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# CONNECTIONS TO CONCRETE–FILLED STEEL TUBES

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## ABSTRACT

An analytical and experimental investigation on the connection of structural steel shapes to concrete–filled tubular columns is presented. Feasible connection alternatives were developed and investigated analytically using nonlinear 3–D finite element models for each connection detail. Analytical results identified the efficiency of various components in the connection, and some details were altered in response to these results. Several promising details were tested using large–scale beam–to–column connection specimens. Experimental results suggest that connection details that attach to the steel tube alone may result in premature fracture of the steel girder flange or weld, thus preventing development of the full plastic bending strength. Further, girders connected exclusively to the steel tube wall can result in premature tearing of the tube wall reducing the confinement of the concrete core near the connection. Connections with embedded components can develop the plastic strength of the connected girder, however, the post–yield strength, stiffness and ductility depended on the connection detail. Extending the girder through the column resulted in large energy dissipation, and was clearly able to develop the full plastic strength of the girder. However, this connection was more expensive compared to other connection details, and may cause some construction difficulties.

## KEYWORDS

Composite Frames, Concrete–Filled Steel Tubes, Composite Columns, Connections, Nonlinear Finite Element Analysis, Earthquake Resistant Structures.

## INTRODUCTION

Concrete–filled steel tubes (CFTs) combine the ductility generally associated with steel structures with the stiffness of concrete components. The concrete–filled steel tube has many advantages compared to other composite column types. Some of these advantages include: the steel tube provides a convenient formwork for the concrete, the steel pipe provides continuous confinement for the concrete core, the pipe prevents spalling of concrete, and the concrete core delays local buckling of the steel pipe. Because of the continuous confinement, the concrete–filled composite column is perceived as offering favorable ductility. Consequently, concrete–filled tube columns may be well suited for buildings constructed in regions at high seismic risk. However, the use of concrete–filled steel tubes has been limited due, in part, to the difficulties in the design and detailing of satisfactory connections, and to the lack of construction experience of such columns. This paper describes analytical and experimental research on the connection of structural steel girders to concrete–filled circular steel tubes. Connections were designed to develop the plastic bending capacity of the structural steel cross section.

Connections to steel tubes can be categorized into two groups: *1)* connections that load the tube wall only, and *2)* connections that penetrate the steel skin and embed components into the concrete core. Several tests have been performed on details classified under *Group 1*. These tests include: welding the girder directly to the tube skin (Valbert, 1968), using web angles or shear tabs to connect girders to tubular columns (Shakir, 1992; Bridge, 1992), providing external or internal diaphragms (Kato *et. al.*, 1992; Morino *et. al.*, 1992), and test specimens with variations on the above details (Ansourian, 1976). Connections that load the steel tube exclusively can cause premature yielding or fracture of the tube wall. This may prohibit adequate performance of a connection detail during a strong seismic event. Further, recent earthquakes have exposed some flaws associated with details in which the steel tube column was discontinued for the diaphragm plates (Toyoda, 1995).

Embedded components that penetrate the steel tube wall, *Group 2* type connections, are generally used in an attempt to transfer part of the girder forces to the concrete core. These embedded components include headed studs (Hawkins *et. al.*, 1980), through bolting girder end plates (Prion *et. al.*, 1992; Kanatani *et. al.*, 1987), continuing structural steel shapes through the column (Azizinamini *et. al.*, 1992), and utilizing weldable deformed bars.

Both connection groups are represented by the details investigated in this research program. A representative sample of the analytical and experimental results are presented in this paper.

## CONNECTION DETAILS

The object of this research was to investigate the elastic and inelastic flexural behavior of a variety of connection details. This research focused on connections to circular CFTs only, primarily because these type of connections tend to present more difficulties in design than the square tube connection counterpart. A sample of some of the connections investigated in this study are shown in Figs. 1 through 5. In response to the welded connection problems noted after the Northridge and Kobe earthquakes each connection was designed using a beam stub. This stub is fabricated in the shop where higher quality control is perceived to exist compared to field construction. The field splice can take place outside the critical connection region, however this splice was beyond the scope of this research.

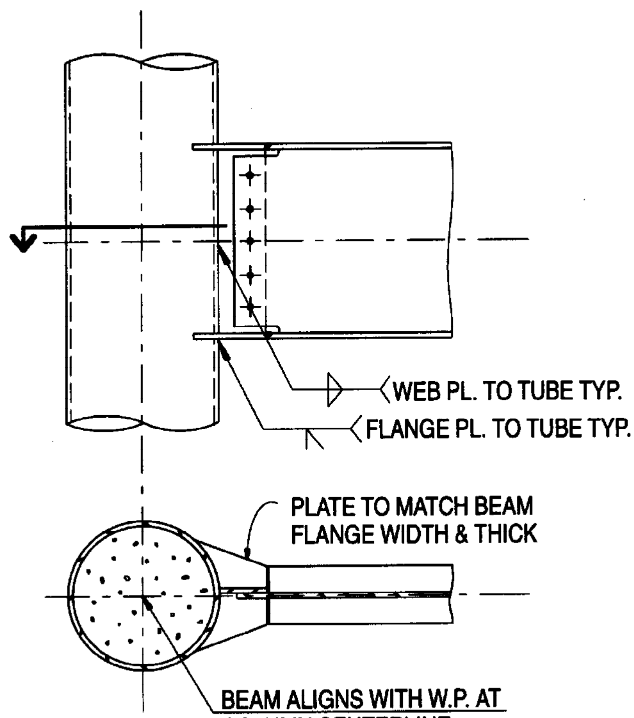


Fig. 1. Simple Connection ( Type I )

