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SEISMIC ANALYSIS FOR STEEL BEAM TO COLUMN JOINT CONNECTION: A REVIEW

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Abstract*— Different steel sections provide an alternative to typical wide flange section in low-rise seismic moment resisting frame. Their beneficial strength to weight ratio as well as bending, compression and torsional resistance increase the versatility of moment frames and potentially improves performance under earthquake loads. With an understanding of current design requirement for seismic moment connection and static beam to column connection. Fully welded unreinforced connection are explored through finite element analyses. However, performance is limited due to column face plastification and inability to develop the plastic moment capacity of the beam. To rectify this problem through plate and external diaphragm plate beam to column moment connection are analysed. Several important geometric parameters such as the beam width-column width ratio, beam thickness-column thickness ratio, plate length and plate thickness are considered to understand their effect on the connection moment capacity and sources of inelastic rotation. The through plate and external diaphragm reinforcement greatly improve the connection behaviour moving yielding away from the column face and into the beam member. The results from the reinforced connection are used in combination with current seismic design provisions to develop a design procedure that optimizes the performance of these connections. The reinforced connection can be detailed to develop plastic hinging in the beam member while minimizing the likelihood of a non-ductile weld failure.

Keywords*— castellated beam section, hollow structural section, rigid and semi-rigid connections, beam - column connections, design (structural design), bending moment, rotation, plastic hinge, structural analysis & design software

I. INTRODUCTION

The general accepted concept of seismic designing is based on the model of a ductile structure. This design approach assumes a partial and controlled plasticity of the structure under the load of the designed earthquake (the strongest expected one). Ductile frames accept horizontal loads primary with bending their columns and beams, and in the case of the designed earthquake they should obtain form of beam mechanisms, which means they should obtain appearance and development of plastic hinges in the beams ends areas, but not in the columns or in the frame nodes. As the jointing of the columns to the beams is obtained with beam-column connections, the nonlinear deformations can be located in the beams and/or connections, as functions of the bearing capacity of the connections. The location of the plastic hinges can be controlled with inclination of the bearing capacity of ones in relation to the others with the specified over strength factors.

II. PROPOSED CONNECTIONS AND CONNECTIONS PERFORMANCE

The proposed connection between a beam and column is essential features of the connection are two channel connections of uniform thickness, welded to the column in the fabrication shop. These channel connections transmit the stress resultants of the beam flange to the column webs by shear lag action. Rectangular openings were provided in the beam webs so as to allow bolting between the beam flange and the channel connector. Adequate tolerance and end clearance can be provided between the beam end and the column face to facilitate erection. The beam can be erected between two such connections by lowering it to the required level and rotating in plan to get the required alignment the main considerations in deciding the size of the opening and the channel connection are that the connection should attain an ultimate moment close to the full plastic moment of the beam and the failure modes like shear failure of the channel connector or bolts should be avoided

III. LITERATURE REVIEW

A. Matthew Fadden, Jason McCormick "HSS to HSS Seismic moment connection performance and design" Journal of constructional steel research , University of Michigan , department of civil and enviromental engineering Ann Anbor , MI, 48109-2125, USA

In this article, Matthew Fadden and Jason McCormick studied on seismic moment frames has mainly focused on systems composed of wide-flange members or hollow structural section columns and wide-flange beams. Hollow structural section (HSS) members have many desirable properties that make them a candidate for more pervasive use in low-rise seismic moment frames. These properties include bending, compression, and torsional resistance leading to potentially more efficient and robust steel moment frame systems. Current seismic design methodologies for steel moment resisting frames require the majority of inelastic behaviour to occur at the beam ends For HSS-to-HSS seismic moment connections is show in fig. 1, the Hollow structural section beam member must be able to undergo large inelastic deformations without a significant drop in strength up to inter story drift levels of 2% or 4 % for intermediate moment frames (IMF) or special moment frames (SMF)

Fig. 2 a and b show the normalized maximum moment capacity with respect to the beam width–column width ratio for the through plate and external diaphragm plate connections. On average both connections showed a decrease in the normalized moment capacity with an increase in the beam width–column width ratioThese connections were limited by column face plastification, column punching shear, beam fracture, column sidewall yielding, and column shear. For connections with small beam–column width ratios ($\beta \leq 0.85$), the moment capacity could be determined by a yield line analysis. Connections with large beam–column width ratios (βN 0.85) were limited by chord side wall failure. To better understand the effect of the beam–column width ratio (β), beam– column thickness ratio (t_b/t_c), and depth of the beam member on the cyclic hysteretic behaviour, finite element models of unreinforced Hollow structural sections moment connection(fig, 1) were studied considering different beam members. The degree of inelastic rotation in the beam and panel zone was compared for the three connections is plotted in fig. 3

