An Overview of Experimental Research on the Welded Joints between No-internal Diaphragm Square/Rectangular Tubular Columns and H-Beams

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Abstract: Plentiful studies are reported on the semi-rigid behavior of joints between open section H shaped beam and column, but researches focusing on the semi-rigid performance of joints between square/rectangular hollow section (SHS/RHS) column-to-H shaped beam is to lag a lot. Most studies shows that when the SHS/RHS column connecting the H beam without internal diaphragm, the joints behavior is semi rigid in most cases. The details of welded connection between SHS/RHS column-to-H beam without internal diaphragm is diverse, and factors influencing the properties are complicated. This paper reviews the current research situation about such type of connections. The classification of connections is conducted basing on the main detail characteristics. The capacity research status of different detail connections is analyzed and summarized. The aspects for further research are presented.

Keywords: Beam-column joint, bearing capacity, SHS tubular column, welded semi-rigid connection.

1. INTRODUCTION

Comparing with traditional H-shaped steel frame column, the slender proportion of closed section column under the same effective height is much smaller than that of open section H-shaped column because of that the turning radius (especially turning radius of minor axis) of closed section is bigger than that of the same sectional area opening H-shaped section. At the same time the residual stress distribution mode of closed section is more favorable than the opening section ones. These factors result in the higher compressive capacity of closed section column than that of H-shaped column. Based on these advantages of the section performance, steel frame system of square/rectangular steel tubular (SHS/RHS) column-H shaped beam attracts more and more attention of engineers and researchers.

In the frame system of square/rectangular steel tubular column to H-beam, beam-column stiffness connections (inner diaphragm connection, diaphragm through type connection) are the common connection types. Considering the difficult of diaphragm construction, in recent years, along with continuous development and perfection of steel structure semi-rigid connection theory, the correlative researches about the no-internal diaphragm semi-rigid connections between SHS/RHS columns and H shaped beams have also made great development. The objective of this paper is to report and review some of typical experimental research about this kind of joints, to analyze and summarize the research status of joints' bearing capacity of different detail connections, At the same time, the classification of joints is investigated according to the main structural feature of the connections, and some problems for future research are pointed out.

2. THE CONNECTION DETAILS OF SEMI-RIGID WELDED JOINTS BETWEEN SHS/RHS COLUMN AND H-BEAM WITHOUT INTERNAL DIAPHRAGM

The connection of SHS/RHS column to H shaped beam without internal diaphragm offer huge convenient to joints' construction. But without the diaphragm to transfer the beam flange's pull and pressure, the stiffness of steel tube wall out of plane is very weak, and the tube wall's deformation becomes an important influence factor for joint deformation. According to the classification principle of Eurocode 3 for beam-column joints [2], the performance of such a joint shows semi-rigid in most cases. This is the most basic feature for no-internal diaphragm welded joints between SHS/RHS columns and H-beams. The specific configuration of this kind of joints is various. Summarizing the existing open analytical research data, it can be generally divided into three forms: 1) welded connection of beam-column without strengthening; 2) welded connection of column face strengthening; 3) welded connection of beam-end strengthening.

2.1. Welded Connection of Beam-column without Strengthening

This kind of connection details is mainly used for middle-small pipe size's column-H shaped beam connection and applied widely in multilayer steel frame house and offshore structures. The basic structure layout is showed in following Fig. (**1**), beam flange and column face adopt butt welding and beam web and column face adopts double-sided fillet weld connection.

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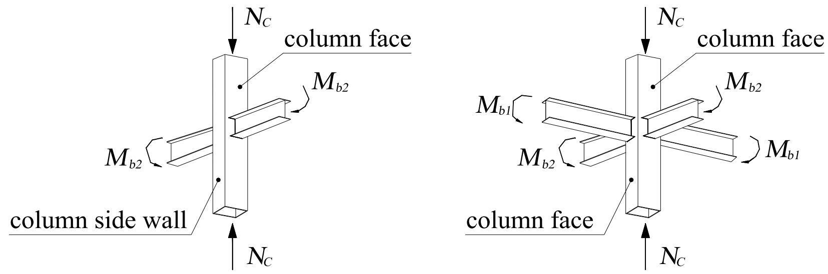


(a) in-planar connection (b) multi-planar connection

Fig. (1). welded connection of beam-column without strengthening.

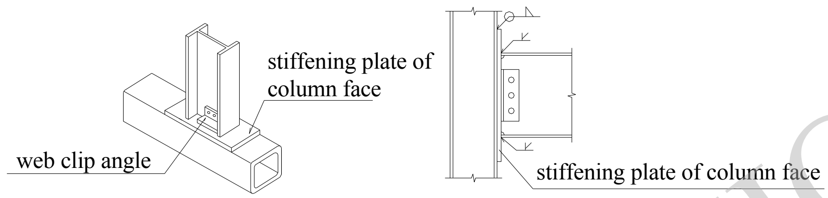


Fig. (2). Column face stiffened connection.

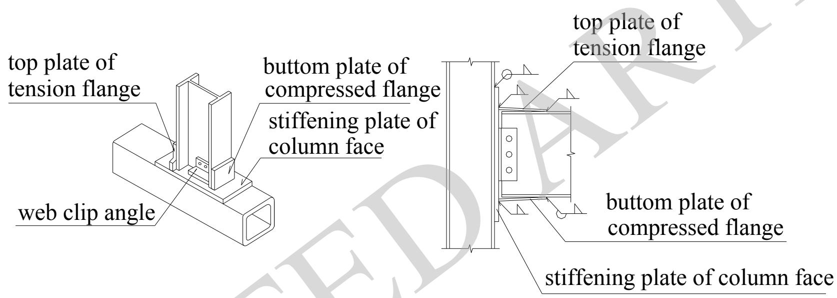


Fig. (3). column face stiffened top and bottom cover plate type connection.