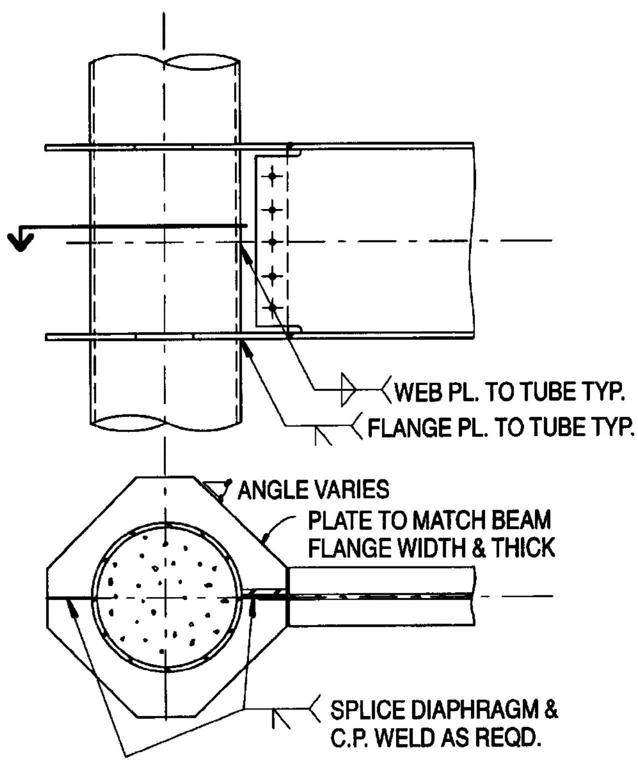


Fig. 2. Diaphragm Plates ( Type II )

While certain connection details are conducive to small diameter CFTs, other connections may be more suitable for large diameter pipes. For example, the simple connection shown in Fig. 1 may be more accommodating for large diameter pipes, while the diaphragm plates in Fig. 2 may be more suitable for small diameter CFTs. Although the type of connection may depend on diameter, this research attempted to investigate a broad range of details on a moderate size CFT. Therefore, the pipe diameter for this study remained constant, to allow comparison of inelastic behavior between various connection details.

For most circular CFTs, the joint was not considered to be an issue, thus connection behavior was the primary concern of this research. Therefore, connections with girders framing to one side of the column only were studied. This is equivalent to a corner column in a perimeter frame, neglecting the orthogonal girder, in which CFTs have clear advantage over more conventional structural elements. Although connections to one side only were investigated experimentally, connections that might be influenced by girders framing normal to, or on the far side of, the CFT were studied using calibrated analytical models.

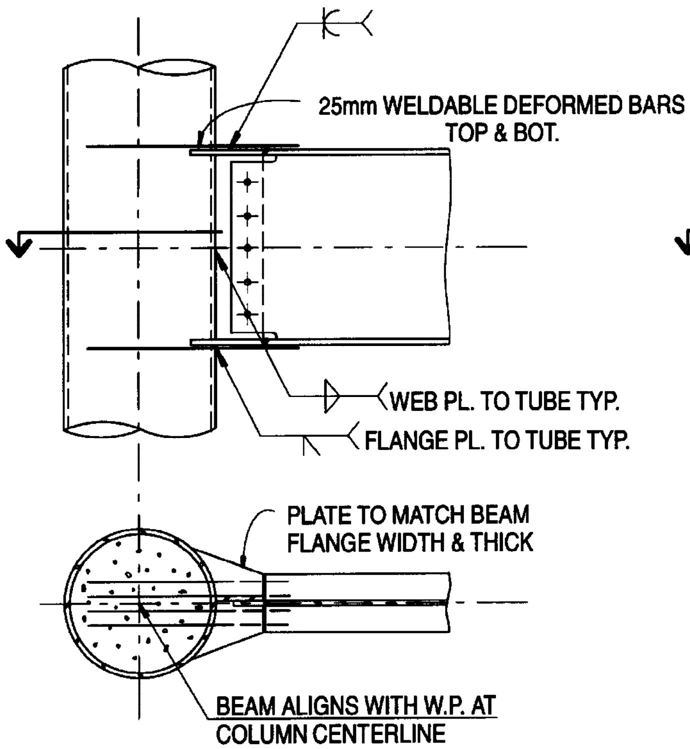


Fig. 3. Embedded Weldable Deformed Bars ( Type III )