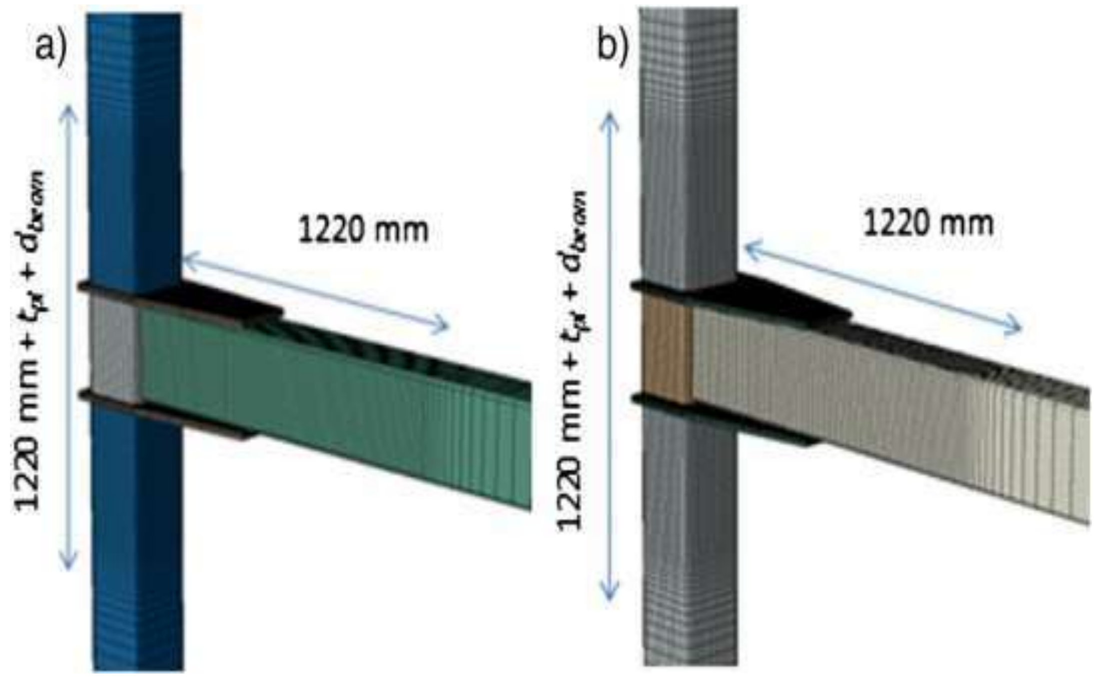

Fig.1 Finite element models for the (a) through plate and (b) external diaphragm plate HSS-to-HSS moment connections.

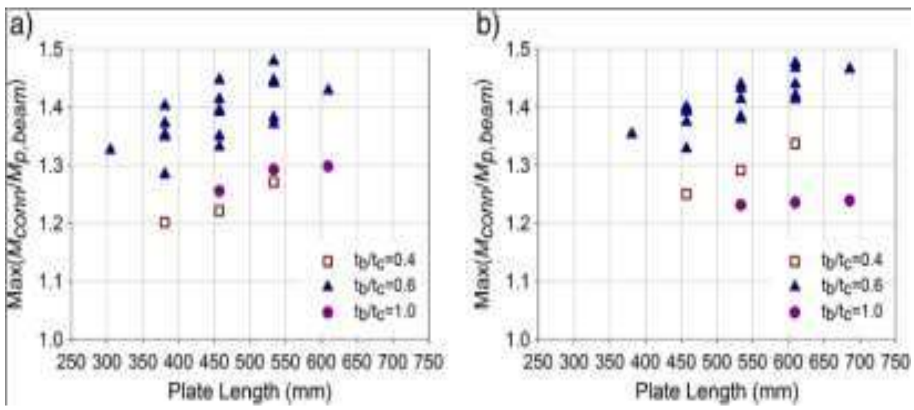

Fig. 2 Maximum normalized moment capacity versus plate length for the (a) through plate and (b) external diaphragm plate connections