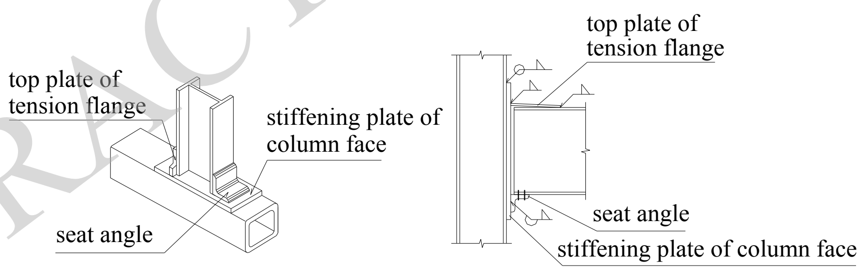


Fig. (4). Column face stiffened top plate and seat angle type connection.

2.2. Welded Connection of Column Face Strengthening

Out-plane stiffness of SHS/RHS column face without internal diaphragm is relatively small. In order to enlarge the out-plane stiffness of column face plate, a stiffening plate may be set to the face plate. The beam-end shear force of such a joint is usually transferred through web clip angle as shown in Fig. (**2**)-Fig. (**4**). The transference of beam-end bending moment is completed through the following three configuration types:

Beam flange and the stiffening plate of column face adopt butt weld connection, and fillet weld is used to connect the stiffening plate and column face. as is shown in Fig. (**2**);

A couple of cover plates are set to the tensile and compressive flange of beam, respectively, which connect the stiffening plate and beam with fillet weld (top and bottom cover plate type, as shown in Fig. (**3**));

At the place of tensile flange of beam, a piece of tension cover plate is set to connect the stiffening plate and tensile flange of beam. While, at the place of compressive flange, a piece of seat angle is used to connect with this two parts, as is shown in Fig. (**4**). (top plate and seat angle type).

2.3. Welded Connection of Beam-end Strengthening

As to the steel frame having the requirement of earthquake fortification, beam-end enhanced connection can easily realize the shifting of plastic hinge away from column face, which is an ideal joints construction type. Analyzing existing research data, there are three typical beam-end strengthened types: 1) beam-end T type steel strengthened (as shown in Fig. (**5**)); 2) beam-end angle strengthened (as shown in Fig. (**6**)); 3) beam-end flat plate strengthened (triangle plate, column outer ring plate) (as shown in Fig. (**7**)). The first two types are strengthened with vertical ribbed plate, the last is a strengthened type with level ribbed plate. About beam-end enhanced welded connection, H-beam web plate and column face plate usually adopt double-sided fillet weld connection, and the other welds adopt equal strength butt welding connection.

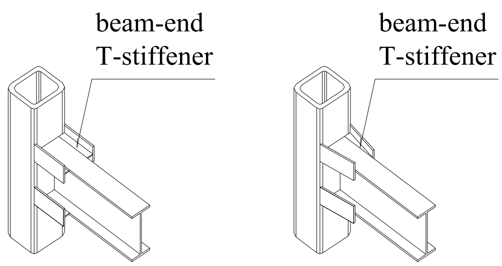


Fig. (5). Beam-end T type steel strengthened connection.

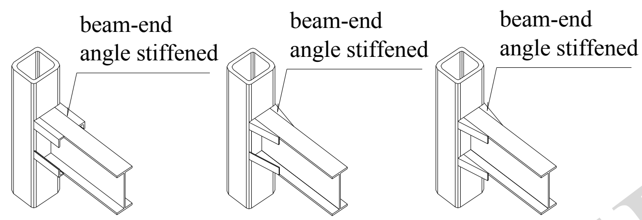


Fig. (6). Beam-end steel angle stiffened connection.

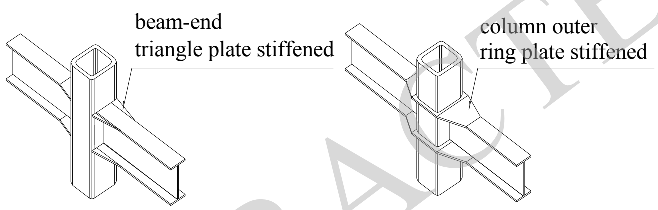


Fig. (7). Beam-end flat plate stiffened connection.

3. TYPICAL EXPERIMENTAL RESEARCH ON THE PERFORMANCE OF SEMI-RIGID WELDED JOINTS BETWEEN NO-INTERNAL DIAPHRAGM SHS/RHS COLUMN AND H-BEAM

Analyzing the existing experimental research, it can be seen that there are usually two types research methods in the process of analysis and research of the performance of no-internal diaphragm welded connections between H-beams and SHS/RHS columns: the first one is that, taking the whole joint as the experimental research object, the static and dynamic performance of joints can be studied, and the other one is that, taking the welded joint assemblage between no-diaphragm SHS/RHS column and flange plate of H-shaped beam as the experimental research object, the stress distribution and deformation of flange plate of beam and column face are investigated firstly, then the static performance of the beam-column joints can be obtained through comprehensive analysis.