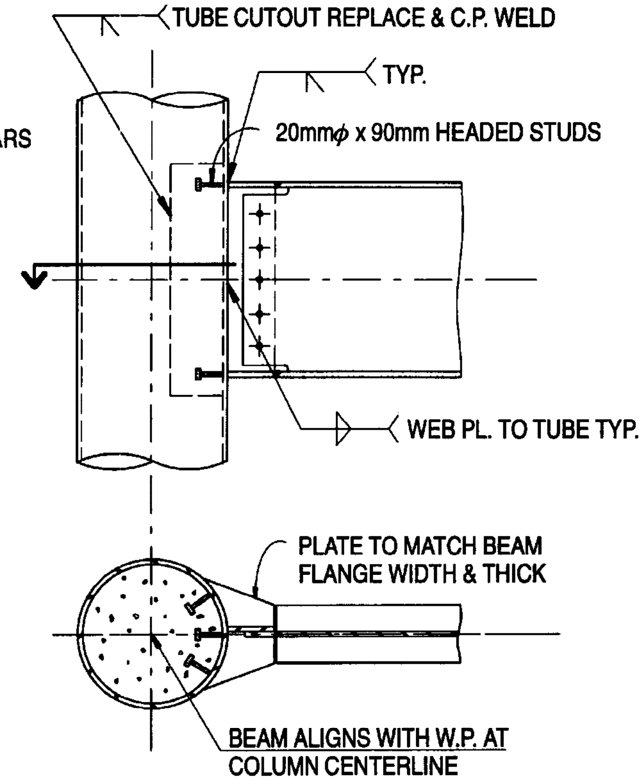


Fig. 4. Interior Headed Studs ( Type IV )

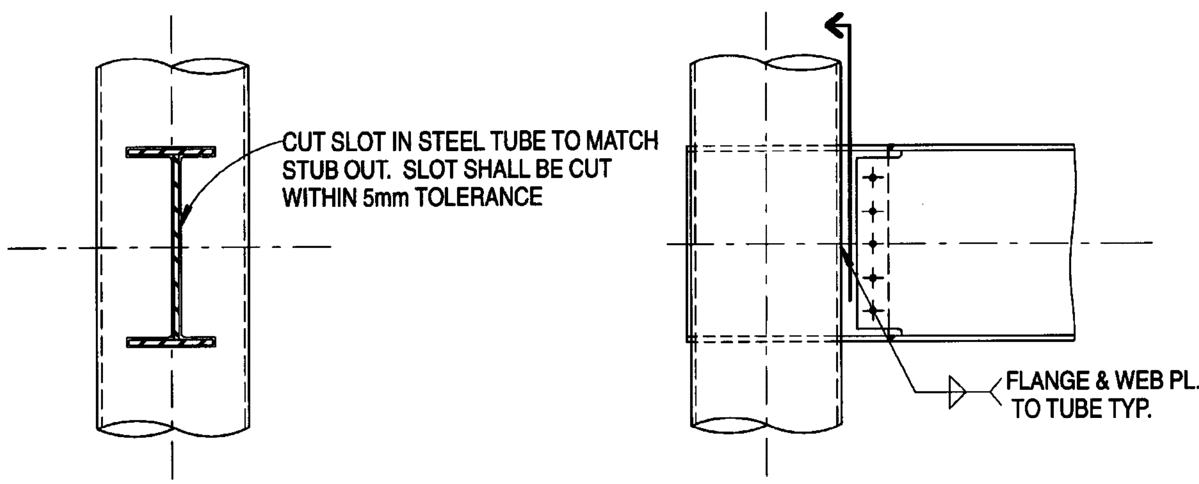


Fig. 5. Girder Continuous through CFT ( Type V )

## ANALYTICAL INVESTIGATION

### *Finite Element Analysis*

To determine the appropriateness of all connections considered, each detail was initially studied using a 3 dimensional nonlinear finite element analysis. All models were generated by PATRAN, and analyzed using ABAQUS. The steel tube wall and the girder were modeled using an 8–node shell element with five degrees of freedom at each node. The concrete core was modeled using a 20–node 3–D brick element. The available material inelastic and geometric nonlinear models in ABAQUS were used for both element types. The interface between the concrete and steel at each boundary was carefully investigated and modeled. Due to the plane of symmetry, only one half of the subassembly was analytically modelled. A hinge was modeled on one end of the CFT, and a roller support was modeled on the other end. For the roller end, the CFT was free to translate in the longitudinal direction to accept an axial load. Axial stresses were applied to the steel tube and concrete core simultaneously, while the shear and moment were imposed on the cantilever portion of the steel girder.

