CHAPTER 5

PANEL ZONE BEHAVIOUR

5.1 GENERAL

It is well known that the beam-to-column joint in frames is constrained by other members, and the behaviour of this region may have an influence on the nonlinear seismic performance of the whole structure. The behaviour of the panel zone is the key issue on the behaviour of the joint. The panel zone is subjected to very high shear forces. If the joint panel is not capable of transferring such forces, it may fail due to joint shear failure.

In this chapter the joint shear versus deformation for the composite joint consisting of steel beam and concrete filled steel tubular (CFT) column (circular and square section) is investigated. Set of equations for calculating the shear deformation and shear strength capacity are presented.

5.2 PREVIOUS STUDIES

A number of experimental and computational studies were conducted for a better understanding of the role of panel zone shear deformation in seismic connection performance.

The analytical shear strength capacity of the panel zone is obtained by the superposition of relations of the steel tube and core concrete parts. Cheng and Chung (2003) suggested that the shear relation for the steel tube could be expressed as a similar tri-linear relation, and for the core concrete a
nonlinear relation could be applied. Fukumoto and Morita (2005) gave shear versus shear deformation relation of the panel zone for circular and square CFST (concrete filled steel tube) joints, where tri-linear relations were applied for both steel tube and core concrete.

Recent research works revealed that the shear capacity of the panel zone is contributed by the “compressive strut” mechanism of the inner core concrete and the shear mechanism of the outer steel tube (Li and Han 2011), as shown in Figure 5.1.

In the current research, the nominal shear strength at the panel zone was calculated considering the contribution of each individual part of steel and concrete core based on their material strength. The calculated nominal shear strength of the panel region is compared with the measured one.

Figure 5.1 Schematic view of CFST panel zone mechanism, Li and Han (2011)
5.3 EXPERIMENTAL MEASUREMENTS FOR SHEAR FORCE AND DEFORMATION AT THE PANEL ZONE

In the CFT column-to-beam moment connection, the flange forces from the beam produce a large shear in the panel zone. If the joint is unable to resist such shear, yielding will occur in the panel region.

To monitor the overall joint shear deformation in an average sense, two LVDTs were installed at the face of the joint in each specimen in “×” shape (Figure 5.2). Considering the two triangles formed by the LVDTs, the angular changes as shown in Figure 5.2 were measured for each step. Then, the joint shear deformation was computed as an average of the two angular changes at the face of the panel.
where $\gamma_j$ (radian) represents the joint shear deformations; $b$ and $h$ are the width and height of the panel zone area between the two LVDT anchor points respectively, and, $\Delta_1$ and $\Delta_2$ are the displacements measured using LVDTs positioned diagonally over the panel.

On the other hand the joint shear strength is evaluated, by studying the equilibrium of the horizontal forces on a horizontal plane at the mid height of the joint, as shown in Figure 5.3. Assuming that the beam bending moment is carried entirely by the flanges, the tensile and compressive forces in the beam flange, $T_f$ and $C_f$, are estimated as:

$$T_f = C_f = \frac{M_b}{j_d} \tag{5.2}$$

where $M_b$ is the beam moment at the joint face; and $j_d$ is the internal lever arm for calculating the moment $(j_d=d-t_f)$.

The effective horizontal shear force acting on the joint panel, $V_{jh}$ is calculated using Eq.(5.3):

$$V_{jh} = (T_f + C_f) - V_c \tag{5.3}$$

where $V_c$ is the column shear force and is estimated from the beam moments at the joint face using Eq.(5.4):

$$V_c(h) = V_{bs}\left[\frac{L}{2}\right] + V_{bsN}\left[\frac{L}{2}\right] \tag{5.4}$$

The joint shear is expressed as follows:

$$V_{jh} = \left[\left(\frac{M_{bs}}{j_d}\right) + \left(\frac{M_{bsN}}{j_d}\right)\right] - [V_{bs} + V_{bsN}]\left(\frac{L}{2. h}\right) \tag{5.5}$$
Figure 5.3  **Free body diagrams of (a) a typical interior beam-column joint test setup and (b) its joint panel**

\(M_{bN}\) and \(M_{bS}\) are the beam moments at the joint face, and are equal to \(V_{bs} \times [(L/2) - (b_j/2)]\) and \(V_{bs} \times [(L/2) - (b_j/2)]\) respectively.

