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Effects of RHS face deformation on the rigidity of beam-column connection

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Abstract. The rigid connections of I-beams to Rectangular Hollow Sections (RHS) in steel structures usually behave as semi-rigid connection. This behavior is directly related to the column face deformation. The deformation in the wall of RHS column in the connection zone causes a relative rotation between beam end and column axis, which consequently reduces the rigidity of beam-column connection. In the present paper, the percentages of connection rigidity reduction for serviceability conditions are evaluated by using the finite element analysis. Such percentages for RHS columns without internal stiffeners are considerable, and can be calculated from presented graphs.

Keywords: steel frames; rigid-connection; semi-rigid connection; finite element; box column; rectangular hollow section.

1. Introduction

Using Rectangular Hollow Sections (RHS) as columns is very popular. The rigid connection of I beams to RHS column is usually constructed by the welded top and bottom plate as shown in Fig. 1.

When a bending moment is applied to a beam-column connection, a big axial force occurs in top and bottom plates. These forces deform the wall of RHS column, and the connected beam rotates relative to column axis. This relative rotation between beam end and column axis will be released some moment at the connection, which consequently reduces the rigidity of beam-column connections; therefore these connections behave as semi-rigid connections and neglecting such a real behavior of the connection in the analysis and design leads to unrealistic results.

The deformation of RHS face is the function of many parameters such as: thickness and width of RHS column, dimension of connection, existence of internal stiffener, bending moment at connection and the other limited parameters. The effect of these parameters must be considered in designing beam-column connections.

Considerable research has been carried out to define design criteria and actual behavior of RHS column connections. Kato and Mukai utilized yield line analysis to predict the strength of bolted end plate connections of hollow sections subjected to pure tensile loading (Kato and Mukai 1985). Settleco studied the behavior of pinned connection of I beam to RHS columns with stiffeners (Settleco 1981). The design guidelines for connection of the longitudinal plate to columns with hollow section proposed

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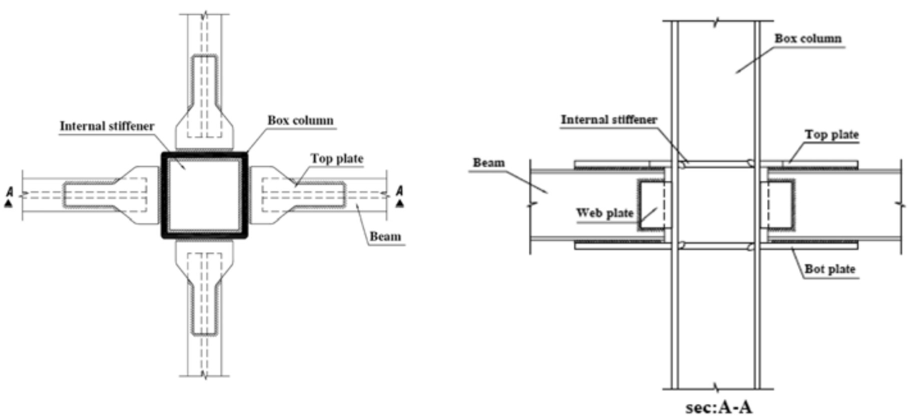


Fig. 1 Rigid connection of I beams to RHS column

by Cao *et al.* (1988). The connection of I-beam to box-column with external stiffener studied by Ting *et al.* (1993). Miura *et al.* presented three full-scale tests of beam-to-RHS column connections without weld access holes. They investigated the plastic deformation capacity and ultimate strength of this type of connection (Miura *et al.* 2001). Han *et al.* experimentally investigated the flexural force-deformation behavior of concrete filled thin-walled steel SHS and RHS beam-columns under cyclic loading (Han *et al.* 2003). Furthermore, research on connection of longitudinal and transverse plate to RHS columns done by Kosteski and Packer (2003). Satish Kumar and Prasada Rao proposed a new RHS beam-to-column connection with web opening. They evaluated the behavior of the connection by cyclic tests and non-linear finite element analysis (Satish Kumar and Prasada Rao 2006). Packer *et al.* studied the static and fatigue design of CHS-to-RHS welded connections using a branch conversion method (Packer *et al.* 2007). Also Bae *et al.* (2009). investigated the behavior and structural resistance of longitudinal double plates-to-RHS connections by experimental and analytical methods (Bae *et al.* 2009). In addition to above researches, extensive experimental and analytical investigations on the behavior and modeling of rigid and semi-rigid beam-column connections have been done. Kishi *et al.* (1993) proposed design aid for analysis of semi-rigid steel frames (Kishi *et al.* 1993). Xu and Grierson (1993) proposed a computer-automated program for designing of semi-rigid steel frames (Xu and Grierson 1993). Hadianfard and Razani studied the effects of semi-rigid connections on the reliability and structural safety of steel frames (Hadianfard and Razani 2003). Also Cabrero and Bayo introduced a practical method for optimum design of semi-rigid steel structures (Cabrero and Bayo 2005).

