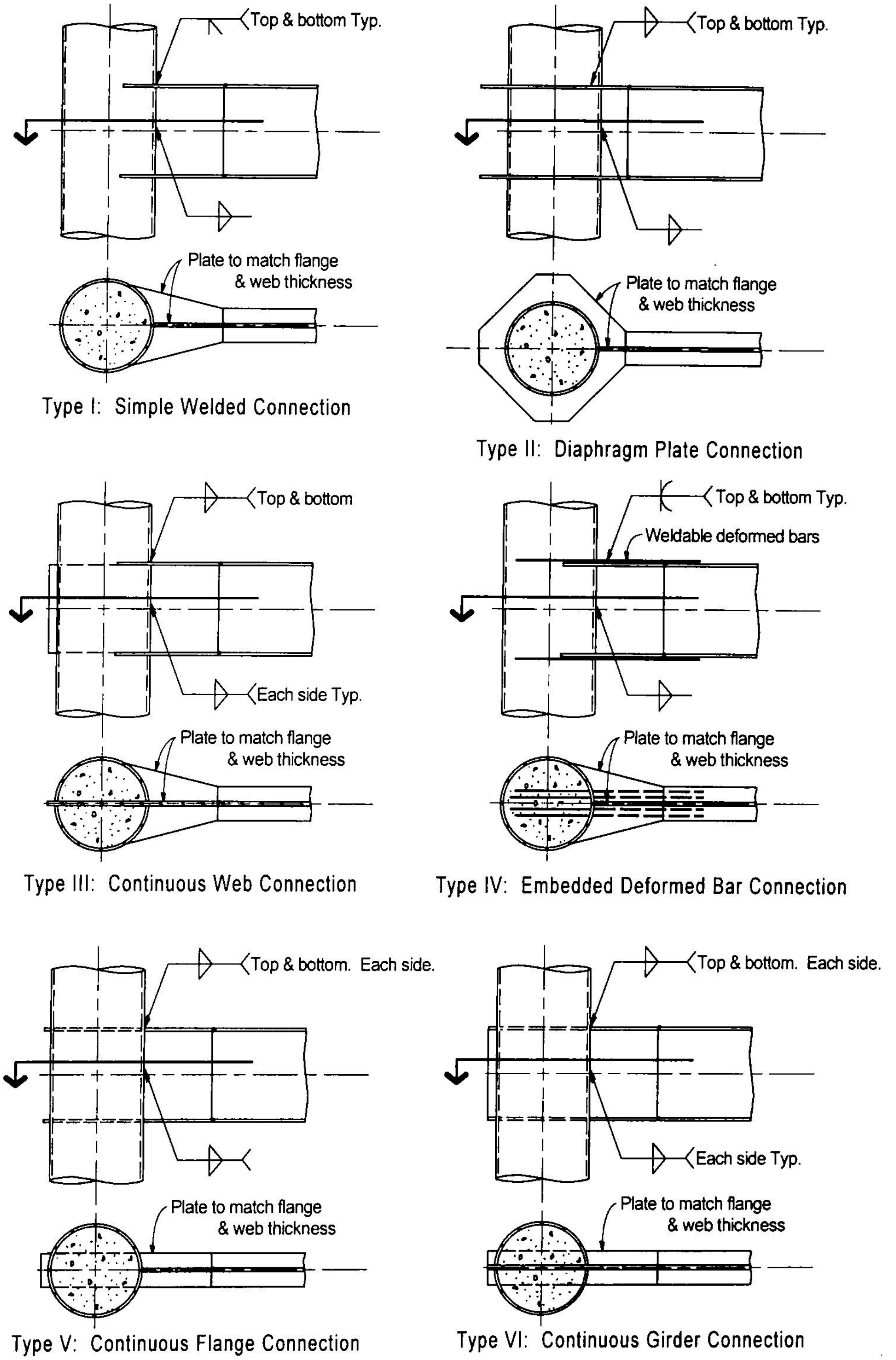

Figure 1. Connection Details

connected girder. Since connection behavior was the primary interest, and not joint or panel zone behavior, the girder connected to only one side of the CFT column was sufficient. Because of the difficulty in connection detailing, only circular tubes were considered. Each test specimen consisted of a *356 mm* diameter pipe with a *6.4 mm* wall thickness, and a *W14x38* for the girder. The yield strength for the pipe and the girder was *320 MPa*, with an approximate concrete strength of *35 MPa*. Cyclic deformation were imposed on all girders according to the *ATC-24* (1992) guidelines for the cyclic testing of components.

