IMPROVED CONFIGURATION OF I-BEAM TO BOX COLUMN CONNECTIONS IN SEISMIC STEEL MOMENT FRAMES

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ABSTRACT :
In countries like Japan and China, a number of steel moment frame buildings are built with box columns and I-beams. The I-beam to box column (IBBC) connection is not straight-forward. Seismic design of moment frames expects these IBBC connections to accommodate 4-6% drift levels that the building sustains during strong earthquake shaking. The studies reported in literature on seismic behaviour of IBBC connections do not clarify whether the current IBBC connections can accommodate these drifts. The configurations of IBBC connections currently in use have major deficiencies, like discontinuity in flow path of forces between beam and column. This paper presents an improved IBBC connection configuration, which overcomes many deficiencies present in current IBBC connections. Displacement-based inelastic finite element analyses are performed to estimate the drift capacities of current and proposed IBBC connections; the proposed connection is seen to perform much better.

KEYWORDS:
Steel frames, box column connections, seismic design, moment connections, drift capacities

1. INTRODUCTION

Steel moment resisting frames (MRFs) are required to sustain large lateral drift levels under seismic shaking. Most of this is to be accommodated through inelastic actions near the beam-to-column joints. This requires that the design of connections for steel beam-to-column joints is based on capacity design concept – connection elements and welds are designed corresponding to beam overstrength capacity to remain elastic at sufficient lateral drift levels. This poses significant challenge in design of IBBC connections due to certain typical geometric differences of box-columns compared to wide-flange I-columns (as in standard strong-axis connections) – the most important difference being the location of webs in box-sections. In standard strong-axis connections, column web is coplanar with beam web. However, in IBBC connections, the two column webs are in planes that are at offset from the plane of the beam web and the beam connects only to the box column flange. The out of plane flexibility of the box column flange significantly affects the seismic behaviour of such IBBC connections. It induces large stress concentration near ends of beam flange leading to local yielding of connection elements, fracture of welds, and thus, hinders mobilization of beam plastic moment capacity.

To counter the challenges imposed by the highly flexible box column flange, beam forces need to be delivered directly only to the two side walls of the box column. This led to the development of various external stiffening systems using, for instance, horizontal flaring of beam to match the column width, along with use of angle and T-stiffeners (Ting et al., 1991; Shanmugam et al., 1991). External T-stiffener connection is effective because it attempts to direct the beam force to column webs. T-stiffener connection also is used in CFT columns. Analytical and experimental studies of IBBC connections with T-stiffeners showed improvement of connection performance in terms of higher strength, stiffness and rotation capacity (Lee et al., 1993; Shanmugam and Ting, 1995; Shin et al., 2004; Kim and Oh, 2007). Additional details studied include RBS cutouts in beam flanges, slotted holes in horizontal part of T-stiffeners (Shin et al., 2008). However, most specimens in the above studies showed brittle failure, particularly in the horizontal part of the T-stiffener, which eventually propagates into the reinforced connection region.

2. FORCE FLOW

In IBBC connections, box column webs (parallel to the beam web) are stiffer than the flanges and thus draw more force. This results in a natural force flow path from the beam centerline towards the column webs. This is illustrated in Figure 1 showing maximum principal stress directions in half-model finite element analysis of unreinforced IBBC connections. This causes stress concentrations at re-entrant corners. The T-stiffener connection attempts to transfer the beam force to the box-column webs, but only at the edges of the beam flanges. Re-entrant corners in force flow path, particularly at junction of horizontal portion of T-stiffener and beam flange becomes possible locations of brittle crack origination and significantly jeopardize the efficacy of joints under seismic conditions. Gradual transition of geometry, devoid of any abrupt change (re-entrant corners) is important in reducing stress concentration in connections and ensures smooth flow of forces. A direct *planar continuity*, between I-beam and box column web planes, needs to be established.