Parameters studied by the 3–D finite element analysis, for each connection type, include: diameter–to–tube thickness ratios of 40.0, 53.3, and 80.0; applied–to–squash load ratios of 0.0%, 6.0%, 18.0% and 36.0%; steel pipe yield strengths of 248 MPa and 320 MPa; and moment–to–shear ratios of 0.5 M, 1.25 M, and 7.5 M. Additional studies were performed, when needed, for girders framing orthogonal, or on the far side, of the CFT column.

Analytical results provided information on the monotonic bending capacity of the connection detail, and on the effectiveness of various components needed for each detail. Due to the results of this analytical investigation, several connection details were modified, and some details were found to be unsatisfactory for seismic demands. The moment– rotation curves for the connections shown in Figs. 1 though 5 are illustrated in Fig. 6. The curve identified as ideal behavior depicts the condition assuming rigid connection behavior.

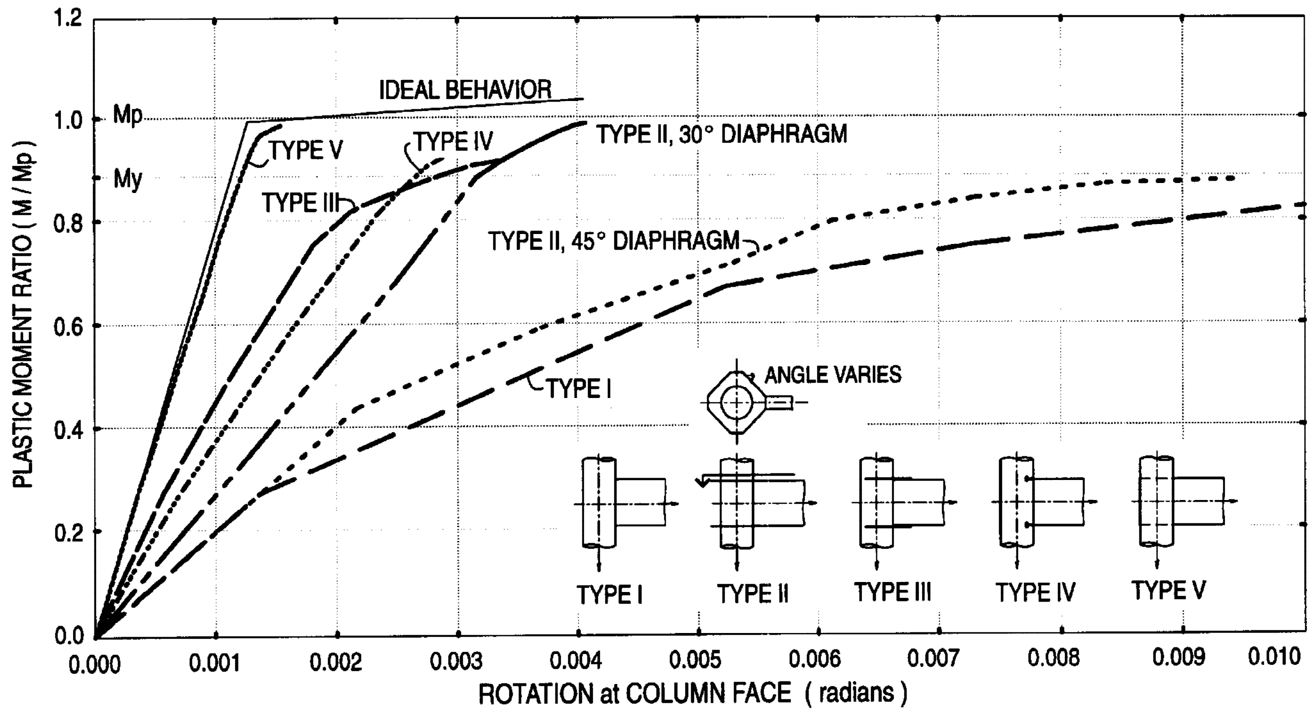


Fig. 6. Analytical Moment–Rotation Behavior for Connections

The apparent large ductility associated with connection *Type I* was due to the flexibility of the tube wall. However, this severe distortion may lead to fracture of the steel tube, and may prevent development of the plastic bending strength of the girder. Increasing the tube wall thickness in the vicinity of the connection improved the connection performance slightly, however this marginal improvement may not justify the additional fabrication cost.

The use of external diaphragms, as in *Type II*, can improve the strength and stiffness characteristics of the *Type I* connection, however the size and shape of the diaphragm plate becomes significant. The 45° diaphragm located the girder too close to the CFT wall, and the behavior was similar to that of the simple *Type I* connection. However, shifting the girder splice further from the CFT face, such as varying the angle of the plate as was done in this study, allowed the stresses to flow around the CFT and through the diaphragm plate. Larger diaphragm plates for this purpose however, may result in large width–to–thickness ratios of the compression diaphragm plate.