Most recent researches have been concentrated on the modeling, design, and behavior of the semi-rigid and RHS connections. The effect of wall deformation of RHS column in the rigidity of beam-column connection has seldom been considered.

In this paper, the wall deformation of RHS columns at connection zone, under service loads is calculated using finite element analysis and ANSYS software, and the effect of this deformation in the rigidity of beam-column connection is evaluated.

2. Structural analysis of semi-rigid steel frames

The actual behavior of the connections of I-beams to RHS columns is seldom fully rigid. When a moment M is applied to a beam-column connection, the connected beam and column rotate relative to each other by an amount of θr. The relationship between M and θr, can be shown by the use of an M-θr diagram. This diagram is usually derived by fitting suitable curves to the experimental data. Various types of M-θr models as: linear, polynomial, exponential, power etc. have been developed by different researchers (Chen and Lui 1991). For example, Kishi and Chen's (1990) power model is shown by Eq. (1). This relation can be used for beam-column connection by angles.

$$\theta_r = \frac{M}{R_{ki}\left[1-\left(\frac{M}{M_u}\right)^n\right]^{1/n}} \tag{1}$$

In which R_{ki} is the initial stiffness of the connection, M_u is the ultimate moment capacity of the connection and n is a shape parameter (Kishi and Chen 1990). Another example of M-θr diagram for end-plate connection is the Frye and Morris polynomial model as given by Eq. (2) below

$$\theta_r = C_1(K_0M) + C_2(K_0M)^3 + C_3(K_0M)^5 \tag{2}$$

Here, K_0 is a standardization parameter dependent upon the connection type, and C1, C2, C3 parameters are curve-fitting constants (Frye and Morris 1976).

Also by using curve-fitting procedure, Ting *et al.* proposed some nonlinear M-θ_r relations for connections of I-beam to box-column (Ting *et al.* 1993).

The effects of connection flexibility are usually considered in the structural analysis by attaching rotational springs with stiffness modulus *Ri* and *Rj* at the ends *i* and *j* of the beams. The rotational stiffness of these springs can be calculated at each point from the slope of the M-θ_r curve, and it appears in the stiffness matrix of the beams (Hadianfard and Razani 2003). For example in the Kishi and Chen's power model (see Eq. (1)) the tangent modulus R_{kt} is as following

$$R_{kt} = \frac{dM}{d\theta_r} = R_{ki}\left[1-\left(\frac{M}{M_u}\right)^n\right]^{\frac{1+n}{n}} \tag{3}$$

And for the Frye and Morris polynomial model (see Eq. (2)) the tangent modulus R_{kt} is as following

$$R_{kt} = \frac{dM}{d\theta_r} = \frac{1}{C_1K_0 + 3C_2K_0^3M^2 + 5C_3K_0^5M^4} \tag{4}$$

The initial stiffness of the connections can be obtained from Eq. (3) or Eq. (4) by replacing $M = 0$.

For columns, the stiffness matrix, takes the usual form. The beam stiffness matrix and the column stiffness matrix can be assembled in the usual manner to form the stiffness matrix for a given structure.