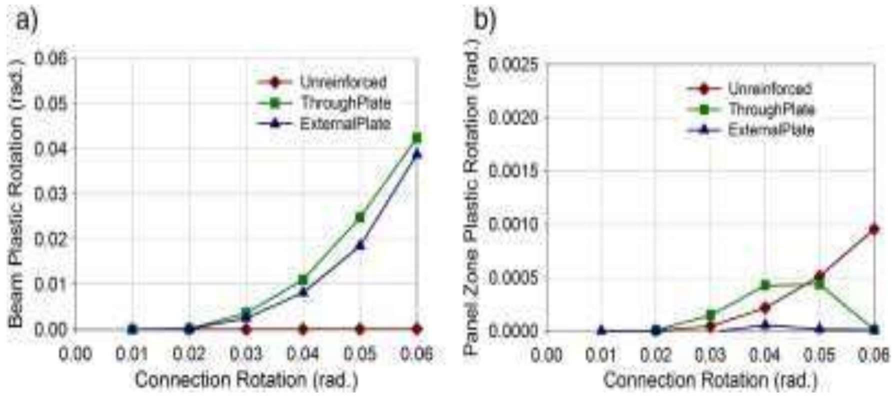

Fig. 3 Plastic rotation in the (a) beam and (b) panel zone for the unreinforced, through plate

B. S.R. Satish kumar, D.V. Prasada Rao "RHS Beam to column connection with opening experimental study and finite element modelling" journal of constructional steel research , department of civil engineering, IIT Madras, Channai 600 036, India-2005

S.R. Satish kumar and D.V. Prasada Rao give information Rectangular Hollow Sections (RHS) have superior structural performance compared to conventional steel sections. There proposed RHS Beam to Column connection is show in fig. 4 However, the application of RHS in structural steel framework is limited because suitable connection configurations have not been developed between such members. Also adequate information on the moment–rotation characteristics of connections between RHS members is not available for design. To overcome these problems, a new and efficient connection is proposed which is easy to fabricate and convenient for erection. The connection employs channel connectors welded to the column flange and bolted to the beam to transfer beam flange forces into the column webs thereby avoiding the need to provide internal diaphragms in the column. An opening in the web facilitates the installation of bolts and can be used to pass service lines. The bolts are loaded in shear, so as to obtain improved performance of the connection under cyclic loading. By choosing suitable dimensions for the channel connectors, the strength and stiffness of the connection can be varied. The behaviour of the connection is evaluated by cyclic tests and non-linear finite element analysis. Test results are presented in the form of hysteretic curves and failure modes. In the case of channel connectors of high strength, failure occurs at the beam net section away from the face of the column, similar to beams with Reduced Beam Sections (RBS). The moment–rotation characteristics of the connection can be expressed in terms of the three-parameter power model, for semi- rigid frame analysis. The results of yield and ultimate loads and corresponding displacements for all the specimens are summarized in Table: 1

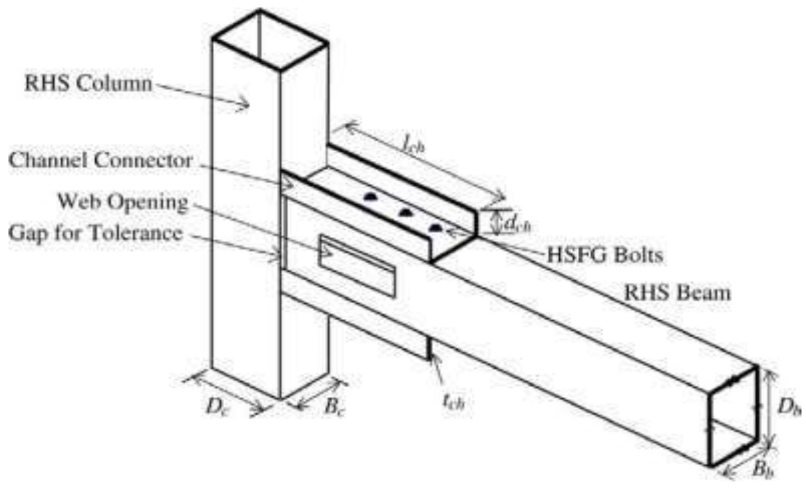

Fig. 4 RHS Beam to RHS column Connection

Table: 1 Results of the tested specimen

Specimen	Yield load (P_y) KN	Yield Displacement (δ_y) KN	Ultimate load (P_u) KN	Ultimate displacement (Δ_u) KN	Initial stiffness (R_{ki}) KN/m
1	54	15.1	92	118	152.2e-3
2	52	15.0	91	115	154.7e-3
3	35.5	12.2	68	95	32.2e-3
4	33.6	10.0	70	90	30.2e-3