Finite element study also shows that beam shear is not transferred by the beam web through column flange or column web in the panel zone. Instead, stress field in the beam near the joint, as shown in Figure 2, indicates a preferential force flow path towards the beam flanges leaving a relatively cold region near the beam centerline. Analytical and experimental studies of conventional strong-axis connections also showed similar results (Goel et al., 1997; Lee et al., 2000; Arlekar and Murty, 2004; Popov and Bertero, 1973). This is because the classical Bernoulli theory does not apply near the joints. This observation indicates that IBBC connection needs to be reinforced at beam flange levels – cold column panel zone suggests that beam web shear is not transferred through the beam web to the column, and as such, connecting beam web to column flange is not structurally needed. Thus, beam web need *not* be connected to the column.

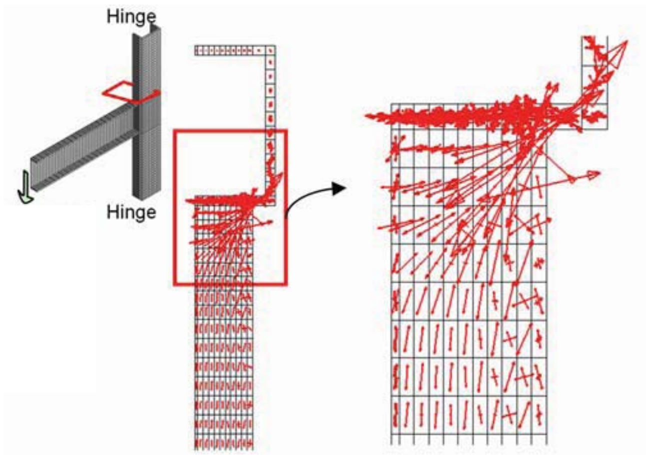

Figure 1 Maximum principal stress directions showing preferential force flow from beam web plane towards two box column webs in IBBC connections.

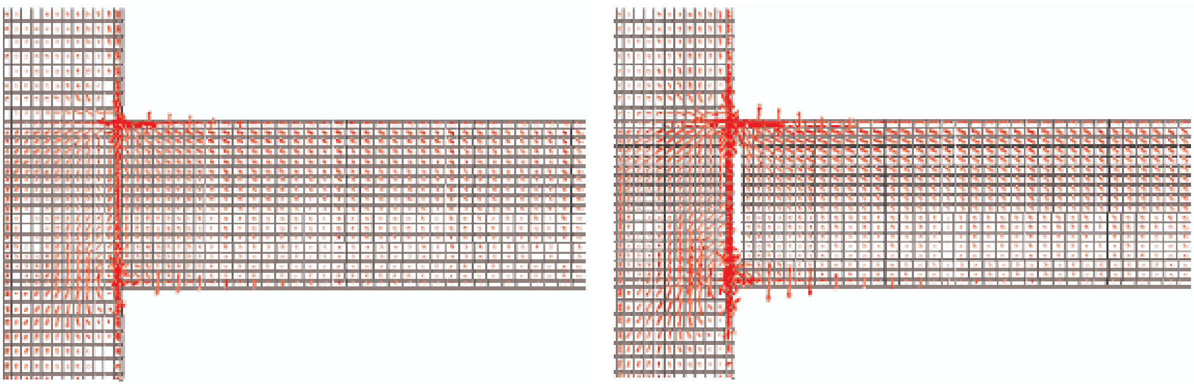

Figure 2 Principal stress directions showing preferential force flow from beam centerline towards beam flanges in IBBC connections with and without beam web connection.

3. CONNECTION CONFIGURATION

To ensure planar continuity between beam and box column webs for smooth flow of beam shear into the column webs, a reinforced connection with inclined rib plates is introduced based on preliminary study on flow of forces. Further, collar plates at beam flange levels encircling the box-column is introduced to keep connection elements in elastic condition when plastic hinge forms in beam with overstrength due to uncertainty in specified minimum yield strength (e.g., R_y = 1.5 for steel with F_y = 250 MPa) and possible strain-hardening. Collar plates also satisfy the natural force flow path going outwards from beam towards the column webs. These arrangements decrease demand on the column flange. Additional web plates in plane with the inclined rib plates along with horizontal flaring of the beam flange up to the column corners further reduce stress concentrations near the weld access holes and at the corner of the column. The configuration, shown in Figure 3, eliminates the need for beam web connection to column flange and further relieves stresses in the region.