Substituting Eq. (5.5) in Eq. (5.4), the joint shear force is expressed as given in Eq. (5.6):

\[
V_{jN} = \left[ V_{bs} + V_{bN} \right] \left( \frac{L - b_j}{2J_d} - \frac{L}{2h} \right) \quad (5.6)
\]

Where, \(L\) is the total length of the beam (includes south and north beam), \(b_j\) is the width of the joint panel, and \(h\) is the total length of the top and bottom columns; (superscripts S and N refer to the South and North directions, respectively).
5.4 EVALUATION OF NOMINAL SHEAR CAPACITY AT THE PANEL ZONE

The horizontal shear force in the joint is resisted by the shear in the steel tube $V_{tn}$, and the shear in the concrete panel zone, $V_{cn}$. For the end-plate type connection, the nominal shear force is calculated using Eq.(5.7a), while in the through beam type connection, the joint nominal shear force capacity is calculated, considering the shear in the steel web and flange, $V_{wfn}$ using Eq.(5.7b), (Azizinamini et al 2004):

$$V_n = V_{tn} + V_{cn} \quad \text{for end-plate type connection} \quad (5.7a)$$

$$V_n = V_{tn} + V_{cn} + V_{wfn} \quad \text{for through beam type connection} \quad (5.7b)$$

The shear capacity of the steel tube ($V_{tn}$) is calculated using Eq. (5.8) (Krawinkler 1978) for the steel tube. For the concrete core the shear capacity ($V_{cn}$) is calculated based on ACI-ASCE 352 (1985) recommendations for reinforced concrete joints confined on all four vertical faces of the joint; since the joint in a CFT column is confined by the tube wall, it is reasonable to consider the same value as that recommended for confined joints and is presented in Eq. (5.9).

$$V_{tn} = A_{sh} \left( \frac{F_{yt}}{\sqrt{3}} \right) \quad (5.8)$$

$$V_{cn} = 1.99 \sqrt{f'_{c}} \times A_{shc} \quad (5.9)$$

where $F_{yt}$ and $f'_{c}$ are the yield strength of the steel tube and concrete compressive strength respectively, while $A_{shc}$ and $A_{sh}$ are the horizontal effective shear area for the concrete core and the steel tube respectively. The effective shear area of the circular tube is given by $\pi d_{c} t_{c} / 2$, (Boresi et al 1993), and that of the rectangular tube is given by $2(d_{c} - 2t_{f}) \times t_{w}$, (Wu et al 2005).
The experimental and analytical results of the steel beam connections to the reinforced concrete columns conducted by Sheikh (1987) indicated that the shear stress varied between $1.99 \sqrt{f'_c}$ and $2.99 \sqrt{f'_c}$ (in MPa), and the panel deforms as a monolithic unit. Thus, in Eq. (5.9) it is assumed that the entire concrete core area is effective in resisting the joint shear.

The shear strength at the web panel, $V_{wn}$ in Eq. (5.10) is provided by means of the shear yielding in the web, $V_{wn}$ and the flexural rigidity of the flanges at the connection panel joint boundaries, $V_{fn}$.

Depending on the shear yield stress of $0.6F_{yw}$ of the Von Mises yield criteria, and as per the (AISC) LRFD specifications (1994), the web shear yield is calculated using Eq. (5.11), based on an average yield shear stress of $0.6F_{yw}$ acting over the horizontal web area within the joint panel.