3.1. Typical Experimental Research and Analysis about Welded Connection of Beam-column without Strengthening

The research of this kind of joints has begun in the 1960s at the earliest. Many researchers in Canada, Australian, Holland and Japan have performed a lot of research work under the direction of ClDECT. Some relative experimental studies have also done in China recently.

3.1.1. Typical Experimental Research of Branch Plate (Flange Plate of Beam)-to-Column Face Unstiffened Weld Connection

To understand the performance of unstiffened welded joints between H-beam to SHS/RHS column, it is an effective approach that investigating the behavior of flange plate of beam to column connections firstly and then analyzing the performance of the whole joint synthetically. There are a lot of experimental research results about such a kind of connections now. Several typical experimental research as follows:

Rolloos [3] conducted some experimental studies on the behaviors of connections between axial tension transverse plate and square hollow section(SHS) column. The width of transverse plate was equal to that of column face for all the test specimen. The test results indicate that the stress distribution of the plate is extremely uneven, and the concept of effective width was presented for transverse flange plate, the equations for calculating effective width were put forward based on the connection details.

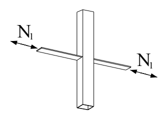
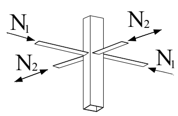
In Rolloons' tests, the influence of width ratios (transverse plate to column face) β was not considered. In order to determine the effective width of flange plate for H-beam to SHS column connections, welded plate to SHS column connections for various width ratios β and various column slenderness 2γ (ratios of the width of column to thickness) have been investigated by Wardenier and Davies *et al.* [4-6] Based on the test data and theoretical analyses, it was indicated by Wardenier [6] that design of these connections can be based on the effective width criterion, the column punching shear criterion and the column wall bearing criterion. The strength formulae (as is shown in Eq.1.) have been established by Wardenier and further adopted in the CIDECT design guide [7]. Eq.2 is the expression formula of effective width of transverse flange plate.

$$P_u = f_{1y} \cdot \left(b_e t_1\right) = 13.5\beta f_{0y} t_0^2 \tag{1}$$

$$b_e = \eta_1 b_1 = \frac{13.5}{b_0 / t_0} \cdot \frac{f_{0y} t_0}{f_{1y} t_1} \cdot b_1 \tag{2}$$

Where η_1 is effective width coefficient, β is width ratio of transverse plate to column face($\beta = b_1/b_0$), b_e , b_0 , t_0 and f_{0y} are, respectively, effective width of transverse flange plate, column face width, thickness and steel yield strength, b_1 , t_1 and f_{1y} corresponding to the width，thickness and steel yield strength of transverse flange plate(as shown in Fig. (**8**)).

Table 1. Bearing capacity analytical expressions of flange plate to SHS column welded joints [10].

Stress mode		Calculation of bearing capacity
	(1) axially loaded plate to SHS column	Chord face yielding： $N_{1,u} = (0.5+0.7\beta)\dfrac{4}{\sqrt{1-0.9\beta}} f_{y0} t_0^2$ (3) Chord side wall failure： $N_{1,u} = 2(t_1+5t_0) f_{y0} t_0^2$ (4) Apply to： $0.2 \le \beta \le 1$， $15 \le 2\gamma \le 37.5$
	(2) axially loaded multi-planar plate to SHS column $J = N_2/N_1$	$N_{1,u} = f(J)(0.5+0.7\beta)\dfrac{4}{\sqrt{1-0.9\beta}} f_{yo} t_o^2$ (5) 当 $J \ge 0$， $f(J)=1$；当 $J<0$， $f(J)=1+(0.22+0.22\beta)J$ Apply to： $0.2 \le \beta \le 0.75$， $15 \le 2\gamma \le 37.5$，

Note: The parameters in Table **1** refer to the following Fig.(**9**).

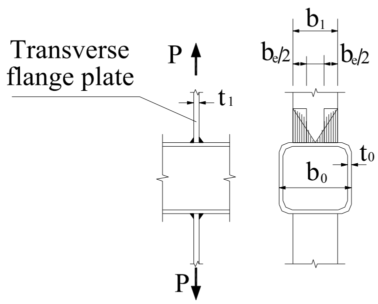


Fig. (8). relative parameter of Wardenier' experiment.

The above experiments only took one way flange plate bearing tension into account. In order to investigate the interaction effect of bidirectional loading, further research work has been carried out for connections between transverse flange plates and SHS columns. Lu and Wardenier (1995, 1997) [8-10] have analyzed a large number of unstiffened connections between transverse flange plates and SHS columns using experiments. During the experimental research, the case of flange plate loaded with compression and tension was investigated respectively. Through the experimental studies, the failure modes and corresponding bearing capacity of the connections have been obtained, and the strength formulae of such a kind of joints was put forward. Table **1** summarizes the bearing capacity analytical expressions of joints corresponding to different stress models.

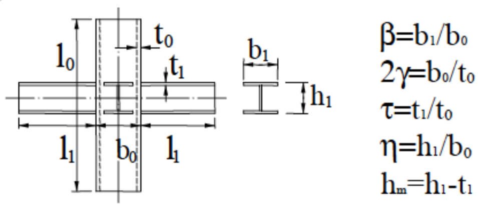


Fig. (9). Relevant parameter definition of Lu,L.H. analytical expression.