In the connection considered, (1) beam flanges welded to column flange with CJP groove welds; (2) collar plates are fillet welded to beam flanges, and CJP groove welded to column and inclined rib plates; (3) inclined rib plates are CJP groove welded to beam flanges, collar plates and column; (4) inclined web-plate is fillet welded to the beam flange, and CJP groove welded to the beam web and column, (5) beam web is not connected to the column flange, and (6) connection length is equal to half the beam depth.

4. CONNECTION DESIGN

The connection design forces are based on capacity design concept. Thus, the maximum achievable beam plastic moment Mpr and the associated equilibrium compatible shear Vpr are given by

$$M_{pr} = R_y R_s M_p, \text{ and} \quad (1)$$

$$V_{pr} = 2M_{pr} / L_o, \quad (2)$$

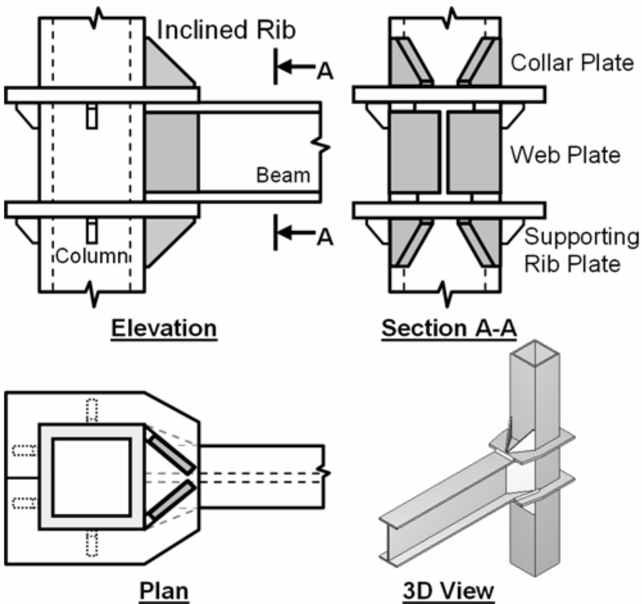

Figure 3 Inclined rib-plated collar-plated IBBC connection configuration with additional web plates. The supporting small rib plates below collar plates are only to facilitate proper welding of the collar plates to the column.

where R_y and R_s are strength modification factors on the nominal plastic moment capacity M_p of the beam, and L_o the shear span between two plastic hinge locations at the two beam ends. In the connection, the inclined rib plates are the most critical elements – they are under combined action of shear and compression (or tension). Thus, stability design of inclined rib plates is required to prevent buckling, apart from strength design. This is achieved by suitably designing the thickness t_{irp} of inclined rib plate as

$$(d/t_{irp}) \leq 16.9\,(l_{irp} / h_{irp} + h_{irp} / l_{irp}) \text{ for } F_y = 250 \text{ MPa, or} \quad (3)$$

$$(d/t_{irp}) \leq 14.4\,(l_{irp} / h_{irp} + h_{irp} / l_{irp}) \text{ for } F_y = 345 \text{ MPa,} \quad (4)$$

where l_{irp} and h_{irp} are the base length and height of the inclined rib plate of diagonal length d.

5. NUMERICAL STUDY

All numerical investigations in this study are done through displacement-controlled nonlinear finite element analysis using ABAQUS (HKS, 2005). Both material and geometric nonlinearity are considered in the analyses. The beam, column, connection elements and welds (subassemblages) are discretised using 8-node solid elements with three translational degrees of freedom at each node. A finer mesh is used to model the connection region and the beam region up to a distance of half the beam depth. The transition of finer mesh to coarse mesh is modeled using 6-node linear solid wedge elements. CJP groove welds and fillet weld are explicitly modeled. Due to symmetry of geometry and loading, only half of subassemblages are modeled. Figure 4 shows a typical finite element model of an IBBC joint-subassemblage with the proposed connection configuration.

