CHS Collar Joints

Author: *Kevin Cowie*
Affiliation: *Steel Construction New Zealand Inc.*
Date: *26th August 2009*
Ref.: *CON1002*

Key Words
Collar joints, moment connections, flange bolted joint

Introduction

This article presents a method for designing the collar joints of a moment connection between an I section beam and a circular hollow section (CHS) column. The procedure is developed principally for the semi-rigid flange bolted joint (FBJ). However the principles can be applied in general to other types of connections. The design procedure has been taken and slightly modified from (Clifton, 2005).

Design Procedure

Figure 1 shows the concept of apply the FBJ to a CHS column, involving the top and bottom flange plates extending around the sides of the column as external transfer plates. These must transfer horizontal actions from the flanges and the web plate into the column.

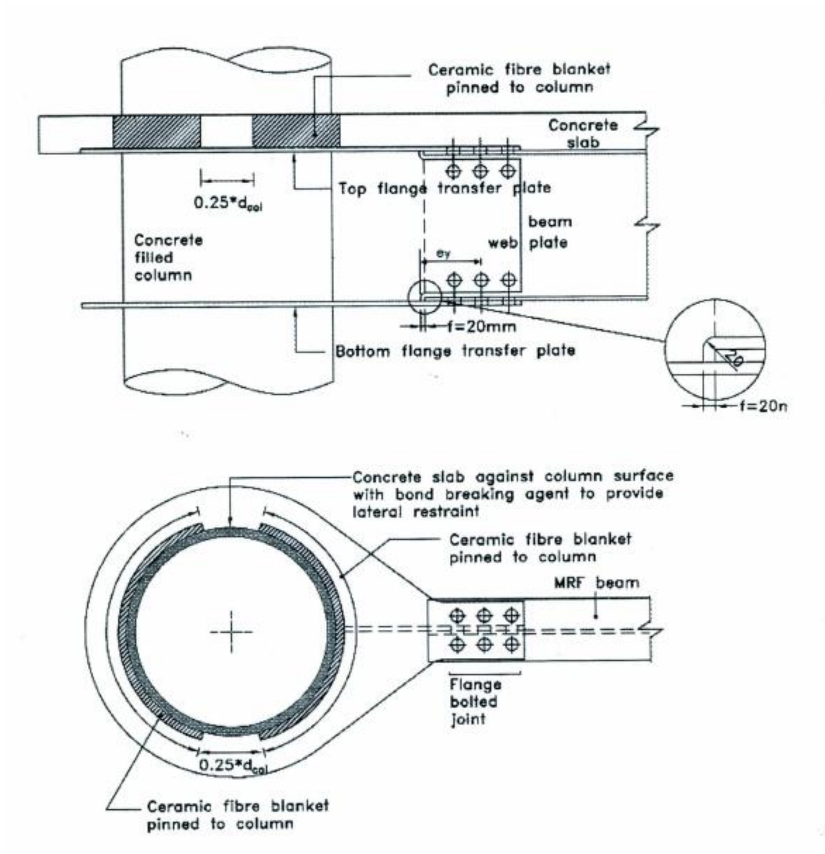

Figure 1: CHS Collar Detail with FBJ Connection (Clifton, 2005)

Figure 2 shows a FBJ onto a CHS as part of a one-way frame. The critical locations for the design checks have been identified.

One reason why a CHS column would be used in preference to an I section column is for a two way frame. Figure 3 shows such a detail, including the critical locations for the design checks.