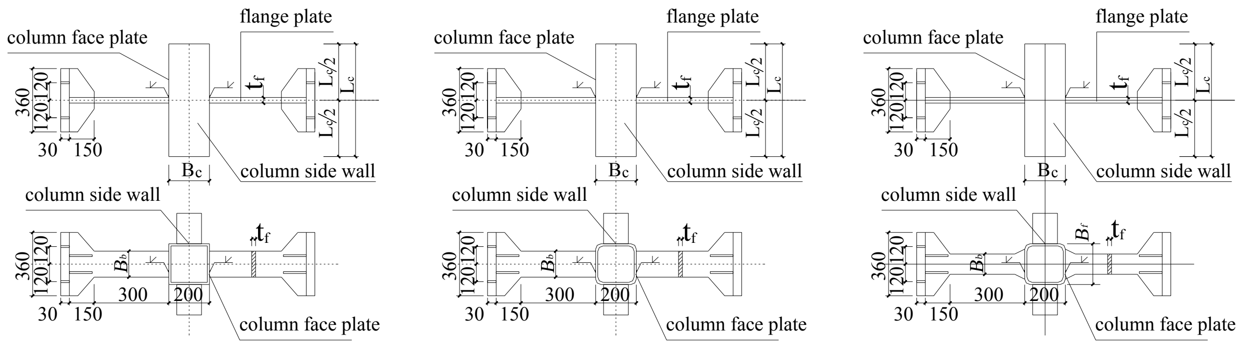
In order to investigate the effect of forming method of column section on the connection behavior, 3 types of joints between tension flange plates and no-internal diaphragm SHS/RHS columns were tested in Tongji University [11]. The column section included cold-formed SHS column and welded box-section column. Two types of flange plate (equal section plate and expanded end section plate) were considered (as is shown in Fig. (**10**)). The research results indicate that, the bearing capacity and stiffness of cold formed section specimens are higher than the welded box-section one, and when the end of flange plate is tapered the average stress of the flange plate will be higher. The experimental results of joint bearing capacity have been compared with the empirical formulae which was proposed by Kato B., *et al.* (1981) [12], and Kamba T, and Tabuchi M. (1994), [13], to predict the ultimate connection strength of plate to column connections. It was found that the measured values is higher than the theoretical ones.

3.1.2. Typical Experimental Research of Unstiffened Weld Connection between HSS Columns and H-beams

The static behavior experiment of joints between H shaped beams to unstiffened SHS/RHS columns was first conducted out in Japan [14]. During the experimental research, a total of 10 H-beam to SHS column T-joints loaded by in-plane bending moments have been tested. The research results indicated that generally four local failure modes, i.e. column face yielding, column side wall failure, local buckling of beam flanges and punching shear, can be expected for this type of connection. The relationship between the failure modes and the dimensional parameters of the specimens has been investigated. The experimental results of joint ultimate flexural capacity have been compared with a empirical formula (as shown in Eq.6). The results showed that for $\beta = 1.0$, the empirical formula are applicable, but for $\beta < 1.0$, the strength of the H-beam connections is overestimated by that formula, and β is width ratio of H-beam flange to column face.

$$M_{max} = 3.85 f_{u1y} \left(\frac{T}{B}\right)\left(\frac{t}{B}\right)^{2/3} B^2 \cdot (d-t) \quad (6)$$

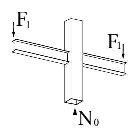
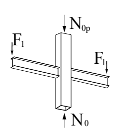
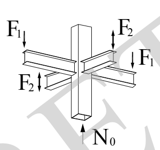
Where, M_{max} is the ultimate flexural capacity of joint, T, B, t, d and f_{u1y} are respectively, the thickness and width of SHS column wall, the flange thickness and section depth of H-beam, and the ultimate strength of beam flange.



(a) BP-type (b) RP-type (c) RPh-type

Fig. (10). Types of specimen and relative details for researches in Tongji University (2011).

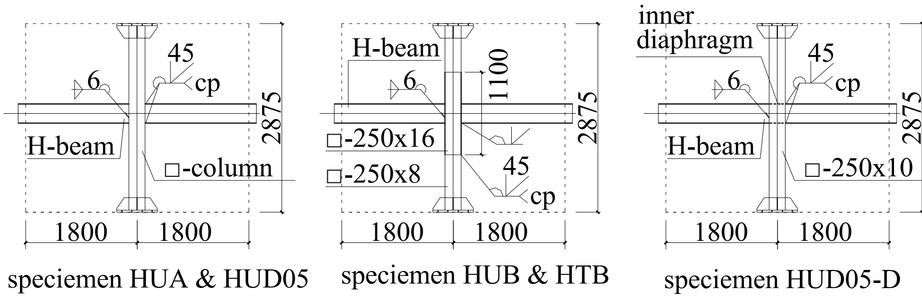
Table 2. The static strength analytical expressions of H-beam to SHS column welded joints established by Lu [10].