2. Experimental Study

The inelastic, cyclic moment-rotation [M- θ] behavior for each connection is shown in Fig. 2. The moment was normalized with respect to the plastic bending strength of the girder.

For the simple *Type I* connection, the girder was welded directly to the skin of the steel tube. Consequently, the steel tube was subject to high local distortions adjacent to the connected region. Fracture initiated in the connection stub flange, at a rotation of 1.5%. This fracture propagated into the tube wall at approximately 3.2% rotation. This tearing propagated from the tips of the flange toward the web. Only one flange fractured, the tube wall separated from the concrete core as the other flange was subjected to tension. This resulted in an unsymmetric M-θ behavior, and a pinching of the hysteretic curves. Degradation in flexural strength began once the flange fractured, and continued upon subsequent deformation cycles. This fracture precipitated high shear demand on the web plate, which led to fracture of the weld between the web and the pipe wall. Consequently, the girder lost all shear capacity shortly after flange failure.

The external diaphragms of connection *Type II* were intended to alleviate the severe distortion on the steel tube skin. These diaphragms improved the cyclic behavior of the simple connection significantly. One diaphragm fractured at approximately 0.5% rotation, while the other diaphragm fractured at almost 1.0% rotation. Upon subsequent cycles, the fracture propagated in each diaphragm, eventually tearing the tube wall at a rotation of 3.5%. Deterioration of the M-θ behavior occurred at the onset of diaphragm fracture. At large cyclic displacements, the diaphragm buckled, and the tube wall fractured along the depth of the girder.

The intent of the continuous web *Type III* detail was to improve the shear behavior of the simple Type I connection. However, significant web tearing was still observed once the flanges failed. Similar to the simple connection, the flange fracture exacerbated the high strain demands on the extreme fibers of the web. Due to embrittlement of the web in the heat affected zone, tearing initiated close to the fillet weld that attached the web to the pipe wall. Fracture initiated at approximately 1.25% rotation, precipitating a 20% decrease in peak flexural strength by the end of these imposed deformation cycles. Eventually, this fracture propagated from each flange toward the center of the web. A significant portion of the web was fractured by approximately 2.5% rotation, resulting in poor hysteretic performance of the connection for larger imposed rotations.

Connection *Type IV* was identical to connection *Type I*, except four $20\ mm\ \phi$ ($F_y = 420\ MPa$) weldable deformed bars were embedded in the concrete core through holes drilled in the steel tube wall. Each deformed bar was then welded to the girder flange. Embedment lengths were sufficient to develop the deformed bar under tested conditions. The M-θ behavior of this detail was a significant improvement compared to connection *Type I*. Initial wall tearing was observed at 3.5% rotation. However, this tearing was located only in the tube wall between the openings for the deformed bars. This minor tearing did not affect the inelastic performance of this connection. Local flange buckling was also observed at 3.5% rotation, which occurred in the girder beyond the connection region. This suggests that the connection was strong enough to initiate significant yield in the girder flange. Failure of this connection was due to fracture of the deformed bars at approximately 5.0% rotation. Three of the four bars failed by tension rupture, while one bar pulled out of the concrete core. No significant stiffness or strength deterioration

