Cyclic Performance of Wide Flange Beam to Concrete-Filled Rectangular Tube Column Joints with Stiffening Plates around the Column

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Abstract
The strength and rigidity of wide flange beam to rectangular tube column joints are reduced significantly if the connections are not reinforced. However, the reinforcement for the connections can be complex and increase the fabrication costs significantly because the tube columns are closed sections. This paper describes the force transfer mechanism and the cyclic performance of wide flange beam to concrete-filled rectangular tube column joints reinforced with stiffening plates. The first phase of this research program was to assess the force transfer mechanism at the joint using an analytical yield line method. An experimental program was conducted to verify the proposed model. The test results showed that the derived nominal strength equation provided a reasonable prediction. The second phase of the research program was to assess the cyclic performance of the joints by full-scale joint subassemblage tests. A total of five specimens were tested under cyclic loading. Test results showed that plastic rotation of 0.02 radian which is required for Intermediate Moment Resisting Frames could be obtained.

Keywords: concrete-filled tube; connection; cyclic performance; force transfer mechanism

Introduction
Rectangular tubes have been popular as compression members because of efficient cross sectional use in compression and torsion. Filling a tube with concrete increases the strength and stiffness of the section by inhibiting its local buckling, and also improves its fire resistance. A five-year research on concrete-filled tubular column systems has been conducted as a part of the U.S.- Japan Cooperative Earthquake Research Program (Morino 1998, Sakino 1998).

Since tube sections are of closed shape, the reinforcement at the connections for tubular sections is complex and increases labor costs significantly. Therefore, a variety of simple frame connections have been developed to connect wide flange beams to tube columns (Sherman 1996).

Research has been carried out on different types of moment connections of wide flange beam to rectangular tube column joints (Shanmugam and Ting 1995, Ricles and Peng 1988, Kato et al. 1981). Practical applications in construction have been found in Japan. The most widely used connection type is running of continuity plates (termed diaphragms in Japan) through the tube as shown in Figure 1(a). The use of through-plates increases the strength and stiffness of the connection significantly. Nevertheless, the column has to be cut and welded at the locations of the beam flanges for the through-plates.

In this study, the behavior of beam to column joints with stiffening plates around the tube column as shown in Figure 1(b) was investigated.
If the thickness of the stiffening plates is increased significantly compared to that of the beam flange, then the use of stiffening plates can increase the strength and stiffness of the connection close to those of the through-plates connection type (Yoshisato et al. 1995). Since the stiffening plates are fitted around the column, the tube column does not need to be cut. Moreover, when the tube is used as a concrete-filled column, concrete can be filled more easily compared to the through-plates connection type.

In the first phase of this study, force transfer mechanism of the joint with stiffening plates around the tube was assessed and a prediction of the nominal strength was proposed. An experimental program was conducted to confirm that the proposed nominal strength equation provides reasonable prediction. In the second phase, full-scale joint subassemblage tests were performed to investigate the cyclic performance of the joint.

**Analytical Model**

Concrete filling increases the strength at the connection where a compression force is transferred from the beam flange. When pulled out by the beam flange in tension, the face of the tube column comes out of its plane. Therefore, the strength of the joint is usually governed by the strength at the connection where a tension force is transferred from the beam flange.

Therefore, a simplified analytical model of beam flange plates in tension connected to a rectangular tube column was used to investigate the ultimate strength of the joint. In this study, three types of simplified analytical models were developed as shown in Figure 2 depending on the location of the beam flange. Model type C represents the case where the centerline of the beam flange passes through the tube core. Model types E and S represent the cases where the end line of the beam flange aligns with the side wall of the tube and the end line of the side stiffening plate, respectively.

Morita developed an analytical model to predict the nominal strength of the wide flange beam to rectangular tube column joints without diaphragms (Morita 1994). In this study, a similar approach was used to derive nominal strength equations of the simplified analytical models (Park et al. 1998a). In Figure 2, it is assumed that the tube has failed as a mechanism with plastic hinges forming along the yield lines and the stiffening plate has failed in shear yielding. At ultimate load P, the external work \( W_E \) is done by P through the virtual displacement \( \delta \).

The internal work \( W_I \) is obtained as the summation of the work done by the tube along the yield lines and the work done by the stiffening plate through the virtual shear deformation.