Stress mode		Calculation of bearing capacity
	(1) loaded by in-planar bending moment	Chord face yielding： $M_{1,u}=(0.5+0.7\beta)\frac{4}{\sqrt{1-0.9\beta}}f_{yo}t_o^2h_m$ (7) Chord side wall failure： when $h_1\geq 2t_1+5t_0$: $M_{1,u}=2(t_1+5t_0)f_{yo}h_m$ (8) when $h_1<2t_1+5t_0$: $M_{1,u}=0.5(t_1+5t_0)^2f_{yo}t_0$ (9) Apply to： $0.2\leq\beta\leq 1$， $15\leq 2\gamma\leq 37.5$， $0.3\leq\eta\leq 2.0$
	(2) considering the effect of axial compression ratio; $n=N_{0p}/N_{0pl}$, $N_{0,pl}=A_0f_{y0}$	$M_{1,u}=f(n)(0.5+0.7\beta)\frac{4}{\sqrt{1-0.9\beta}}f_{yo}t_0^2h_m$ (10) If $n>0$ （column tension）: $f(n)=1$; If $n\leq 0$ （column press）: $f(n)=1+1.48(2\gamma)^{-0.33}n-0.46(2\gamma)^{(0.33-0.1\beta^2)}n^{1.5}$ but $f(n)\leq 1$ (11) Apply to： $0.2\leq\beta\leq 0.9$， $15\leq 2\gamma\leq 37.5$， $0.3\leq\eta\leq 2.0$
	(3) multi-planar I-beam to SHS column loaded by in-planar bending moment $J=F_2/F_1$	$M_{1,u}=f(J)(0.5+0.7\beta)\frac{4}{\sqrt{1-0.9\beta}}f_{yo}t_0^2h_m$ (12) If $J\geq 0$， $f(J)=1$; If $J<0$， $f(J)=1+0.4J\leq 1.0$ Apply to： $0.2\leq\beta\leq 0.75$， $15\leq 2\gamma\leq 37.5$， $0.3\leq\eta\leq 2.0$

Note: the meanings of the parameters in the table is shown in Fig. (**9**)

In the above-mentioned experiments, the joints was only loaded by in-plane bending moments, and the effect of axial compression ratio of column was also ignored. In order to further study the behavior of welded joints between H shaped beams and unstiffened SHS/RHS column, Wardenier.J (1993) [15], Lu,.L.H., *et al.* (1993-1997) [10],[16]~[20] conducted a series of experimental researches. All of the test joints were applied by static load, and the major studies parameter included: axial compression ratio, ratio of beam depth to width of column face (η), width ratio of beam flange to column face(β),thickness ratio of beam flange to column face(τ), width-to-thickness ratio of column face(2γ). Through those experimental studies, the failure models of such kind of joints and the ultimate strength corresponding to differential failure models were obtained. Based on the extensive experimental studies and finite element analysis, the ultimate strength expression formulae have been established by Lu. Table **2** summarizes the bearing capacity analytical expressions of the joints corresponding to different stress models.

In order to research the hysteretic behavior of non-diaphragm joint connecting SHS/RHS column and H-shaped beam, 5 specimens (including 1 inner diaphragm joints) were tested under hysteretic load in Tongji University [21]. Five columns were all cold-forming section, and the beams were hot rolled H shaped steel(as shown in Fig. (**11**)). The



speciemen	beam	column
HUA	H350X125X6X8	□ 250x16
HUD05	H350X175X6X8	□ 250x10
HUB	H400X150X6X8	□ 250x16/8
HTB	H400X150X6X8	□ 250x16/8
HUD05-D	H400X150X6X8	□ 250x10

Fig. (11). types of specimen and sections of beam and column for researches in Tongji University (2012).

research results showed that, the joints possessed with stable energy dissipation capacity with failure modes of both beam and joint zone yield, and the yield capacity of the joints with and without internal diaphragm are similar, the punch shear failure of tube wall will be produced when the joints is composed of tube with large width-thickness ratio and beam with large width ratio.

3.2. Typical Experimental Research and Analysis about Welded Connection of Column Face Strengthening

3.2.1. Experimental Research of Connections between Transverse Flange Plate and Face-reinforced SHS/RHS Column

The bearing capacity and deformation performance of connection between transverse flange plate and face-reinforced SHS/RHS column should be different from those of unstiffened plate-column connection. To investigate the behavior of such kind of joints, Guravich, S. J. and Dawe, J.L., (1992,1993) carried out a series test studies [22] [23]. The column face of all the joints reinforced with a strengthening plate fillet-welded all around (as shown in Fig. (**12**)).

In these experiments, the connections in tension and compression were investigated respectively. The mainly study parameters included: edge distance(c), thickness (t_b)、width(b_b) of branch plate and the thickness of reinforcing plate(t_p) (see Fig. (**12**)). The test results indicate that among all the parameter investigated, the reinforcing plate thickness is the one that most significantly affects the performance of joint. The research also shows that: 1) As to tension joints, the width ratio of transverse flange plate to reinforcing plate of column face is the important influencing factor of joints performance, and joints failure mode mainly shows punching shear of reinforcing plate; 2) As to compression joints, the thickness of reinforced plate and column face plate is the important influencing parameter of joints performance. The main failure mode is column side wall buckling. Based on the above experimental studies, the bearing capacity calculation formulae of these two kinds of connection joints were established:

tension ultimate bearing capacity：

$$P_u = \frac{2}{3} f_{yP} t_P \cdot \left(b_b + t_b\right) \tag{13}$$

press ultimate bearing capacity:

$$N_u = 2 f_{0y} t_0 \cdot \left[w_s + 2n\left(t_0 + t_p\right)\right] \tag{14}$$

Where, f_{yP} and t_P are respectively yield strength and thickness of reinforcing plate, f_{0y} and t_0 are respectively the yield strength and thickness of column face, w_s is the thickness of compression flange plate including fillet weld size, n is a constant confirmed by the experiment, as is shown in Fig. (**12**).