Connection *Type III* with added weldable deformed bars increased the stiffness and strength of the *Type I* connection significantly. This improved behavior was due to the weldable deformed bar transferring the flange stresses away from the steel tube wall and into the concrete core. Results from the finite element analysis suggest that the flange force transferred to the reinforcing bars was not sensitive to the pipe wall thickness, although the flanges in this study were welded to the tube wall. For different tube wall thickness, analytical results suggest that approximately 70% of the flange forces were transferred to the concrete core by the tension bars, and about 65% of the flange force was resisted by the compression bars.

The analytical results for connection *Type IV* suggest that the strength of connections with headed studs was limited by the strength of these studs. These trends seem to be verified by experiments as reported by Hawkins *et. al.* (1980). Further, the behavior of this type of connection is greatly influenced by the moment–to–shear ratio on the connection. The D/t ratio, the yield strength of the pipe, and the axial load ratio were shown to have little influence on the connection behavior. The headed studs on the compression flange resisted up to 40% of the girder shear force, while the studs in tension resisted only about 4% of the shear force. This difference was due primarily to the excessive yield in the tension studs which reduced the remaining shear capacity.

Extending the girder through the CFT, as in connection *Type V*, showed analytically that this was perhaps the most effective method to approximate the rigid connection condition. Analytical results for this connection do not indicate overstressing of the concrete core in the connection or joint region. Further, there are no signs of overstressing in the pipe wall. Thus, analytical results suggest that the likely mode of failure will be by the formation of a plastic hinge in the girder.

## EXPERIMENTAL INVESTIGATION

The analytical study identified several viable connection alternatives. The more promising connections were constructed and tested at the Newmark Laboratory at the University of Illinois. A schematic of the test apparatus is shown in plan view in Fig. 7. Each test specimen consisted of a W14x38 girder connected on one side of a 355mm $\phi$ concrete–filled steel pipe. These sizes constitute a reasonably large–scale subassembly, approximately 2/3–scale or larger, compared to the element sizes needed for an 8 story, 5 bay and a 14 story, 3 bay prototype perimeter frame design. The target concrete strength for all specimens was 35 MPa, and the specified strength for the steel girder and pipe were 248 and 320 MPa, respectively.

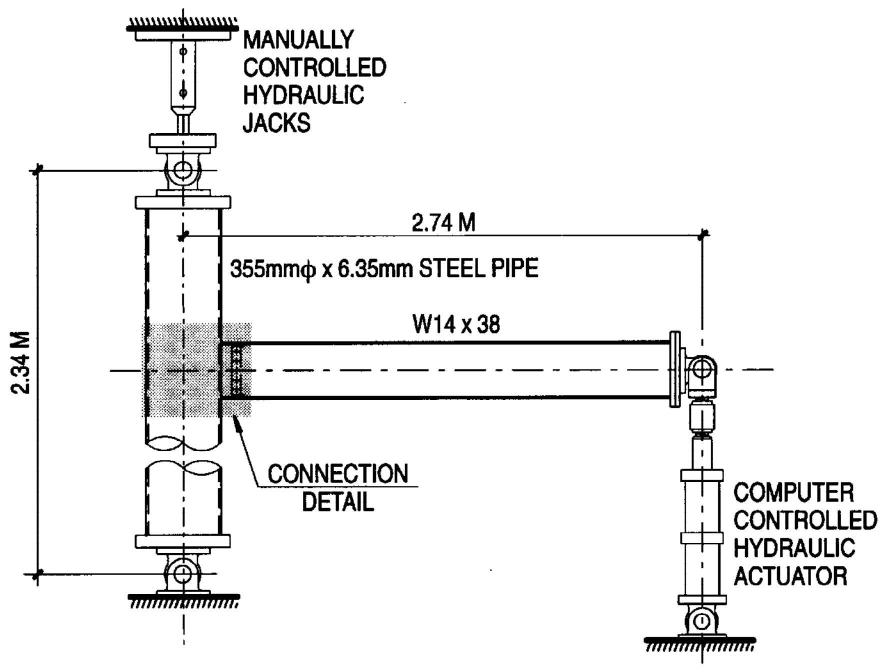


Fig. 7. Plan View of Test Set–Up

The cyclic displacement history imposed at the cantilever girder tip was consistent with the quasi–static test guideline as specified by the *ATC–24: Guidelines for Cyclic Seismic Testing of Components of Steel Structures* (1992). During each test, a constant axial force of approximately 10% of the squash load was maintained in the CFT column by the manually controlled hydraulic jacks. Axial load was applied to the steel tube and concrete core simultaneously. Details representing connection *Types I, III, and V* were tested in the initial phases of the experimental study. Figures 8 through 10 show the hysteretic load–displacement relations for these tested connections. On the vertical axis, the load is normalized by $P_U$, where $P_U$ is the load needed to cause a plastic hinge in the girder, using actual material properties, at the connection stub location. For comparison with experimental results, the analytical monotonic force–displacement behavior is illustrated on each graph. As shown by these figures, a good correlation was observed between the analytical and experimental results.