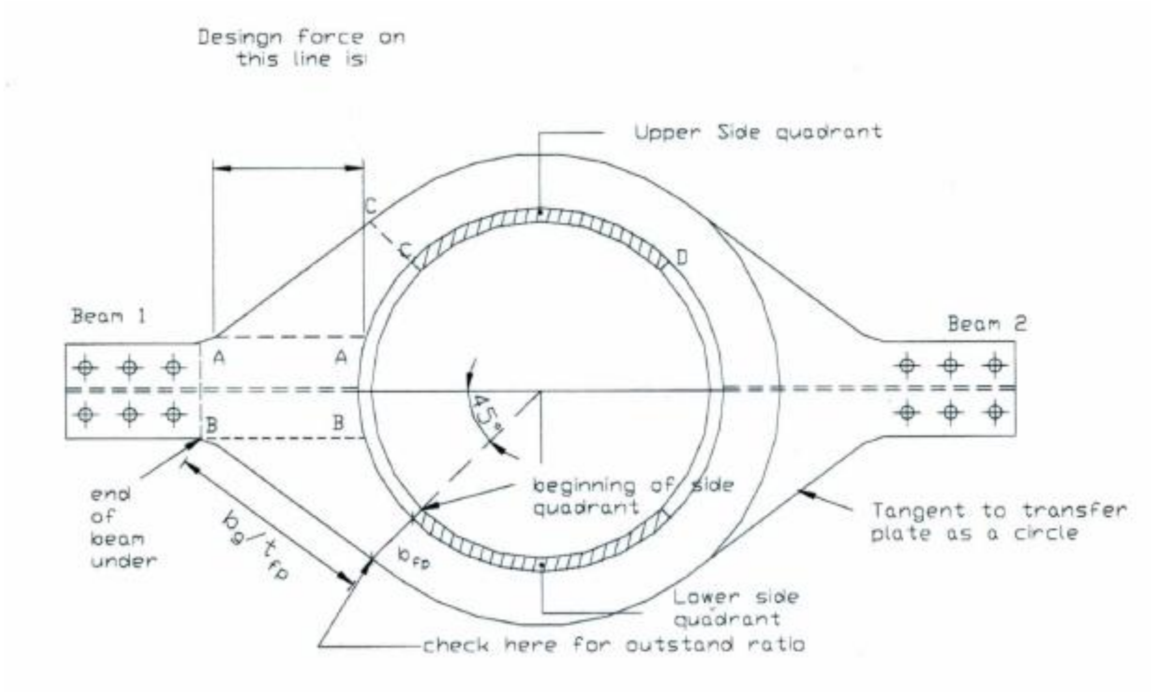

Figure 2: FBJ to CHS One Way Frame (Clifton, 2005)

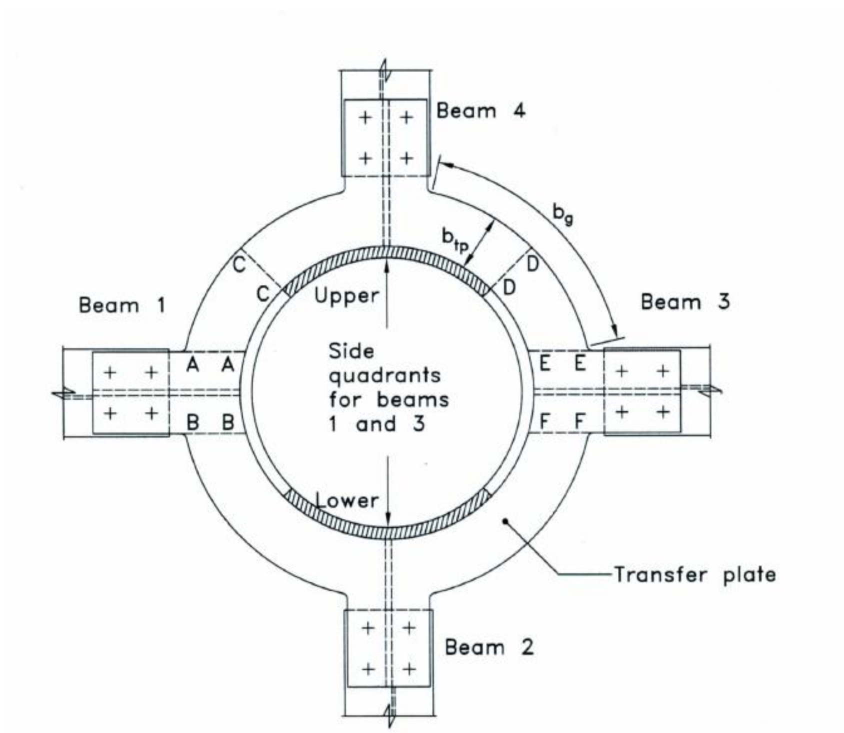

Figure 3: Two-way frame detail (Clifton, 2005)

The design concepts / critical checks are as follows:

1. The top and bottom flange transfer plates must transfer the horizontal forces from the connection/beam into the CHS
    a. Horizontal actions from the connection flange plates / beam flange
        i. For a semi-rigid FBJ the flange plate is designed to yield and therefore the horizontal actions is equal to the design capacity of the flange plate.
    b. Plus horizontal actions from the moment in the connection web plate / web beam
        i. For a semi-rigid FBJ the horizontal actions is equal to design moment capacity of the web plate divided by the beam depth
    c. 82% of these horizontal actions must be transferred into the side quadrants of the CHS. The remaining 18% of the force is transferred into the front and back quadrants in equal measure. This determines the sizing of the transfer plates in that the transfer plates must be able to transfer this force around from the end of the connection and into the side walls.
    d. The first check is for shear yielding. For beam 1 in figure 2, this check is done along lines AA and BB.
        i. The design shear action is equal to (82+9)% of half the horizontal action determined from above.
        ii. The design shear strength is equal to $\Phi x0.83x0.6xf_y = 0.45f_y$
        iii. This check must be done for each beam separately. For beam 3 of figure 3 the check is along lines EE and FF
    e. The second check is at the beginning of the side quadrant, shown as line CC in figures 2 and 3.
        i. This is taken at the minimum plate dimension between the beam and side quadrant
        ii. At this point the transfer plate must be able to transfer $((0.82+0.09)/2)x\sqrt{2}$ of the horizontal force from beam 1 into the side quadrant CD.
        iii. For the two way frame as shown in figure 3 concurrency effects must be considered. When direction of moment in beam 1 and beam 4 is the same, the forces though line CC are additive and plate must be able to resist the vector sum
    f. The transfer plate slenderness must be such that it will not buckle in compression
        i. Check outstand ratio at beginning of side quadrant $(b_{tp}/t_{tp})\leq 22/\sqrt{(ftp/250)}$. This is based on requirements of category 4 members in table 12.5 of NZS3404 (SNZ,2007). For a category 3 member the ration equation becomes $(b_{tp}/t_{tp})\leq 9/\sqrt{(ftp/250)}$. If this ratio is not met then the plate edge must be stiffened.
        ii. Check unsupported edge dimension from the end of the supported beam to the beginning of the side quadrant satisfies the ratio $(b_g/t_{tp})\leq 45/\sqrt{(ftp/250)}$ This equation is equation 10.16 of HERA Design Guide Volume 1 (Clifton, 1994). If this ratio is not met then check the maximum outstand ratio between these points meets the $22/\sqrt{(ftp/250)}$ limit.
2. The vertical shear force on the web plate is resisted directly by the CHS column.
    a. The local vertical shear capacity of the CHS column wall is given by $\Phi(0.83x0.6xf_{ycol})2t_{col}d_b$. Where $0.83x0.6xf_{ycol}$ is the shear yield stress and area over which this check is made is $2t_{col}d_b$.
3. The welds between the web plate and the top and bottom horizontal transfer plates must be designed to transfer the horizontal forces from the web plate.
4. The double sided welds between the transfer plates and the column wall for the cruciform arrangement are sized for the following:
    a. A longitudinal shear force equal to the overstrength action from the 82% proportion of the out of balance horizontal forces that goes into the side quadrants.
    b. An overstrength transverse tension force equal to 9% of the horizontal force generated by the incoming beam that is attached directly to that side quadrant for a cruciform section
    c. These two forces are combined using the vectorial sum method of NZS 3404 Clause 9.7.3.10.2 (SNZ, 2007)
    d. The resulting weld size is used around the full circumference
    e. For example the upper quadrant of the CHS shown in figure 3, the longitudinal shear force is generated by beams 1 and 3 and the transverse tension force is generated by beam 4.
5. A check for local yielding of the CHS side quadrants under the incoming overstrength shear force must be made.
    a. The design capacity of these side quadrants is given by $\Phi x(0.83x0.6xf_{ycol})4t_{col}L_{sidequad}\eta$
    b. The factor η is as defined in NZS3404 Eq 12.9.5.3(5) (SNZ, 2007)
    c. The design action is equal to the out of balance overstrength action from the 82% proportion of the out of balance horizontal action that goes into the side quadrants

6. A check is required for the CHS panel zone
    a. The panel zone shear capacity of a bare steel CHS member is determined from NZS3404 clause 5.11.4.2 (SNZ, 2007)
        i. $V_w = 0.36 f_y A_e \eta$
        ii. Slenderness ratio of CHS to comply with table 12.5 NZS3404 for Category 1,2 or 3 and $82(\sqrt{250/f_y})$ for Category 4
    b. The same equation can be used for a concrete filled CHS to NZS 3404 clause 13.8.3.
        i. The slenderness ratio of 120 can be used for all categories
        ii. The shear capacity of the concrete core can be added to that of the steel section.
    c. The panel zone shear action is determined from the overstrength moment capacity of the joint or beam

References

Clifton C., Semi-Rigid Joints for Moment-Resisting Steel Framed Seismic-Resisting Systems, PhD Thesis, Department of Civil and Environmental Engineering, University of Auckland, 2005 (Also available as a HERA document)

Clifton, C., Panel Zones in Moment-Resisting Beam to Column Connections : Design and Detailing Recommendations, HERA Steel design and Construction Bulletin (DCB), Issue 57, Manukau City, 2000

Clifton, C., Structural Steelwork Limit State Designs Volume 1; Report R4-80, HERA, Manukau City, New Zealand, 1994

SNZ, Steel Structures Standard (Incorporating Amendments 1 and 2), NZS 3404:1997, Standards New Zealand, Wellington, 2007