As shown in the Eq. (1) and Eq. (2), most of the M-θ_r relations are nonlinear. The nonlinear behavior of the connections is due to a number of factors as: material discontinuity of the different parts of a connection, local yielding of some parts of a connection, stress concentration, local buckling, geometric changes under loads etc. (Chen and Lui 1991). In the nonlinear analysis, the connection stiffness degrades gradually from an initial stiffness, R_{ki} to zero following a nonlinear M-θ_r relationship. Then for calculating the structural stiffness matrix, the tangent stiffness of the connections should be derived from the M-θ_r curve. For simplicity, in the design of the frames under the serviceability limit state, a linear approximation of M-θ_r curve usually is acceptable. But in the design of the frames according to the ultimate limit state, it usually becomes unacceptable.

In the conventional analysis of semi-rigid structures, usually the connections are not considered directly as a structural member, but they are identified as their flexibility or rigidity percentages. Hence, the evaluation of the flexibility percentage of beam-column connections is very important and it is main goal of this research.

3. Serviceability and ultimate deformation of RHS face

The exerted forces on connection, where affect the face of RHS, will deform walls of hollow section. For RHS connections (as shown in Fig. 2) the face deflection of 1% of the section width ($\Delta_s = 0.01b$) has been named as a serviceability deformation limit. The corresponding load P in connection plate, which creates deformation of $\Delta_s = 0.01b$ has been shown as $P_{S,1\%}$. An ultimate deformation limit, in which the connection deems to failing, is the face deflection of 3% of the section width ($\Delta_u = 0.03b$) and the load corresponding to this deflection, shown as $P_{U,3\%}$ (Kosteski and Packer 2003, I IW 1989). The ultimate and the serviceability deformation limits have been used to define the maximum allowable deformation of box section walls under the ultimate or service loads.

Service loads or unfactored loads are used for the calculation of RHS deformations, and maximum deformation is bonded to $\Delta_{max} = 0.01b$, where b is the width of box section. For the range of deformation values between 0 to 0.01b, we assume our structure to be linear and elastic.

4. Modeling of RHS connections by finite element software

To calculate the stress and deformations in the frame members, static linear analysis is used. To fulfill this purpose the structure and its loads are idealized to produce a numerical (finite element) model. Then a finite element software as Ansys is used for analyzing the numerical model. Ansys is a finite element software with capability to analyze a wide range of different problems. There are different types of multi dimensional elements such as 1D, 2D and 3D in this software. One-dimensional elements (1D) represent line shapes, such as beams or springs. Two-dimensional elements (2D) represent triangle and square shapes, such as shells and plates. Three-dimensional elements (3D) represent solid shapes. Also 2D elements are in 3 different types of linear (with 4 nodes), quadratic (with 8 nodes) and cubic (with 12 nodes). The 2D elements can be used for modeling the wide range of structures, such as structures with thin members (for modeling all parts of a thin wall sections).

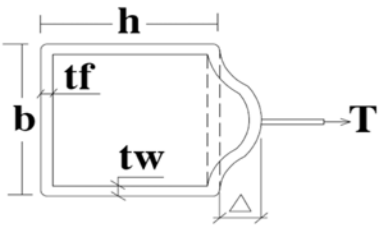


Fig. 2 Deformation of box-column wall

In the present paper different steel frames with RHS columns and I beams that were connected together with rigid connections are modeled by Ansys software. The steel sections of columns, beams and connections have been modeled by using shell-93 elements. This shell element is quadratic and defined by eight nodes, with six degrees of freedom at each node (3 translations and 3 rotations), four thicknesses, and the orthotropic material properties (see Fig. 3). Within this element, the deformation shapes are quadratic in both in-plane directions. The elements have plasticity, stress stiffening, large deformation and large strain capabilities. Size of shell elements have been selected very small, so that the analysis results have sufficient accuracy.

A two story portal frame that is modeled by the aforementioned elements is shown in Fig. 4. The beam to column connection of this frame can be modeled in two methods as shown in Figs. 5(a) and 5(b). In Fig. 5(a) the rigid connection of beam to column is defined by constraining all of the degrees of freedom of the nodes of beam end to column face. In Fig. 5(b); the overall parts of rigid connection is defined by shell elements.