From the principal of virtual work \( (W_E = W_I) \), the nominal strength \( P \) of model type C can be calculated as

\[
P = 4M_{pc} \left( \frac{B_c}{x} + \frac{2x}{m} + \frac{t_s}{m} \right) + \frac{2F_{uw}t_s h_s}{\sqrt{3}}
\]

where the geometrical parameters are shown in Figure 2 and

\[
x = \sqrt{\frac{B_c m}{2}} \quad \text{(obtained as the value to minimize P)}
\]

\[
M_{pc} = \frac{F_{ye} t_c^2}{4}
\]

\( B_c \) = width of the tube column

\( t_c \) = thickness of the tube column

\( B_f \) = width of the beam flange

\( t_f \) = thickness of the beam flange

\( h_s \) = width of the stiffening plate
Similarly, the nominal strength for model type E is derived as

\[
P = \frac{2M_{pe}(2x + t_s)}{m} + \frac{4M_{pe}B_c}{x} + \frac{2F_{us}t_xh_x}{\sqrt{3}} + \frac{F_{ys}t_yy}{2}
\]

and the nominal strength for model type S model is given by

\[
P = \frac{2M_{pe}(2x + t_s)}{m} + \frac{4M_{pe}B_c}{x} + \frac{2F_{us}t_xh_x}{\sqrt{3}} + \frac{F_{ys}t_yy}{2} + \left( h_x + \frac{t_c}{2} \right) F_{ys}t_f
\]

where

\[
x = \sqrt{\frac{ymB_c}{y + m}}
\]

\[
y = 1.6 \times B_{c}^{0.3} \times \left( \frac{t_c^2F_{pc}}{t_fF_{ys}} \right)^{0.6}
\]

F_{ys} = yield stress of the beam flange

In a wide flange beam to rectangular tube column joint, the ratio of the bending moment carried by the beam web is less than 5% even for very thick tube thickness, and most of the beam moment is transferred primarily through the beam flanges (Tsai 1992). Therefore, the nominal flexural strength of wide flange beam to rectangular tube column joints can be obtained by multiplying the nominal strength \( P \) of the simplified model by the distance \( d \) between the mid-planes of top and bottom flanges of the wide flange beam

\[
m = P \times d
\]

Experimental Results

To verify the applicability of the yield line theory, tests were performed on 35 specimens of simplified models of Fig. 2 (Park et al. 1998a). The specimens were placed in a universal testing machine and loaded to produce tensile force in the connection. When the strength of the connection calculated using the Equation (1), (2), or (3) was smaller than the maximum strength of the flange plate \( P_{uf} = F_{uf}B_{ft} \) (\( F_{uf} \) = tensile strength of flange plate), the ultimate strength was reached as a crack occurred at the connection of the flange plate and the stiffening plate. The crack started from the end of the weld due to the stress concentration and propagated into the stiffener as shown in Figure 3(a). When the calculated strength of the connection was greater than the maximum strength of the flange plate, the ultimate strength was reached as the necking down of the flange plate occurred as shown in Figure 3(b).

The values of theoretical strength based on the yield line theory were compared with test results. The mean
and standard deviation of the ratios of test strength to theoretical strength were 1.054 and 0.139, respectively.

In-plane bending moment tests were conducted on 10 specimens consisting of a rectangular tube column and wide flange beams to investigate the ultimate flexural strength of the joints (Park et al. 1998b). When the flexural strength of the joint calculated by Equation (4) was smaller than the plastic moment of the beam, the ultimate strength was reached as a crack occurred at the connection of the beam tension flange and the stiffening plate as shown in Figure 4(a). The crack started from the end of the weld and propagated into the stiffening plate. When the calculated flexural strength of the joint was greater than the beam plastic moment, the ultimate strength of the test specimens was governed by the local buckling of the compression flange of the beam as shown in Figure 4(b).

The mean and standard deviation of the ratios of test strength to theoretical strength for the bending moment tests were 1.127 and 0.142, respectively. Relatively high mean value was attributed to the strain hardening effect of the beam section, which had a moment gradient.