The failure mode of connection *Type I* was by fracture of the flange in the heat affected zone, fracture of the flange weld, and by rupture through the thickness of the tube wall just above the flange. Eventually, approximately half the flange failed by plate or weld fracture, and the other half failed by tube wall tearing. Fracture propagated from the tips of the flanges toward the web. This fracture precipitated premature strength and stiffness deterioration as noted by the experimental results shown in Fig. 8. It is of interest to note that this fracture occurred in one flange only. Tension in the opposite flange resulted in the tube wall separating from the concrete core, where no weld or tube wall fracture was observed. This behavior explains the somewhat lopsided nature of the force–displacement curve, and the pinching

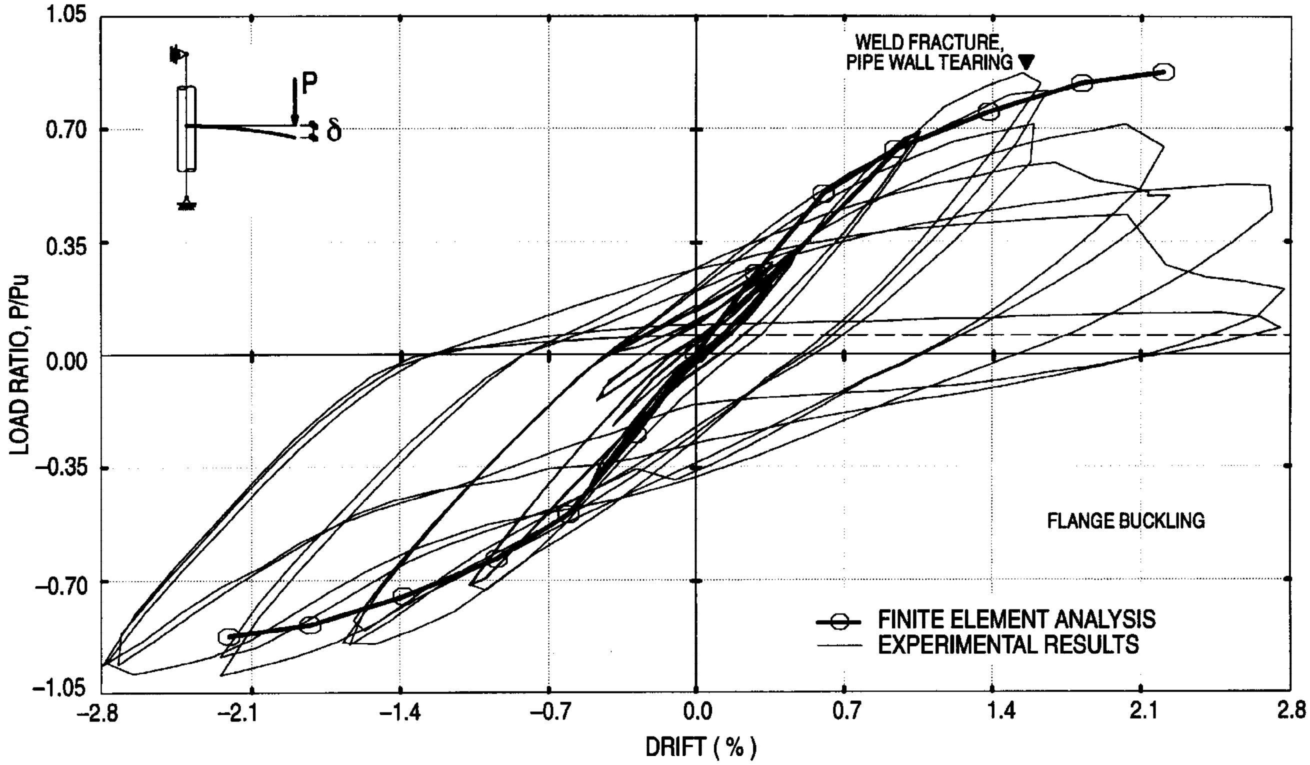


Fig. 8. Load–Deflection Behavior: Connection Type I.

of the hysteresis. The flange fracture eventually propagated to the web, leading to the failure of the weld between the web and the pipe wall. Consequently, the girder lost all shear capacity by the end of the test. This detail sustained only a few cycles of inelastic deformations, and satisfactory performance was observed only up to a drift of about 1.4%.