In all finite element models, beams are of grade ASTM A36 grade steel with yield strength F_y of 250 MPa, and columns and connection elements of ASTM 572 Grade 50 steel with F_y of 345 MPa. The connecting weld material (E70 electrode) has yield strength of 345 MPa and ultimate tensile strength of 480 MPa at 20% elongation. All materials have same initial elastic modulus of elasticity E of 200 GPa. Poisson's ratio is assumed to be 0.3 for all the three materials. Material nonlinearity is accounted through a classical isotropic plasticity model based on von Mises yield criterion and associated plastic flow. True stress-strain data are given as input for the three materials. Cumulative plastic strain excursion is used for performance evaluation and comparison, apart from element stresses, element strains, global deformations and global reactions. Strong column weak beam (SCWB) design philosophy is employed along with seismically compact sections (AISC, 2005).

Performance evaluation of the proposed IBBC connection configuration is done in two broad steps. First, monotonic pushover analysis (up to a maximum of 4% drift) of exterior joint-subassemblages are done to assess the effectiveness of the proposed design in terms of degree of mobilization of inelasticity in the beam. Joint subassemblages consist of 3 m long (from column centerline to tip of beam free end) I-beams connected to 3.8m

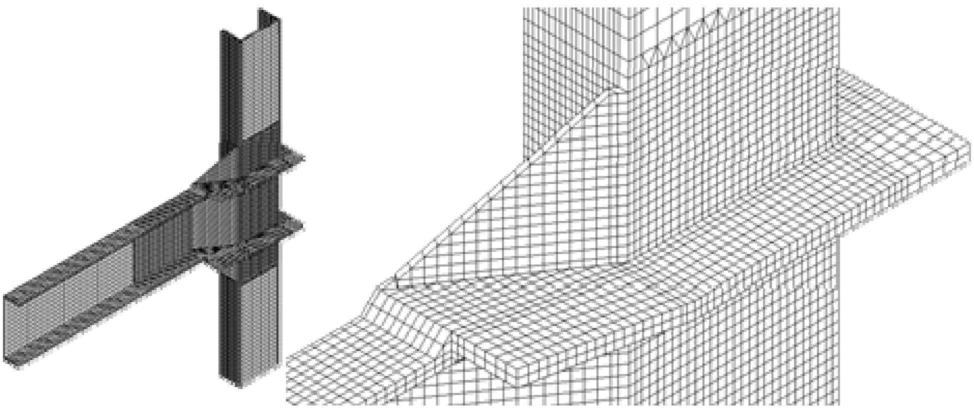

Figure 4 Typical finite element discretisation of a symmetric half of subassemblage
(W27×178 beam; Box 500×500×30 column).

long box column stubs. The ends of columns are hinged. The range of column size is chosen from practical considerations – the size of box-columns that can be usually used in low-medium rise steel MRFs. The geometric and strength properties of the 16 joint-subassemblages studied are given in Table 1. The symbols b, d, h, l, and t denote beam, depth, height, length and thickness, respectively. Subscripts $_{b}$, $_{bf}$, $_{bw}$, $_{c}$, $_{cp}$ and $_{irp}$ respectively represent beam, beam flange, beam web, column, collar plate and inclined rib plate. I and M_p denotes moment of inertial about bending axis and nominal plastic moment capacity respectively.

Next, displacement-controlled cyclic pushover analysis is done on representative exterior joint subassemblage using proposed connection configuration. In seismic performance of welded beam-to-column connections, behavior of complete joint penetration welds between the connection elements and column face are most critical under reversed cyclic loading, and therefore of steel MRF buildings. Further, performance of beam-to-column joints under large reversed cyclic displacement loading provides understanding of the likely inelastic response of the connections during strong seismic conditions. Thus, FEMA (FEMA 350, 2000) recommended multi-cycle displacement load history for connection prequalification is used for performance evaluation (Figure 5).

Table 1 Geometric and strength properties of IBBC joint-subassemblages studied.