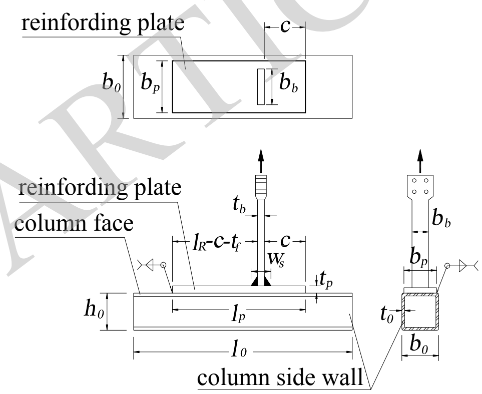


Fig. (12). Sketch map of parameters in Dawe,J.L.'s experimental research.

3.2.2. Experimental Research of No-internal Diaphragm Weld Connections between H Shaped Beam and Face-stiffened HSS Column

The relative details of no-internal weld connections between H shaped beam and face-stiffened HSS column is shown in Fig. (**2**), (**3**) and (**4**). In order to investigate the performance of such kind of joints, Dawe, J.L. and Grondin, G.Y. [24, 25] (1985,1990) performed unidirectional load test on two types of 10 joints. 9 of them is no-internal diaphragm weld joints between H shaped beam and face-stiffened HSS column(including top-bottom cover plate connection and top plate and seat angle connection), the rest one is face-unstiffened joint between H-beam and HSS column. The research results indicate that the column face set strengthened plate improves the joints performance and the connection can reach the plasticity bending strength of beam, meanwhile, the column face will not occur oversize deformation. Combined with the main failure mode of the beam-column joints test specimen (8 types), semi-empirical formula to predict the failure mode and corresponding moment capacities is put forward.

Considering the effect of axial force on joints performance has not be considered in Dawe, J.L.'s experimental study (1985), afterwards, Dawe, J.L. and C.K. Liew(1986)

Table 3. Failure mode of column flange strengthened connection joints and the corresponding bearing capacity analysis of various failure modes.

Item	Failure mode	the corresponding bearing capacity
1	Snap of beam tensile flange cover plate	$P_{u1} = f_u A_{ft1}$ (15-1)
2	Snap of beam tensile flange	$P_{u1} = f_u A_{ft}$ (15-2)
3	buckling of beam press flange cover plate	$P_{cr} = \sigma_{cr} A_{fc1}$ (15-3)
4	Web lateral deflection (distribution of shear stress indicated as Fig. (**13**))	$R_{\max} = (2/3) f_y N w$ (15-4)
5	Punching shear failure of column reinforcing plate (distribution of shear stress indicated as Fig. (**14**))	$P_{u2} = \frac{2}{3}\left[T_R \tau_u (w_0 + w_s) + (1+2\alpha)\right]$, (15-5) $\alpha = \left(\frac{b_R - b_T}{b_R}\right)^3$, $\tau_u = \frac{f_u}{\sqrt{3}}\left[1 - \frac{(b_R - b_T)^2}{b_R^2 - 3t_R^2}\right]^{1/2}$
6	Punching shear failure of column flange (distribution of shear stress indicated as Fig. (**15**))	$\tau_{xc} = \tau_c (1 - c_0 f_b)$ (15-6) $f_b = x_c^4 / EI_b$
7	Column web buckling (distribution of web stress indicated as Fig. (**16**), θ=25^0)	$\sigma_{\max} = \begin{cases} \sigma_L + \sigma_{uD} & L < 2b \\ \sigma_C + \sigma_{uD} & L \geq 2b \end{cases}$ (15-7) $\sigma_{uD} = \frac{\tau_c b_R (1+2\alpha)}{6L}$ (15-8)

[26] conducted further research. They conducted monotonic loading experimental study on column face strengthened connection joints, which belongs to the kind of top plate and seat angle connection, a total of 4 groups, 14 specimens, to examine the influence of the thickness of reinforcing plate and HSS wall, and the axial compression ratio of column on joints performance. The research results indicate that: 1) The column axial compression ratio is a important influential factor for the performance of joint. When the ratio is less than 0.5, axial pressure has very little effect on the rotational stiffness of beam, but if axial compression ratio is more than 0.5, axial pressure will significantly enlarge the joints' rotation capacity. 2) The increase of thickness of reinforcing plate may improve the bearing capacity of joints but lower the rotation capacity of joints.

Based on the experimental analysis Dawe,J.L. proposed the stress distribution model of joints' components(see Fig. (**13**)-Fig. (**16**)), and the corresponding assessment methods of bearing capacity were also presented according to failure mode which may appear(see Table **3**). Finally he conducted a whole analysis on joints' bearing capacity.

In the above analytical expression, A_{ft1} is the cross sectional area of tension flange cover plate, f_u is steel strength of extension, A_{ft} is the cross sectional area of tension flange, A_{fc1} is the cross sectional area of beam press flange cover plate, f_y is yield strength of steel, I_b is the equivalent inertia positive combined by reinforcing plate and the leg of the angle steel.(see Fig. (**17**)), c_0 is a constant which coordinate the dimension on both sides of the formula, σ_{cr} is stable critical stress of freedom of press cover plate's two opposite sides fixing the rest two sides, τ_c is the max shear stress of compressive zone, or steel shear strength under invalid condition. The rest parameters are respectively indicated in the corresponding stress distribution figures.