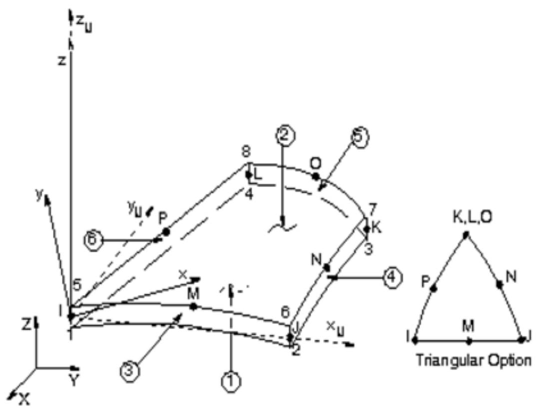
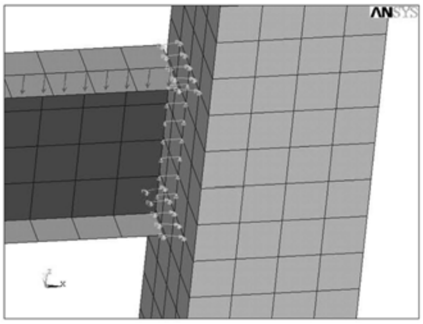


Fig. 3 Shell 93 elements for modeling of members in Ansys software

Fig. 4 Model of two story portal frame in Ansys software



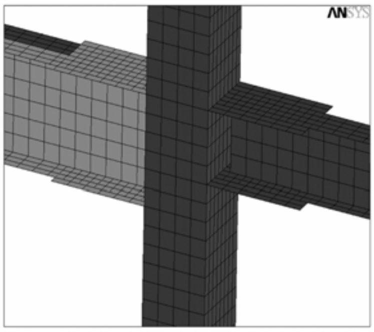


Fig. 5 Model of rigid connection in Ansys software

5. Calculation of stress and deformation in the face of RHS

Different structural analysis has been done on the frames modeled by shell elements. In each analysis the wall deformation of RHS column under the service loads at the connection zone has been calculated. Also the effects of wall thickness of RHS and internal stiffeners, on the column face deformation and stress at the beam ends have been studied. The deformation in the face of box column will cause relative rotation between beam end and column axis. This relative rotation will cause a reduction in bending moment at beam-column connection; hence, decreasing the rigidity of beam to column connection. The reduction of connection rigidity is the direct function of RHS face deformation. The rigidity of beam-column connection can be increased by using internal stiffeners or increasing wall thickness of box column.

The actual bending moment at the beam ends can be calculated by stress obtained from the analysis of steel frames modeled by shell elements (2-D elements) as shown in Fig. 4 and 5. The nominal bending moment can be calculated by analysis of rigid frames modeled by beam elements (1-D elements). By comparing the actual bending moment with nominal bending moment at each beam end, the percent of rigidity reduction in each connection can be calculated. Following, some numerical calculations are presented. In each, the deformation and stress at box column face near the connection zone is calculated. Also the effects of column wall thickness and internal stiffeners are considered in the analysis. Finally the percent of rigidity reduction in each connection is evaluated.

6. Numerical calculations

In this section, some samples of numerical calculations are presented.

6.1 Sample calculations for frame under gravity loads

As a first sample, a one-bay two stories portal frame with rigid connections is considered as it is shown in Fig. 6.

The beams of this frame are under uniformly distributed loads equal to 10 KN/m. The sections of the frame members have been selected based on the primary structural design. The design of the members