	Dimensions (mm) and Ratios													
	Column	Beam					I_b / I_c	M_{pb} / M_{pc}	Collar Plate			Inclined Rib Plate		
	d_c , t_c	Name	d_b	b_{bf}	t_{bw}	t_{bf}			l_c	b_{cp}	t_{cp}	l_{irp}	h_{irp}	t_{irp}
1	650×650×38	W33×241	869	404	21	36	1.01	0.52	435	804	50	480	435	24
2		W27×178	706	358	18	30	0.50	0.32	353	678	40	428	353	20
3	550×550×32	W33×241	869	404	21	36	1.99	0.87	435	796	49	454	435	22
4		W27×178	706	358	18	30	0.98	0.52	353	670	39	395	353	20
5		W24×162	635	330	18	31	0.72	0.43	318	586	32	373	318	18
6		W21×147	561	318	18	29	0.51	0.34	281	566	31	349	281	16
7		W18×106	475	284	15	24	0.27	0.21	238	476	24	325	238	14
8	500×500×30	W27×178	706	358	18	30	1.40	0.68	353	670	39	379	353	20
9		W21×147	561	318	18	29	0.72	0.44	281	566	31	332	281	16
10		W18×106	475	284	15	24	0.38	0.27	238	476	24	307	238	14
11		W16×89	427	264	13	22	0.26	0.21	214	440	22	292	214	14
12		W12×58	310	254	9	16	0.09	0.10	155	398	18	261	155	12
13	450×450×30	W21×147	561	318	18	29	1.01	0.56	281	566	31	316	281	16
14		W16×89	427	264	13	22	0.37	0.26	214	440	22	278	214	12
15	390×390×24	W12×58	310	254	9	16	0.25	0.21	155	398	18	222	155	10
16	350×350×22	W16×89	427	264	13	22	1.05	0.59	214	440	22	244	214	12

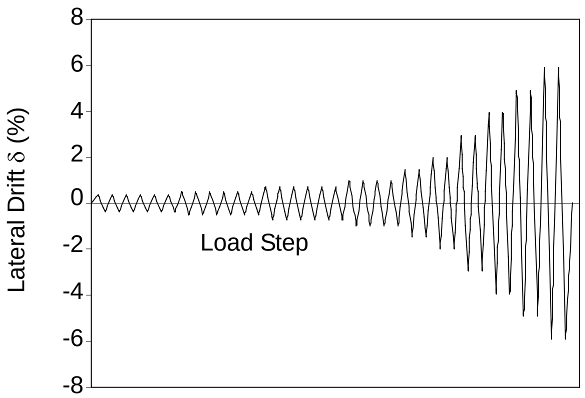

Figure 5 Reversed multi-cycle loading for performance evaluation.

5.1. Monotonic Pushover Analysis

Figure 6 shows the beam transverse load V, normalized with V_{pb}, versus drift curves of the 16 IBBC joint-subassemblages studied, where V_{pb} is the load at the cantilever beam tip that would result in bending moment equal to nominal plastic moment capacity M_{pb} of the beam at end of connection reinforcement region. Table 2 shows the average level of inelasticity mobilized in the subassemblages at 0.33%, 1%, and 4% drift levels. The nominal beam plastic moment capacity is mobilized with sufficient strain-hardening.