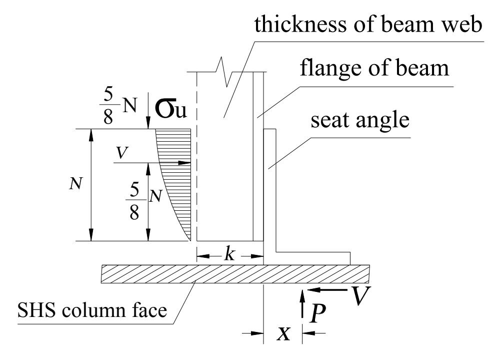


Fig. (13). Distribution of web stress.

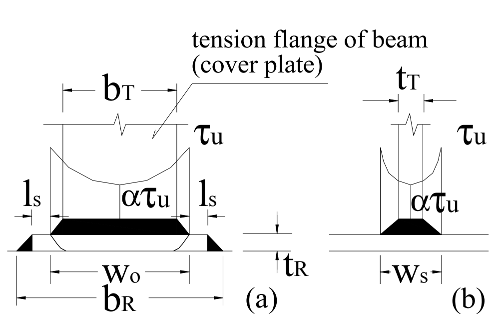


Fig. (14). Peripheral shear stress distribution of tension flange(cover plate).

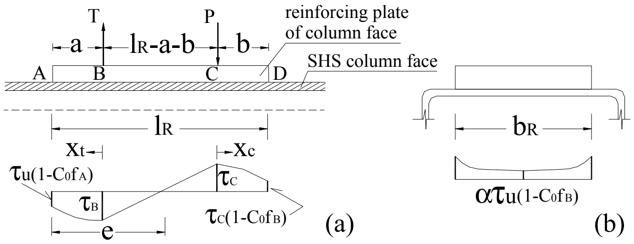


Fig. (15). Peripheral shear stress distribution of reinforcing plate.

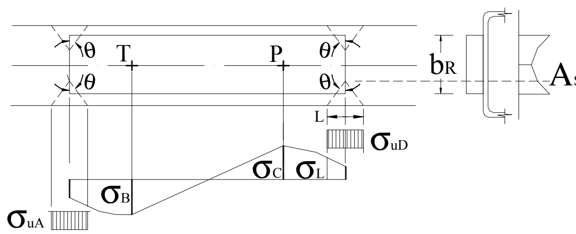


Fig. (16). Column web stress distribution.

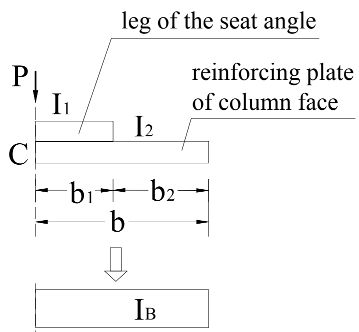


Fig. (17). Equivalent inertia moment for the pressure zone of reinforcing plate.

3.3. Typical Experimental Research and Analysis about Welded Connection of beam-end Strengthening

Tabuchi, M. *et al.* [27] (1988) conducted experimental study on the joints performance of 9 groups under reciprocating loading. All of the joints belongs to no-internal diaphragm welded connection joints between hollow section tube column and H-beams, and the beam-end was strengthened with ring plate (see Fig. (**18**)). The forming way of the column section included two types as follows: cold-formed section for SHS columns and welding composite section for RHS columns. The experiment has examined the effect of column radius-thickness ratio （D/T) and axle load ratio on the joints' performance and failure mode under the lateral load effect. Aiming at two kinds of different failure modes (shear failure of joints area and connection partial yield failure), the corresponding bearing capacity expression were proposed:

Shear invalid of joints area, beam-end bending moment:

$$M_{by} = f_v V_p \sqrt{1-\left(\frac{N}{N_y}\right)^2}\,\frac{1-\lambda}{1-\lambda-\mu} \tag{16}$$

For column flange partial yield failure, beam-end bending moment:

$$M_{by} = 2.23 f_{u1y}\left(\frac{T}{D}\right)^{2/3}\left(\frac{t_d}{h_d+T}\right)^{2/3}\left(\frac{h_d+T}{D}\right)D^2\cdot D_B \tag{17}$$

In above expression, N/N_y is the column axial compression ratio, V_p is the volume of joints area, f_v and f_{u1y} are respectively steel shear strength of joints area and the steel tensile strength of outer ring plate. The rest parameters are indicated in the following Fig. (**18**).

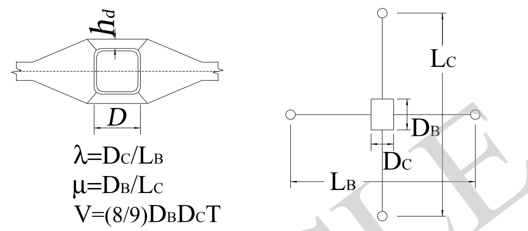


Fig. (18). Construction of outer ring plate enhanced joints (Tabuchi, M.1988).