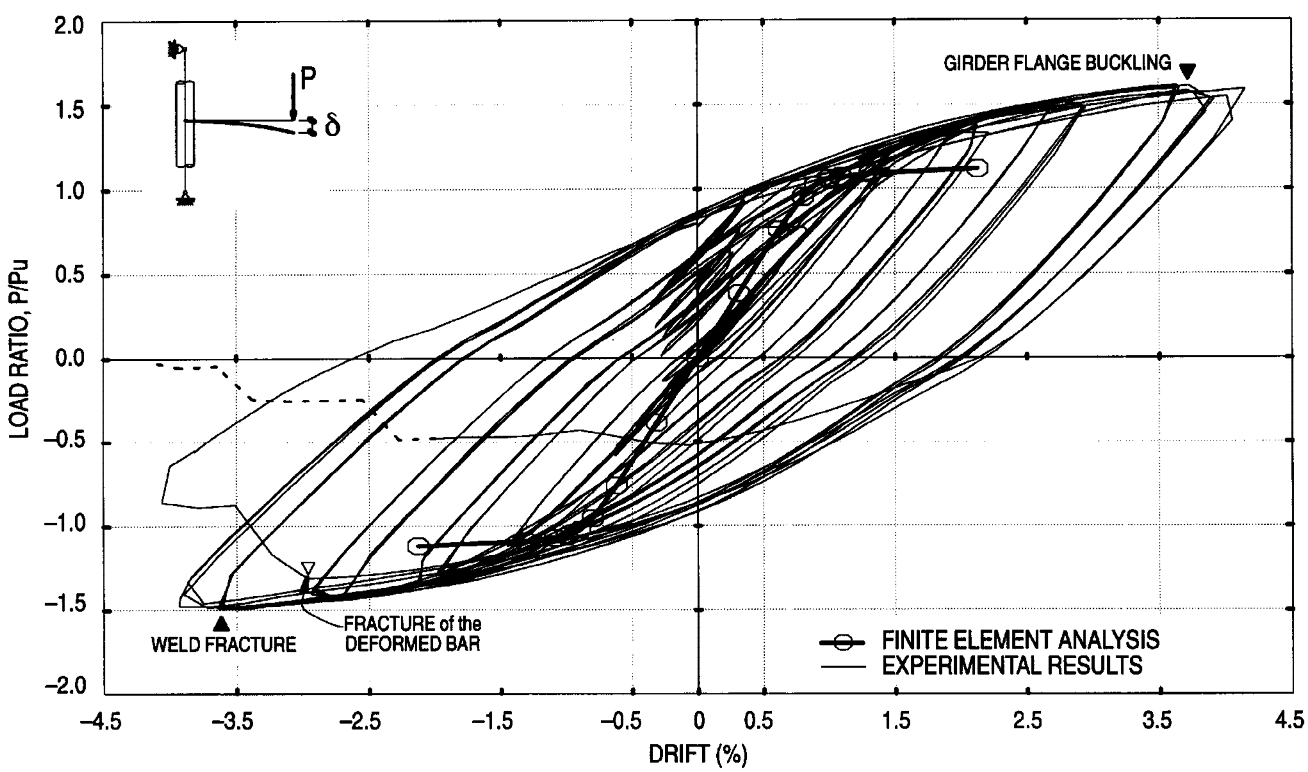


Fig. 9. Load–Deflection Behavior: Connection Type III.

Connection *Type III* was identical to connection *Type I*, however four 25mm $\phi$ ($F_y$ = 420 MPa) weldable deformed bars were attached to each flange and were embedded in the concrete core. Embedment lengths were sufficient to develop the deformed bar under test conditions. Deformed bars were placed into holes drilled in the steel tube wall, and no attachment was made between the wall and the bar. Welds that attached each bar to the girder flange was approximately 50% more weld than was needed to develop the bar strength in tension. The hysteretic behavior of this detail, as illustrated in Fig. 9, showed significant improvement compared to connection *Type I*, and the increase in the fabrication cost was only 25%. Initial wall tearing was observed at a drift of approximately 3.7%, however this tearing was located only in the tube wall between the openings for the deformed bars. This minor wall tearing did not affect the inelastic performance of this connection. Local flange buckling was also observed at a drift of approximately 3.7%, and this buckling occurred beyond the ends of the deformed bars. This suggests that the connection was strong enough to initiate a plastic hinge in the girder. Eventually, failure of this connection was due to fracture of the deformed bars. Three of the four bars failed by tension rupture, while one bar pulled out of the concrete core. No significant stiffness or strength deterioration 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. This connection exhibited stable behavior up to drifts of approximately 3.5%.