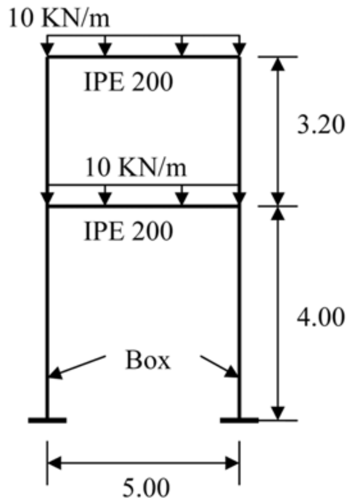


Fig. 6 Rigid frame under gravity loads

under service loads have been done by using the allowable stress design (ASD) method. Then, the stress in the frame members is about 60% of yielding stress and the structure behaves linear and elastic. Therefore, based on the results of the primary design, the depth of beams has been selected d = 20 cm (IPE 200), and the width of columns has been selected b = 20 cm with a thickness of t = 1.2 cm. The nominal stress at the beam ends is calculated from the analysis of the frame by beam elements in the SAP2000 software. For example, the bending moment at the beam ends of the first story is M1 = 18.85 KN.m and at the middle of this beam is M2 = 12.23 KN.m. The axial stress (σ) due to bending moment, can be calculated by equation: $\sigma = M / S$, which M is bending moment and S is section modulus. Accordingly, the stresses at the ends (σ_1) and at the middle (σ_2) of the first story beam are as Eq. (5) and Eq. (6).

$$\sigma_1 = \frac{18.85}{194 \times 10^{-6}} = 97164.9 KN/m^2 = 97.16 MPa \tag{5}$$

$$\sigma_2 = \frac{12.23}{194 \times 10^{-6}} = 63041.2 KN/m^2 = 63.04 MPa \tag{6}$$

The actual stress at the beam ends is calculated from analysis of the frame by shell elements in the ANSYS software. Because of symmetry in the beam cross section, the average of compression and tension stress in the beam cross section is assumed as maximum axial stress. This analysis is done in two ways; one is without and the other is with internal stiffeners in the columns. In the latter (without stiffeners), the stress at the ends of first story beam is $\sigma_1 = 90.85 MPa$ and in the middle of the beam is $\sigma_2 = 66.48$ MPa. Then the difference between nominal and actual stress are as Eq. (7) and Eq. (8).

$$\Delta\sigma_1 = \frac{97.16 - 90.85}{97.16} \times 100 = 6.49\% \ : \textit{At the ends of beam} \tag{7}$$

$$\Delta\sigma_2 = \frac{63.04 - 66.48}{63.04} \times 100 = -5.45\% \ : \textit{In the middle of beam} \tag{8}$$

Also these differences in the former (with stiffeners) are as Eq. (9) and Eq. (10).

$$\Delta\sigma_1 = \frac{97.16 - 94.79}{97.16} \times 100 = 2.4\% \ : \textit{At the ends of beam} \tag{9}$$

$$\Delta\sigma_2 = \frac{63.04 - 63.94}{63.04} \times 100 = -1.4\% \ : \textit{In the middle of beam} \tag{10}$$

The mean of $\Delta\sigma_1$ and $\Delta\sigma_2$ can be used as percentage of rigidity reduction (ΔR) in each connection. Then, the following results will be attained for ΔR:

Without stiffeners: $\Delta R = 5.97\%$ and with stiffeners: $\Delta R = 1.9\%$.

This example indicates that in columns without internal stiffeners, the actual bending moment at the beam ends is about 6 percent less than the ideal case of fully rigid connection. For columns with internal stiffeners this value is about 2 percent. The maximum deformations of the column face near the connection zone are $\Delta = 0.53$ *mm* and $\Delta = 0.13$ *mm* for the cases without and with internal stiffeners respectively. The two values are less than the serviceability deformation limit equal to $\Delta_S = 0.01b = 0.01 \times 200 = 2$ *mm*. Therefore, in the method of allowable stress design, the uniform load on the beams can be increased to a maximum value using the following factors $a_1 = \frac{2}{0.53} = 3.77$ in columns without stiffeners and $a_2 = \frac{2}{0.13} = 15.38$ in columns with stiffeners. The behavior of the frame near the connection zone is nonlinear for uniform loads more than the mentioned values, and maximum deformation of the column face under ultimate loads is limited by ultimate deformation $\Delta_u = 0.03b = 6$ *mm*.

6.2 Sample calculations for frame under lateral loads

In this section, the frame of Fig. 6 is loaded by only two lateral loads at the first and second story level, equal to 10 KN and 20 KN respectively. The beams and columns dimensions are as before. The nominal bending moments and axial stresses in the beams for fully rigid connections are calculated by SAP2000 software (by using beam elements) as below:

At the beam ends of the first story: M = 26.2 *KN.m* and $\sigma = 135.05$ *Mpa*.