C. Qian-Yi Song, Amin Heidarpour, Xiao-ling Zhao, Lin-Hai Han "Performance of flange-welded/ web bolted steel I-beam to Hollow tubular column connection under seismic load" Thin-Walled structure

Qian-Yi Song, Amin Heidarpour, Xiao-ling Zhao, Lin-Hai Han have studies on 4 types of connections were fabricated at the Monash Civil Engineering Workshop. The flange-welded/web-bolted connections were made of I-beams and tubular columns through welds, double angle and bolts, as well as the configurations of the simple welded connection and the bolt connection investigated previously for comparison, as shown in Fig, 5. The flanges of the I-beam were directly welded on the column face through single-V welds. Two steel angles were bolted to the I-beam web and welded on the column face through fillet welds. All the I-beams were 200UB22.3 of Grade 250 structural mild steel. Square tubular columns with cross-section of 200 mm×200 mm, made of Grade 350 structural mild steel, were adopted. Two column wall thicknesses, of 5 mm and 9 mm, were used in this study. The cross-section of the adopted steel angle was 65 mm×65 mm×6 mm whilst 8.8M10 bolts were utilised. Additionally, plates with nominal thickness of 8 mm were welded on the top and bottom of the columns, and two stiffeners with nominal thickness of 5 mm were welded to the I-beam, 1 m away from the column panel surface where the load was applied during testing