### Cyclic Performance

#### Test Program

The 1994 Northridge earthquake caused widespread damages to steel moment frame connections (SAC 1995). In the 1995 Kobe earthquake, many beam to rectangular tube column connections suffered severe damages (AIJ 1995). As shown in Figure 2, the stiffening plate type connection can develop inelastic deformation through the shear yielding of the stiffening plate. This is believed to increase the seismic performance of the joint by reducing the plastic deformation demand at the beam flange to stiffener connection, where premature fracture due to potential weld defects and stress concentration may contribute to a brittle failure.

To investigate the behavior of the joints under cyclic loading, full-scale subassemblage tests were conducted as shown in Figure 5. The testing protocol used in this project was that of ATC-24 (ATC-24 1992). In this test, the reference yield displacement of the loading history was determined from the load-displacement curves of the specimens obtained using a nonlinear finite element package – ANSYS (ANSYS 1992). No axial load was applied to the column. A total of five
specimens of model type C were tested. Displacement rates lower than 0.5 mm/sec were used to avoid dynamic effects.

Fig.5. Cyclic Test Set-up

All beam sections were H600x200x11x17 and all tube column sections were 400x400x10. The test specimen sizes are typical of low to medium rise steel buildings in zones of moderate seismicity. The shear strength of the column panel zone based on the Architectural Institute of Japan provisions for composite construction was significantly greater than the beam plastic moment due to the filled concrete (AIJ 1987). Material properties are shown in Table 1.

Table 1. Mechanical Properties of Materials

| Thickness (mm) | $F_y$ (MPa) | $F_u$ (MPa) | Elongation (%) |
|----------------|-------------|-------------|----------------|
| Column         | 10          | 287.1       | 428.3          | 35.7           |
| Beam flange    | 17          | 303.8       | 444.9          | 37.5           |
| Beam web       | 11          | 337.1       | 491.0          | 37.5           |
| Stiffener      | 30          | 248.9       | 445.9          | 35.3           |

Specimens CN, CM, CW, and CW-E had the all welded connection detail as shown in Figure 6(a). Specimen CW-B had the same dimensions as Specimen CW but different web connection detail. Specimen CW-B had the welded flange-bolted web connection detail as shown in Figure 6(b). Specimens CN, CM, and CW had the same connection detail, but the width of the stiffening plates was varied. The ratio of the joint strength calculated using Equation (4) to the plastic moment of the beam was 0.74, 1.05, and 1.32 for Specimens CN, CM, and CW, respectively. The tube columns of Specimens CN, CM, CW, and CW-B were filled with concrete whose cylinder compression strength was 32 MPa. Specimen CW-E was the same as Specimen CW except that the tube column was not filled with concrete. The test specimens are summarized in Table 2.

Beam flanges were welded to stiffening plates using complete joint penetration groove welds. Backing bars were used and left in place throughout the test. Stiffening plates and beam webs were welded to the tube column using fillet welds. All specimens were fabricated using the flux cored arc welding (FCAW) process with E70T-4 electrodes except the beam flange to stiffening plate welding of specimen CW-B, which was made using the shielded metal arc welding (SMAW) process with E7018 electrodes to simulate the field welding.
Table 2. Cyclic Test Results

| Spec. | Stiffening plate (mm) | Beam plastic moment (kN-m) | Calculated strength of joint (kN-m) | Observed maximum strength (kN-m) | Test/ Theory | Total plastic rotation (rad) | Failure Mode |
|-------|-----------------------|---------------------------|-----------------------------------|---------------------------------|--------------|-----------------------------|--------------|
|       | h, t                  |                           |                                   |                                 |              |                             |              |
| CN    | 60 30                 | 869.8                     | 641.5                             | 647.1                           | 1.01         | 0.025                       | Brittle cracking through stiffening plate |
| CM    | 90 30                 | 869.8                     | 911.7                             | 808.6                           | 0.93         | 0.019                       | Brittle cracking through stiffening plate |
| CW    | 120 30                | 869.8                     | 1181.9                            | 947.7                           | 1.09         | 0.023                       | Brittle cracking along weld line            |
| CW-E  | 120 30                | 869.8                     | 1181.9                            | 911.6                           | 1.05         | 0.020                       | Brittle cracking along weld line            |
| CW-B  | 120 30                | 869.8                     | 1181.9                            | 979.8                           | 1.13         | 0.017                       | Brittle cracking through beam flange        |

* Smaller value of beam plastic moment and calculated joint strength

All the welds were made by a certified welder and passed the ultrasonic test.