Aiming at a series of different kinds of strengthening ways for beam-end, N.E.Shanmugam, L.C.Ting, and S.L.Lee [28-35] conducted a wide range of test study and analysis. The strengthening ways included: the type of T stiffening rib plate, the type of steel angle brace and the type of triangle panel brace. According to the comparative analysis, they studied the effects of arrangement situation of different stiffening ribs on joints performance. The research indicates that: 1) After beam-end set with stiffening rib, the compound section breadth of beam flange and stiffening rib plate is equal with column flange width, so the beam flange stress can transmit preferably to SHS column's web and the joints performance gets great improve; 2) Joints failure mode mainly shows shear yield of stiffening rib and afterwards buckling of column web or beam compression flange (shown in Fig. (**19**)) ; 3) Among several kinds of beam-end strengthened constructions, the joint performance of T shape bracing constructions is the most ideal one. Based on the experimental analysis, a simplifying design formula was presented to confirm stiffening rib dimension:

$$\frac{T_p}{2} = l t_{sw}\tau_y + h t_c \tau_y \tag{18}$$

In above formula, t_{sw} is the thickness of stiffening rib web, t_c is the thickness of SHS column wall, τ_y is shear yield strength of column, the rest parameters is shown in Fig. (**19**). In this figure, the angle θ is the internal diffusion angle for beam flange, the research indicated that: when $\theta = 20^o$, the joints' performance is the best.

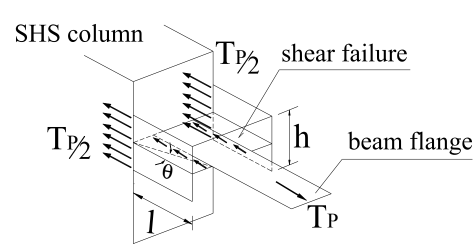


Fig. (19). internal force transmittal mode of T type stiffening rib joints.

4. DETERMINATION OF JOINTS' ULTIMATE BEARING CAPACITY

Ultimate flexural capacity of joints M_u is the most important parameter in the study of semi-rigid connection joints performance of steel frame. Only after determining the joints' ultimate bearing capacity, the design resistance of joints could be confirmed further. It is also an important parameter to confirm joints analytical model.

Generally speaking, the ultimate bearing capacity of joints is different for diverse failure mode of the joints. Basing on the summary of existing experimental study results, the total failure modes of such kind of joints (no-internal diaphragm welded joints between SHS/RHS columns and H-beams) can be divided into the following three types: 1) Local buckling of component parts of joint lead to the decline of joint's moment-rotation curves; 2) Emergence of brittle failure for joints, such as weld fracture or delamination of connecting plates;. 3) The obviously decline is not appeared for the moment-rotation curve of joints, but the experiment is ended because of the overlarge joints deformation. The ultimate bearing capacity of the former two types can be confirmed very easily, but for confirming the third ultimate bearing capacity, different tolerable limit state would appear different ultimate bearing capacity. Considering the feature of easy to be deformed for column face, and the characteristics of stressed-skin effect for closed section, the third failure mode is even more common.

Aiming at the tolerable limit state of the third kinds failure model, Yura, J.A, [36]、Korol an Mirza [37]、Lu,L.H. [38], Zhao,X.L. [39] *et al.* carried on some analysis and research, respectively. In China, the allowable deformation standard of the Lu,L.H.'s is mostly adopted, which takes local deformation reaching 3% b_0 (b_0 is the width of column face) as the allowable deformation of this kind of joints. The ultimate bearing capacity of joints is shown in Fig. (**20**), in which ϕ_c^3 is the joint rotation when local deformation of column face reaches 3%b_0, and M_u^1、M_u^2、M_u^3 are the ultimate bearing capacity of above corresponding three types of failure mode, respectively. It should be noted that, for different application conditions, the limit allowable deformation of joints can be different. For example, when the structure has good ability of deformation, joints resistance exceeding normal ultimate capacity (M_u^3) could be adopted as the design value of bearing capacity. It is a problem worthy of further study that how to adopt the most reasonable value.

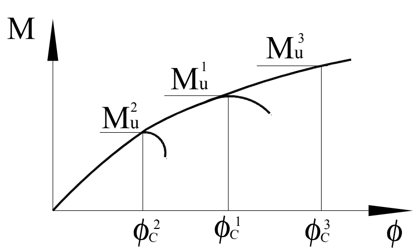


Fig. (20). Joints failure mode and the ultimate bearing capacity.

5. PROSPECT AND THE DIRECTION FOR FURTHER STUDY

With the continuous popularization and application of square steel pipe column in construction industry, especially the popularization in steel structure housing construction, the frame system of RHS/SHS column and H-beam will become more and more popular. And the joint's form of SHS (RHS) column and H-beam without internal diaphragm welded connection will become more and more popular too. Though many experimental studies have been conducted aiming at the performance research of this kind of joints, there are still some problems and further research is needed for the promotion of the joints.

1) Analyzing all the above experimental work, we can see that, an absolute majority research work focus on the static performance of joints, and much less attention has been paid to the studies on the joints' dynamic performance, specially for the connection types of beam-column without strengthening and column face strengthening. If this kind of connection joints is adopted in steel frame system, the joints performance under reciprocating load should be researched. For these kinds of welded connections, whether the welded joint produces brittle rupture under reciprocating load, and the joints dissipation capacity, etc. which need further experimental study.
2) The performance of connections between SHS（RHS） column and H-beam without internal diaphragm mostly shows the semi-rigid performance. For these kinds of connections, besides the bearing capacity, the initial rigidity and ultimate rotation capacity are the other two important performance parameters. For these two performance parameters, related research is not too much, and there is much more left to do.

6. SUMMARY

Welded joint between no-internal diaphragm RHS/SHS column and H-beam is becoming a more and more popular joint structure. After summarizing the constructions classification of these kind of joints, the present review paper reports some typical experimental studies carried out by various researchers in the same field. In the end, some research works are put forward which still need to be carried out.

CONFLICT OF INTEREST

The authors confirm that this article content has no conflict of interest.

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