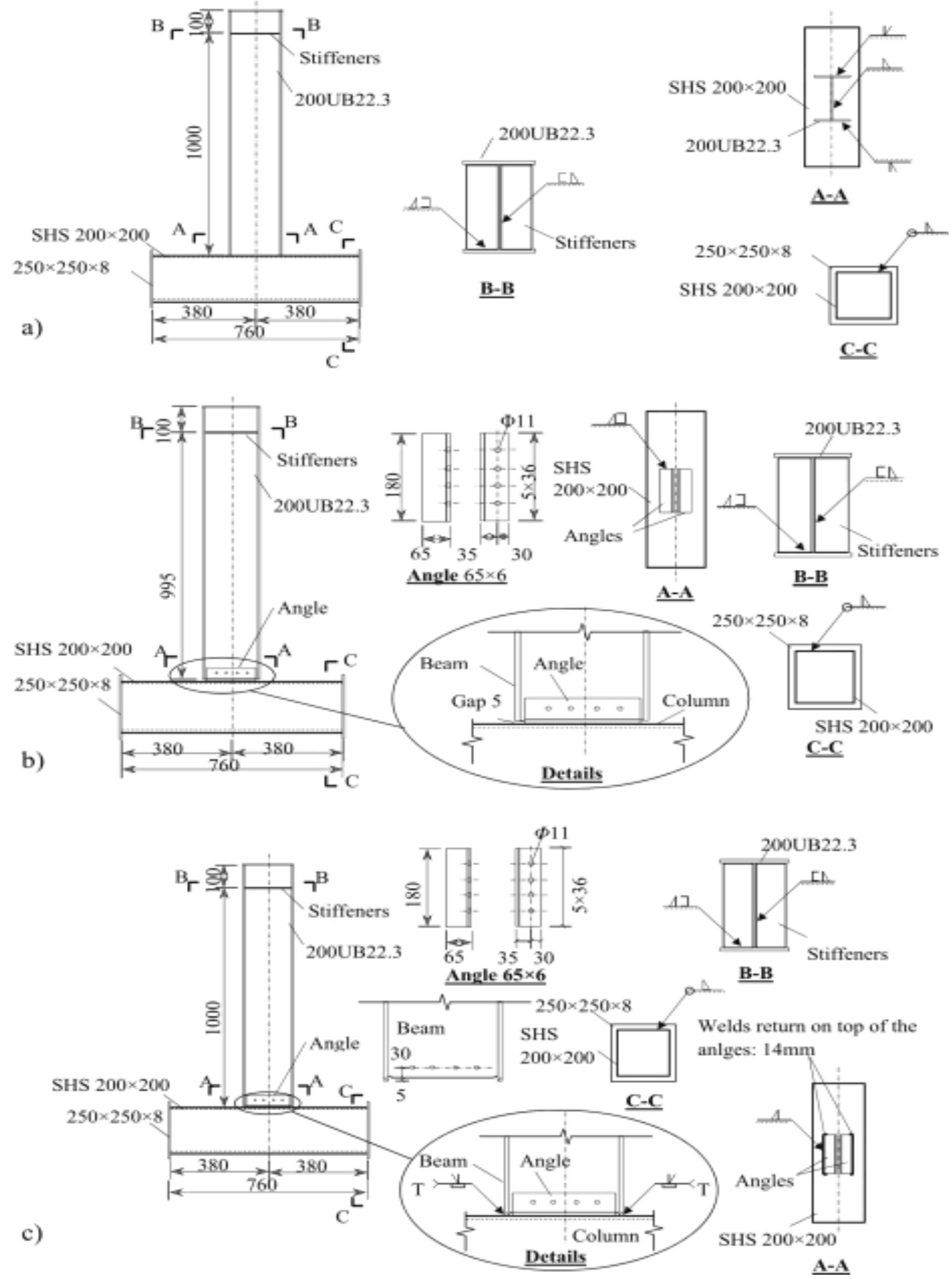

Fig. 5 Configuration details of steel I-beam to hollow square column connections (Unit: mm)

a) Unstiffened welded connection b) Double-angle bolted shear connection c) flange-welded/web-bolted connection.

D. "Cyclic behaviour of hexagonal castellated beam in steel moment resisting frame with post-tensioned connection- 2017"

Hassan Abedi Sarvestani in this studies, behaviour of ten beam specimens, including five hexagonal castellated beams and five typical wide flange beams, in the post-tensioned semi-rigid beam-to-column steel connections of moment-resisting frames has been analyzed theoretically and numerically by well-known finite element software (ABAQUS) under the earthquake simulated by cyclic loading up to 4.4% lateral drift. Hexagonal castellated beams provide lower weight and higher bending strength in comp arison with typical wide flange beams in post-tensioned semirigid steel connections. The inelastic deformations of the bolted top and bottom angles provide reliable level of energy dissipation, and the steel strands provide self-centering capability and revert the moment-resisting frame to its initial position without residual drift after a severe earthquake. The results of finite element analysis verified against experiment have proved that the theoretical method based on the previous researches accurately predicts the behaviour of hexagonal castellated beams in the post-tensioned connections. All beam specimens did not suffer from the flange buckling and web shear buckling under the standard cyclic loading up to 4% lateral drift; likewise, hexagonal castellated beams in the post-tensioned connections showed adequate strength against web-post failure and vierendeel mechanism up until 4% drift. In order to investigate the ultimate failure modes of the specimens, they have also been subjected to an added half-cycle loading up to 4.4% lateral drift beyond the standard loading. The further drifts have leaded to the web-post buckling in most specimens with hexagonal castellated beams and flange buckling in all specimens with wide flange beams. In addition, the parametric study is suggested as a further investigation to comprehensively understand the behaviour of hexagonal castellated beams in the PT connections and it is required to reach more general conclusions.

The inelastic behaviour of the steel materials representing the Bauschinger effect was defined in order to develop the FE models. Since the results of bolt tests and steel material coupons were not available, practical stress-strain relationships for steel materials were considered to incorporate the cyclic kinematic strain- hardening law and flow rule within the inelastic behaviour after yielding points. Table 2 shows the properties of three steel materials in which high-strength bolts have the material type of A490 Grade50 with trilinear cyclic behaviour high-strength steel strands have the material type of A416 Grade270 with bilinear cyclic behaviour and the rest of the steel components have A572 Grade50 as steel material with trilinear cyclic behaviour The properties of all considered materials have been comprehensively defined in Table 2. Although, the cyclic kinematic strain-hardening law and flow rule of A572 Grade50 has been practically required to fully be considered in this study, it is not important for the steel strands and high-strength bolts designed to remain elastic under lateral cyclic loadings and this information has been only added for full assessment of the selected steel type.