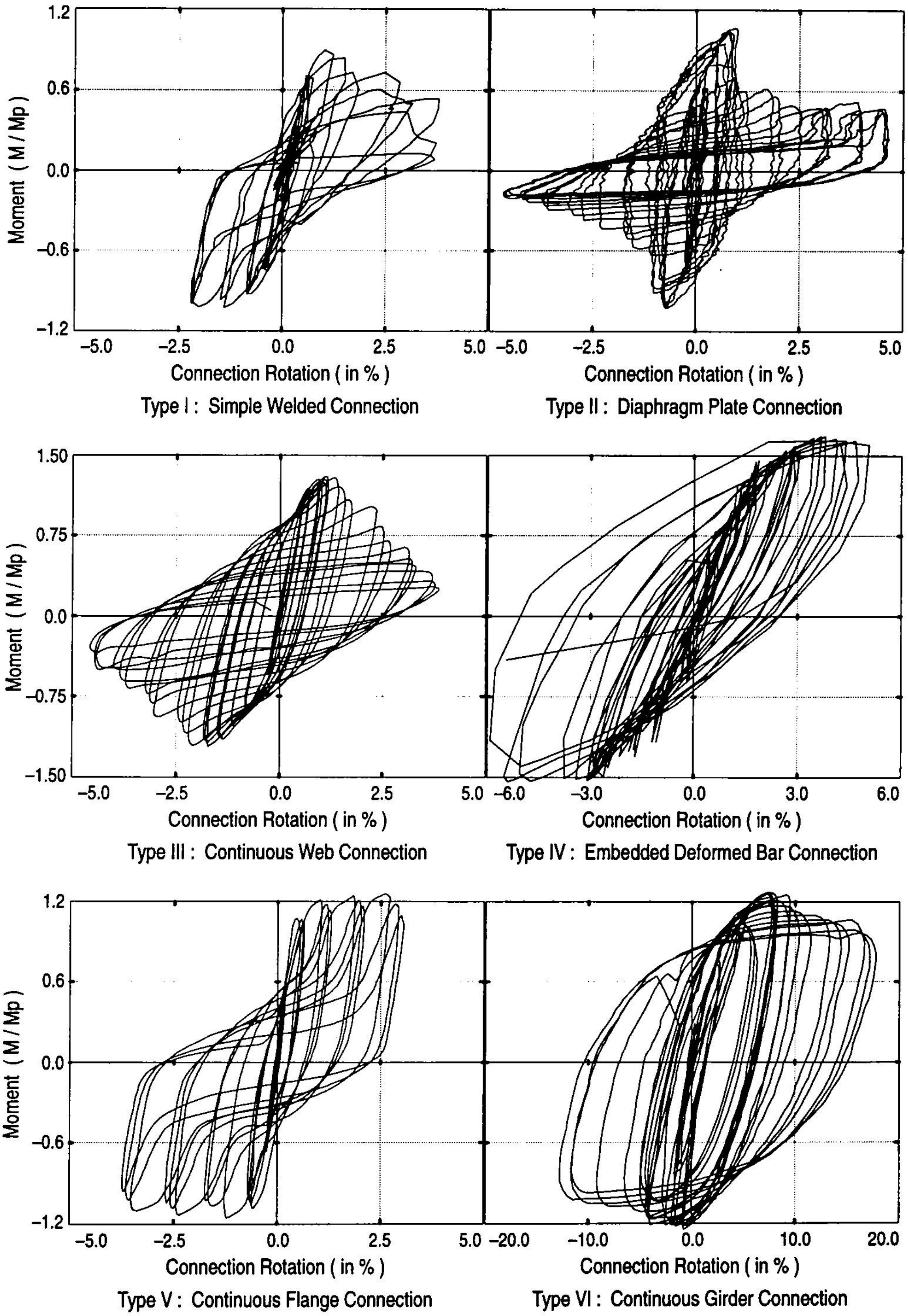

Figure 2. Experimental Moment–Rotation Behavior of Each Connection

was observed prior to the fracture of the deformed bars. Each deformed bar that failed in tension ruptured between the tube wall and the first weld location attaching the bar to the girder flange.

The *Type V* connection detail was tested to study the effect of continuing only the flanges through the CFT column. Flange plates were welded to the tube wall on each side of the column to transfer girder flange forces. The resistance was to be provided by bearing of the steel tube against the concrete core as needed. No attempt was made to enhance the bond between the embedded plates and the concrete core. The lack of bond impacted the hysteretic performance of this connection significantly compared to the other connection types. Both flange welds on the girder side of the column connection fractured at approximately 0.5% rotation. The flange welds on the column-side opposite the girder remained in tact. This caused the flanges to push through the concrete core resulting in fracture of the steel tube. This tube wall fracture initiated at approximately 1.0% rotation. As new deformation amplitudes were reached the connection resistance increased, but this resistance diminished upon subsequent cycles. This resulted in excessive pinching of the hysteretic behavior.

Results for connection *Type VI* exhibited quite stable inelastic behavior. Local flange buckling was observed at approximately 4.0% rotation, and web buckling was observed at about 5.0% rotation. Deterioration of the inelastic characteristics were observed after the onset of the local web buckling. This connection was clearly able to develop the plastic bending strength of the girder. Failure of this connection was due to fracture of the beam flange in the connection stub region. This flange tearing eventually propagated into the web. Although the flexural strength decreased approximately 30% compared to the peak value, hysteretic behavior remained stable even at very large rotations. After the test, the steel skin was removed from the concrete core in the region around the connection. No crushing of the concrete core was observed, and the tube wall showed no apparent signs of distress.