It is believed that E70T-4 and E7018 electrodes do not necessarily provide metal of adequate toughness. SAC requires the use of weld metal providing CVN toughness of 20ft-lbs at -20°F and the removal of the backing bars and reinforcing fillet weld at the girder bottom flange to prevent notch effect at the backing bars (SAC 2000). However, there is no evidence to suggest that superior welding quality and removal of backup bars are enough to ensure good cyclic performance. In Korea, change in weld materials and removal of backup bars are not required even after the 1995 Kobe Earthquake from the expectation that satisfactory cyclic performance can be achieved by the modification of the connection details.

Test Results

All the specimens failed in a brittle manner. In all specimens, fracture initiated at the corner of the beam flange to stiffening plate connection where stress concentration occurred. Crack propagated through the stiffening plate to the column face for Specimens CN, CM as shown in Figure 7(a). However, crack propagated along the weld line or through the beam flange in the heat-affected zone as shown in Figure 7(b) for Specimens CW, CW-E, and CW-B, whose joint strength by Equation (4) was significantly greater than the beam plastic moment. For Specimen CW-E, which was not filled with concrete, slight panel zone yielding was revealed by the white-wash spalling, but the level of plastification was low.

For all the specimens, most of the plastification was observed in the stiffening plate, and some in the beam flange as shown in Figure 8. No local buckling of the beam flange was observed. The observed maximum strength of each specimen prior to the failure and the failure mode are listed in Table 2.

Specimens CM, whose joint strength by Equation (4) was slightly greater than the beam plastic moment, developed 93% of the beam plastic moment. Specimens CW, CW-E, and CW-B, whose joint strength by Equation (4) was much greater than the beam plastic moment, were supposed to fail by the local buckling of the compression. However, they failed by sudden fracture at the connection before the full plasticity was developed within the beam section. This was attributed to the stress concentration at the corner of the beam flange to stiffening plate connection.

The load versus total displacement response is shown in Figure 9. The total plastic work represented by cumulative hysteretic area is shown in Figure 10. Beam plastic rotations achieved by the specimens are
listed in table 2. The plastic rotation was calculated by dividing the plastic displacement at the beam end by the distance between the beam end and column face. Test specimens could not achieve plastic rotation of 0.03 radian, which is required for Special Moment Resisting Frames (AISC 1997). Specimens CW and CW-E achieved plastic rotation greater than 0.02 radian, which is required for Intermediate Moment Resisting Frames. Specimen CW-B, which had the bolted web connection detail, could not achieve the plastic rotation of 0.02 radian.

Specimens CN could achieve the largest plastic rotation. However, the total plastic work was relatively small because of the low connection strength. Improved performance by concrete filling can be observed when comparing Specimens CW and CW-E. A possible explanation is that concrete filling increased the shear strength and stiffness of the panel zone, reducing the stress concentration at the beam flange to stiffening plate connection. Comparison of specimens CW and CW-B shows that all welded connection provides a more improved performance than welded flange-bolted web connection.

Summary and Conclusions
An analytical yield line method was developed to predict the nominal strength of steel wide flange beam to tube column joints with stiffening plates around the column, and an experimental program was conducted.

Fig.8. Yielding of Stiffening Plate and Flange (CW)

Fig.9. Load versus Total Displacement Curves
(a) Specimen (CN)
(b) Specimen (CM)
(c) Specimen (CW)
(d) Specimen (CW-E)
(e) Specimen (CW-B)

Fig.10. Total Plastic Work
to verify the analytical model and the proposed strength equation. Test results showed that the proposed strength equation gives satisfactory prediction.

To investigate the cyclic performance of the joints, five full scale subassemblage tests were conducted under cyclic loading. Based on the test results, the following conclusions are made:

- Steel wide flange beam to tube column joints with stiffening plates can achieve plastic rotation greater than 0.02 radian mainly through the plastification of the stiffening plates.
- The decrease in the width of the stiffening plate can increase the rotation capacity, but reduce the total plastic work due to low connection strength.
- Concrete filling of the tube column improves the cyclic performance.
- All welded connection detail shows better performance than the welded flange-bolted web connection detail.

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