E. SAMARAT BISWAS, "Seismic connection for steel square hollow beam to square hollow column joint" Department of civil engineering , National Institute of technology, Rourkela Odisha 769008, India (MAY-2015)

Samarat Biswas have studies for this project a square hollow beam to square hollow column connection was selected and modelled in commercial finite element software ABAQUS. This model was analysed for nonlinear static analysis considering a number of connection details. Following four alternative scheme of connection details were selected for this study: (i) using end-plate, (ii) using angle section, (iii) using channel sections (iv) using collar plates. The material nonlinearity in the present study stress strain curve of steel is considered as per. Fig. 6 presents the stressstrain relation of mild steel used in the present study. The base model is also studied for reference. This section presents the comparison of the capacity curves obtained from pushover analysis of the joint for different type of connection details. Fig. 7 compares the performance of selected connection details through the resulting capacity curves of the joint. The important characteristics of these curves are presented in Table : 2 This figure and table shows that the load carrying capacity of the joint and the maximum deformation capacity is highly sensitive to the type of connection used.

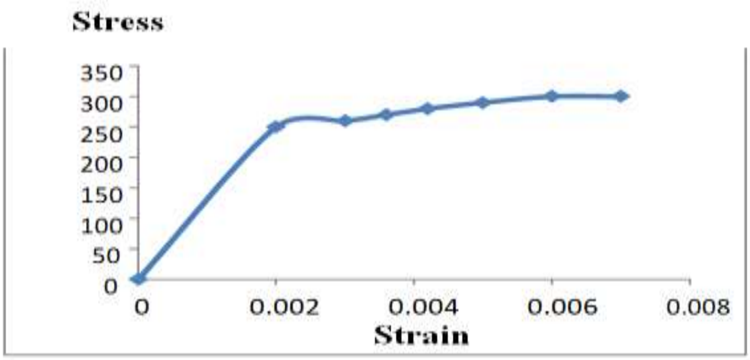

Fig. 6 Stress strain curve for mild steel used in present study

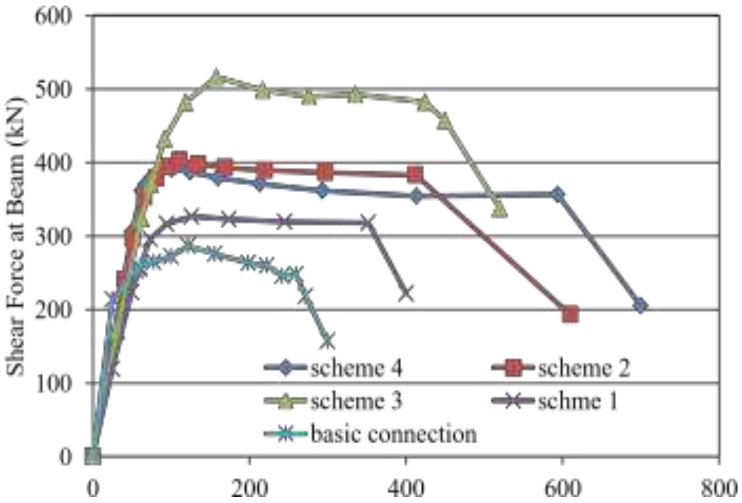

Fig. 7 Comparison of capacity curves for different scheme of connections

Table: 2 Pushover analysis results of the joint for different scheme of connections

Connection Scheme	Maximum Strength (kN)	Yield Deformation (mm)	Ultimate Deformation (mm)	Ductility Factor	Formation of Plastic Hinge at
Basic	267	76	306	3.98	Beam-to-Column Joint
Scheme-1	315	104	413	3.97	Beam-to-Column Joint
Scheme-2	404	100	592	5.92	Beam-to-Column Joint
Scheme-3	**506**	**152**	**627**	**4.20**	**Beam**
Scheme-4	398	98	587	5.99	Beam-to-Column Joint

IV. CONCLUSIONS

From the study of above research papers it can be concluded that,

1. Unreinforced HSS-to-HSS moment connections lack the ability to develop the beam moment capacity due to undesirable column face plastification even for large β values.
2. Reinforced HSS-to-HSS moment connections are able to develop the plastic moment of the beam member with normalized moment capacity greater than unity for all values of β. Increasing values of β for the reinforced connections lead to small decreases in the moment capacity of the connection, while a t_b/t_c ratio of 0.6 is optimal in terms of maximizing the moment capacity of the reinforced connections. An increase in plate length leads to a minor increase in moment capacity.
3. Load carrying capacity of the joint and the maximum deformation capacity is highly sensitive to the type of connection used
4. Performance of the joint with connection details of columns jacketed with two channel sections and connected with beam by welding performs best among others with respect to the ultimate load, deformation at collapse and formation of plastic hinges

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