The shear resistance of the flanges is calculated using Eq. (5.12), (Sheikh 1987 and Deirlein 1988). The web panel shear strength is given by the following equations:

\[
V_{wn} = V_{wn} + V_{fn}
\]  (5.10)

\[
V_{wn} = 0.6 F_{yw} b_j t_w
\]  (5.11)

\[
V_{fn} = \frac{4M_{pf}}{d_b}
\]  (5.12)

\[
M_{pf} = \frac{F_{yf} t_f^2 b_f}{4.0}
\]  (5.13)

where $F_{yw}$ and $F_{yf}$ are the yield stress of the beam web and flange respectively, and $t_w$ and $t_f$ are the beam web and flange thicknesses, while $b_j$ and $d_b$ are the width of the panel joint and the depth of the beam respectively.
5.5 COMPARISON OF CALCULATED AND MEASURED SHEAR CAPACITY OF PANEL ZONE

The joint shear reaches its capacity, when all the contributing mechanisms have reached their individual shear strengths. In this study, the calculated nominal shear capacity of the joint for all the specimens are compared with the corresponding measured shear strengths, to predict the panel zone behaviour as shown in Figures 5.4 to 5.7.

For Specimen No.1, with end-plate type connection the maximum shear capacity at the panel zone during testing ($V_{jh}$) was found to be 1034kN and 1057kN in the positive and negative loading directions respectively, while the calculated nominal shear capacity ($V_n$) was 945.5kN. Figure 5.4 shows the hysteretic curves of panel shear force versus shear strain for Specimen No.1. This figure clearly demonstrates that the panel zone was subjected to high shear stresses during testing and the magnitude of maximum shear force was close to the calculated nominal shear capacity of the joint.

Similar behaviour was noticed for Specimen No.2 (Figure 5.5), with the maximum shear force of 1077kN and 1002kN, in both directions of loading, while the computed nominal shear capacity ($V_n$) was 874kN. However, in both cases the panel zone was still elastic and far from joint shear failure or panel mechanism. This is consistent with the general observation of no inelastic deformation in the panel zone region.

For through beam type connections (Specimens No.3 and No.4), Figures 5.6 and 5.7 show the hysteretic curves of shear deformation at the panel zone. The shear force at the panel zone during testing is less than the calculated nominal shear capacity, due to the beam failure mechanism.
Figure 5.4  Hysteretic curve of shear deformation at the panel zone for Specimen No.1

Figure 5.5  Hysteretic curve of shear deformation at the panel zone for Specimen No.2
In Specimens No.3 (Figure 5.6), the maximum shear force ($V_{jh}$) was 910kN and 950kN in positive and negative directions of loading respectively, and the calculated nominal shear capacity ($V_n$) was 1229kN.

For Specimens No.4 (Figure 5.7), the maximum shear force ($V_{jh}$) during testing was 950kN and 893kN in the positive and negative directions of loading respectively, and the calculated nominal shear capacity ($V_n$) was 1066kN.

In this type of connection (through beam type) also, no inelastic deformations were observed, indicating that these equations used for calculating the nominal shear capacity for panel zone are conservative.

![Hysteretic curve of shear deformation at the panel zone for Specimen No.3](image)

**Figure 5.6** Hysteretic curve of shear deformation at the panel zone for Specimen No.3
SUMMARY

In this chapter, the joint shear versus deformation for the composite joint consisting of steel beam and concrete filled steel tubular (CFT) column (circular and square section) was investigated. The nominal shear strength at the panel zone was derived from the contribution of each individual part of the steel and concrete core based on their strength. The behaviour of the panel region is examined, by comparing the calculated and the measured shear capacities of the panel zone and it is indicated that the equations used in this study for calculating the panel zone capacity are conservative.

It is worth mentioning here that, the mechanism of the bolted beam-to-column connection failure is influenced by the relative strength of the
beam, the column and the panel zone. If the panel zone is relatively weak, the energy dissipation concentrates on the panel zone and the plastic hinge cannot be generated in the beams to dissipate energy. With pre-stress applied to the rods at the beam end-plate, a confining force is generated between the column tube and the surrounding concrete and it helps to eliminate the outward buckling experienced by the column flange resulting in the better seismic resistance of the bolted connection.