3. Conclusions

In general, inelastic connection behavior improved significantly when a larger portion of the girder force was transferred to the concrete core. However, the inelastic performance depended significantly on the connection detail. Some conclusions from this study are worth noting:

1. Connection *Type I* could not develop the plastic bending strength of the girder, while the flange, weld, and tube wall were susceptible to fracture. This connection lost almost all of its flexural strength at moderate inelastic demands. This connection should not be used in moment-resisting frames in regions of moderate to high seismic risk.

2. Connection *Type II* was able to develop the yield strength of the girder, however, the inelastic properties deteriorated rapidly at the onset of the diaphragm fracture. This connection might be used in low to moderate seismic zones.

3. Extending the web through the connection, as in detail *Type III*, improved the inelastic performance of the simple connection. However, the *Type III* connection still experienced significant web tearing and an eventual deterioration of the shear capacity. Partially embedded shear tabs may offer more favorable shear resistance for the girder if the flanges or steel tube skin fracture.

4. Weldable deformed bars transferred much of the flange stresses into the concrete core. This connection showed considerable improvement over simple connection *Type I*, and could be used in regions with moderate to high seismic activity.

5. Continuing flange plates, as tested in connection *Type V*, did not produce satisfactory inelastic, cyclic behavior. The inability of the weld to transfer flange forces to bearing stresses to the concrete core on either side of the connection resulted in large deformations with little resistance unless a new deformation amplitude was imposed. This produced a significantly pinched hysteretic behavior.

6. The *Type VI* through connection exhibited favorable hysteretic behavior. This appeared to be the most effective method to develop the plastic bending strength of the steel girder, and was closest to representing ideal rigid connection conditions. While there appeared to be a 30% flexural strength decrease, the subsequent flexural resistance was equivalent to the plastic bending strength of the girder. Further, the cyclic behavior remained stable even at very large rotations.

Acknowledgments

This research was funded by the National Science Foundation under Grant No. **NSF CMS 93-00682** with Dr. S.C. Liu as program director. This support is gratefully acknowledged. The opinions in this paper are those of the author, and do not necessarily reflect the opinions of NSF.

References

ATC-24 (1992). Guidelines for Cyclic Seismic Testing of Components for Steel Structures. *Applied Technology Council.* Redwood City, CA.

Azizinamini, A. and Prakash, B. (1993). A Tentative Design Guidelines for a New Steel Beam Connection Detail to Composite Tube Columns. *AISC Engineering Journal*, 3rd quarter, 108-115.

Ansourian, P. (1976). Connections to Concrete-Filled Tube Columns. *International Association of Bridge and Structural Engineers*, **36-I**, 1-22.

Bridge, R. and Webb, J. (1992). Thin Walled Circular Concrete Filled Steel Tubular Columns. *Composite Construction of Steel and Concrete II, ASCE*, 634-649.

Kanatani, H., Tabuchi, M.. Kamba, T., Hsiaolien, J., and Ishikawa, M. (1987). A Study on Concrete Filled RHS Columns To H-Beam Connections Fabricated with HT Bolts in Rigid Frames. *Composite Construction of Steel and Concrete, ASCE*, 614-635.

Kato, B., Kimura, M., Ohta, H. and Mizutani, N. (1992). Connection of Beam Flange to Concrete-Filled Tubular Columns. *Composite Construction of Steel and Concrete II, ASCE*, 528-538.

Morino, S., Kawaguchi, J., Yasuzaki, C., and Kanazawa, S. (1992). Behavior of Concrete Filled Steel Tubular Three Dimensional Subassemblages. *Composite Construction of Steel and Concrete II, ASCE*, 726-741.

Prion, H.G.L., and McLellan, A.B. (1992). Connecting Steel Beams to Concrete-Filled Steel Columns. *In Proceedings, ASCE Structures Congress on Composite Compression Members*, San Antonio, TX, 918-921.

Shakir-Khalil, H. (1992). Full Scale Tests on Composite Connection. *Composite Construction of Steel and Concrete II, ASCE*, 539-554.

Valbert, G. (1968). Essais d' Assemblages Soudes d'une Solive Sur un Poteau en Tube Rempli de Beton. *Construction Metallique*, 4 Dec., 27-38. (in French).