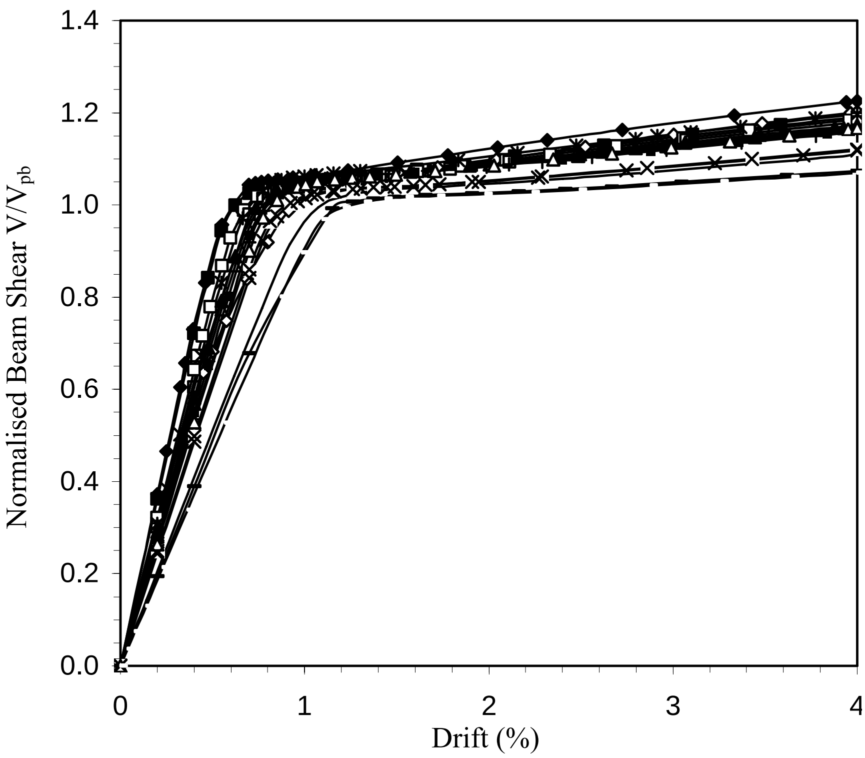

Figure 6 Monotonic normalized load versus drift curves of the subassemblages with proposed IBBC connection.

Table 2 Average beam capacity mobilized in subassemblages with proposed connection under monotonic loading.

Drift (%)	V / V_{pb}
0.33	0.46
1	1.03
4	1.16

5.2. Cyclic Pushover Analysis

Figure 7 shows cumulative plastic strain contour of a typical subassemblages with the poposed connection configuration at the end of the full multi-cycle loading history (6% drift). Inelasticity is confined in the beam span away from the connection reinforcement region; all connection elements including the CJP groove welds remain elastic even at the end of the full multi-cycle loading history. However, the entry fillet weld connecting the collar plate to the beam flange is the critical connection component here, which could be stressed beyond its yield strength. This is expected because of the presence of large inelasticity in the beam flange immediately beyond the entry fillet weld; the beam flange is expected to undergo large strain-hardening under strong seismic shaking. This requires the collar plate to be provided to reduce the stress intensity within the connection region by accommodating the beam overstrength forces. Nonetheless, the improved seismic response in terms of concentrating inelasticity only in the beam is achieved through the inclined rib plates and web plates bridging the otherwise discontinuity in the beam and column webs in IBBC connections, ensuring smooth flow of forces. Table 3 shows the maximum stresses and strain in a typical IBBC connection.

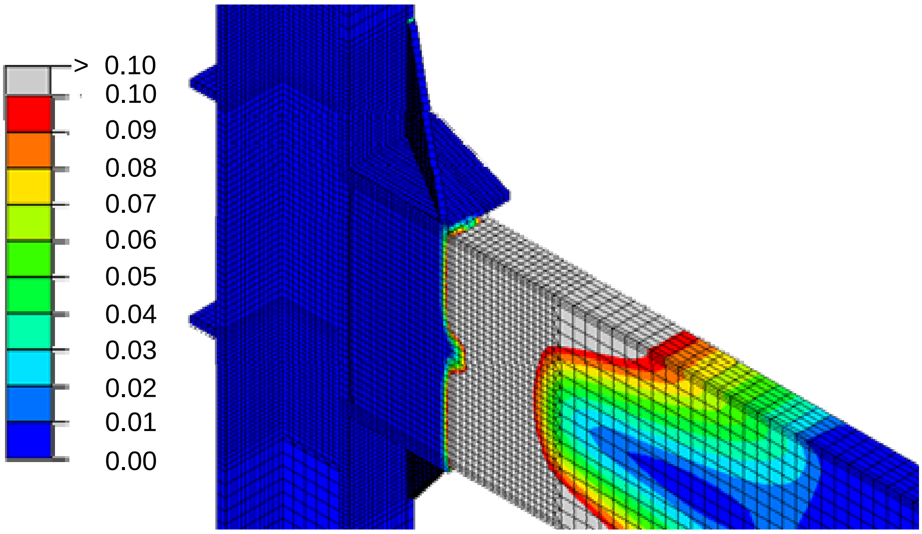

Figure 7 Yielding (cumulative plastic strain) of only beam beyond connection reinforcement region at end of multi-cycle displacement loading up to 6% drift.