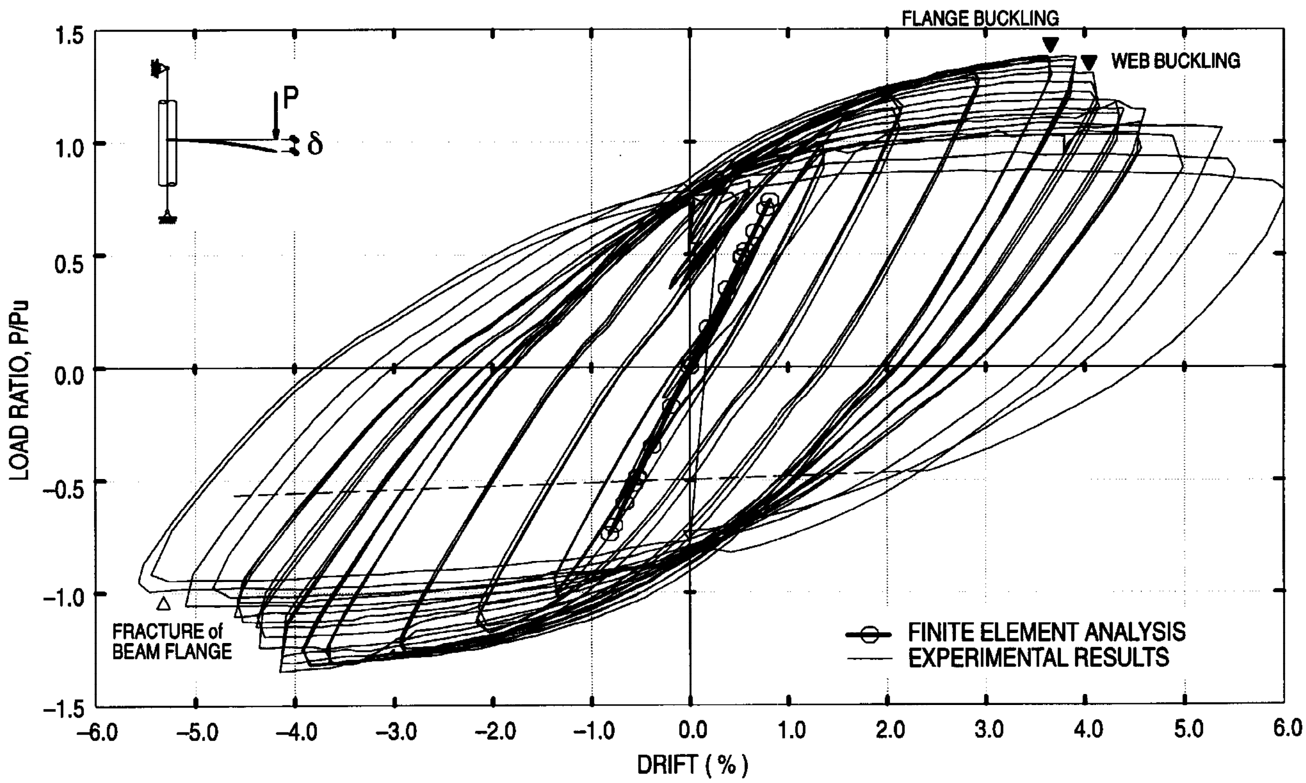


Fig. 10. Load–Deflection Behavior: Connection Type V.

Results for the connection *Type V*, shown in Fig. 10, exhibited quite stable inelastic behavior compared to the previous details. Local flange buckling was observed at approximately 3.7% drift, and web buckling was observed at about 4% drift. Deterioration of the inelastic characteristic were observed after the onset of the local flange and web buckling. This connection type sustained several cycles of deformation at large drift amplitudes. This connection was clearly able to develop the plastic bending strength of the girder. Ultimate failure of this connection was due to fracture of the beam flange in the connection stub region. This fracture was relatively abrupt, and eventually propagated into the web. No significant post–yield stiffness deterioration was noted, however the strength of the connection deteriorated about 30% of the maximum under the extreme drift cycles. 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 apparent, and the tube wall did not display outward signs of being overstressed. The fabrication cost of connection *Type V* was about 50% higher than that for connection *Type I*.

## CONCLUDING REMARKS

Connection performance improves significantly when a larger share of the connection load is distributed to the concrete core. The inelastic performance depends significantly on the connection detail. Some conclusions can be inferred from these results:

1. While the *Type I* connection appears convenient to detail, and easy to construct, it should not be used in lateral–load resisting frames subject to moderate or high seismic excitations. The connection cannot develop the plastic bending

strength, the flange weld is susceptible to fracture, the tube wall is susceptible to tearing, and the connection may lose shear capacity under extreme seismic conditions.

2. Analytically, the diaphragm plates in connection *Type II* offer some improvement to the strength and stiffness of connection *Type I*. If designed properly for the shear flow, this connection may alleviate some of the tube wall tearing observed in connection *Type I* behavior, and may prolong the shear capacity of the connection.

3. Weldable deformed bars transferred much of the flange stresses into the concrete core. This connection showed considerable improvement over the simple connection *Type I*, and could be used in seismic regions. However, proper installation of the deformed bars, if welded at the construction site, becomes a critical issue.

4. Analytical results for connection *Type IV* showed satisfactory strength and elastic stiffness characteristics, however the contribution of the shear studs to the strength of the connection is limited. For example, increasing the number of studs does not necessarily correspond to a proportional increase in flexural strength for the connection. Further, the construction of this detail may be too labor intensive for many applications.

5. The connection in which the girder continued through the CFT exhibited favorable hysteretic behavior compared to the other connection types tested. This appeared to be the most effective method to develop the plastic bending strength of the steel girder. However, this detail was also the most expensive connection fabricated for testing, and may offer difficulties during construction.

## ACKNOWLEDGMENTS

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