Table 3 Inelastic finite element results under multi-cycle displacement loading at end of 6% drift.

Maximum (or minimum) true normal stress (MPa) in CJP groove weld connecting the beam to the column	± 228.7
von Mises stress (MPa) in CJP groove weld connecting the beam to the column at end of full load cycle	286.0
Maximum cumulative plastic strain in beam flange	4.920
Location of maximum weld stress	Entry fillet weld

6. CONCLUSIONS

The results of finite element analyses predicts good seismic performance of the proposed IBBC connection configuration; validation of the same needs to be done through experiments. From the analytical study, the following salient conclusions are drawn:

- Gradual transition in geometry without re-entrant corners ensuring planar continuity between beam web and two webs of box column is critical to good seismic behaviour of I-beam to box column connections.
- Inclined rib plated collar plated connection configuration is effective in pushing the location of energy dissipating plastic hinge away from column face to the end of the connection reinforcement region in the beam.
- It is not necessary to connect the beam web to box column flange as the entire beam shear is directed towards the column webs by triangular inclined rib plates along with vertical beam web plates.
- Mobilization of nominal beam plastic moment capacity with sufficient strain hardening of beam flanges can be achieved in I-beam to box column connections.

REFERENCES

AISC (2005). Seismic Provisions for Structural Steel Buildings, American Institute of Steel Construction, Inc., Illinois, USA.

Arlekar, J.N. and Murty, C.V.R. (2004). Improved Truss Model for Design of Welded Steel Moment-Resisting Frame Connections. *Journal of Structural Engineering*, ASCE, **130:3**, 498-510.

FEMA 350 (2000). Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings. SAC Joint Venture, CA, USA.

Goel, S.C., Stojadinović, B. and Lee, K.-H. (1997). Truss Analogy for Steel Moment Connections. *Engineering Journal*, AISC, **34:2**, 43-53.

HKS (2005). ABAQUS/Standard User's Manual, Hibbitt, Karlsson & Sorensen, ABAQUS Inc., RI, USA.

Kim, Y.-J. and Oh, S.-H. (2007). Effect of the Moment Transfer Efficiency of a Beam Web on Deformation Capacity at Box Column-to-H Beam Connections. *Journal of Constructional Steel Research*, **63:1**, 24-36.

Lee, K.-K., Goel, S.C. and Stojadinović, B. (2000). Boundary effects in Steel Moment Connections. Proceedings of the 12th World Conference on Earthquake Engineering, 12WCEE, Paper No. 1098, Auckland, New Zealand.

Lee, S.L., Ting, L.C. and Shanmugam, N.E. (1993). Use of External T-Stiffeners in Box-Column to I-Beam Connections. *Journal of Constructional Steel Research*, **26:2-3**, 77-98.

Popov, E.P. and Bertero, V.V. (1973). Cyclic Loading of Steel Building Connections. *Journal of Structural Engineering*, ASCE, **99:6**, 1189-1204.

Shanmugam, N.E. and Ting, L.C. (1995). Welded Interior Box-Column to I-Beam Connections. *Journal of Structural Engineering*, ASCE, **121:5**, 824-830.

Shanmugam, N.E., Ting, L.C. and Lee, S.L. (1991). Behaviour of I-Beam to Box-Column Connections Stiffened Externally and Subjected to Fluctuating Loads. *Journal of Constructional Steel Research* **20:2,** 129-148.

Shin, K.-J., Kim, Y.-J. and Oh, Y.-S. (2008). Seismic behaviour of composite concrete filled tube column-to-beam moment connections. Journal of Constructional Steel Research, **64:1**, 118-127.

Shin, K.-J., Kim, Y.-J., Oh, Y.-S. and Moon, T.-S. (2004). Behavior of welded CFT column to H-beam connections with external stiffeners. *Engineering Structures*, **26**, 1877-1887.

Ting, L.C., Shanmugam, N.E. and Lee, S.L. (1991). Box-Column to I-Beam Connections with External Stiffeners. *Journal of Constructional Steel Research*, **18:3**, 209-226.