At the beam ends of the second story: M = 20.3 *KN.m* and $\sigma = 104.63$ *Mpa*.

Below are the actual axial stresses in the beams calculated by ANSYS software (by using shell elements):

At the beam ends of the first story: $\sigma = 121.82$ *Mpa*

At the beam ends of the second story: $\sigma = 96.75$ *Mpa*.

Thus, the percentage of stress difference between the nominal and actual state is:

For first story beam: $\Delta\sigma = 9.8\%$ and for second story beam: $\Delta\sigma = 7.5\%$.

Accordingly in this case, percentages of rigidity reduction of the connections for first and second story are 9.8 and 7.5 percent respectively. These values are a little more than quantities obtained in section 6-1. We conclude that, the rigid connection of the beam to RHS column, under lateral loads is a little more flexible than under gravitational loads.

6.3 Sample calculations for frame under gravity and lateral loads

In this section a portal frame under gravity and lateral loads as shown in Fig. 7 is considered. Columns are RHS and without internal stiffeners. Two analyses are done, one by beam elements (nominal analysis) and another by shell elements (actual analysis). Results of these analysis and percentage of connection rigidity reduction are shown in Table 1. In this manner, also the rigidity reduction is about 8%.

7. Tables and graphs for calculation of connection rigidity reduction

The amount of reduction in the connection rigidity is related to the deformation of column face at the connection zone. Also it is the function of parameters as: thickness of column walls (t), width of column section (b), depth of beam (d), existence of internal stiffeners etc. As a sample, the effects of the above parameters are considered on the analysis of the frame shown in Fig. 6 (section 6-1) and results

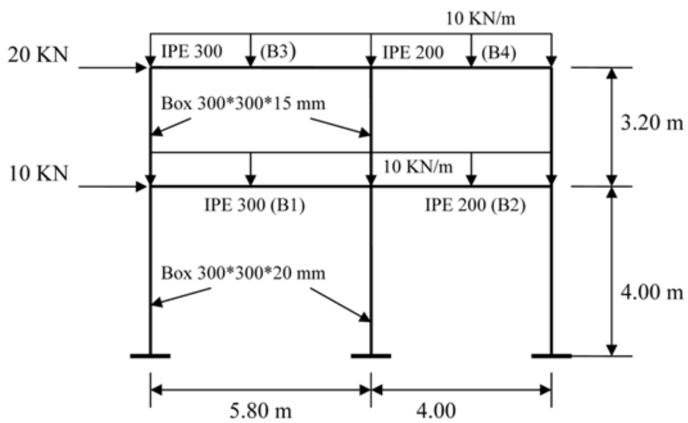


Fig. 7 Rigid frame under gravity and lateral loads

Table 1 Stress and percentage of connection rigidity reduction for the frame of Fig. 7

Beams	Beam-B1			Beam-B2			Beam-B3			Beam-B4		
Location	Begin.	End	Mid.	Begin.	End	Mid.	Begin.	End	Mid.	Begin.	End	Mid.
Bending moment at nominal analysis (KN.m)	12.06	44.38	16.91	8.60	18.46	6.72	12.19	42.03	17.93	10.06	17.55	6.63
Bending stress at nominal analysis (Mpa)	21.65	79.68	30.36	44.33	95.15	37.64	21.89	75.46	32.19	51.86	90.46	34.18
Bending stress at actual analysis (Mpa)	19.75	73.20	32.59	40.32	86.44	37.51	20.15	69.67	34.37	47.66	82.80	36.86
Difference between two analysis%	8.78	8.13	−7.33	9.05	9.15	−8.28	7.96	7.27	−6.76	8.10	8.47	−7.84
Average of connection rigidity reduction%	8.08%			8.83%			7.33%			8.14%		

Table. 2 Percentage of connection rigidity reduction for the frame of Fig. 6 at various conditions (Without internal stiffener)

	Width of column: b = 15 cm			Width of column: b = 20 cm			Width of column: b = 25 cm		
	d = 25	d = 20	d = 15	d = 25	d = 20	d = 15	d = 25	d = 20	d = 15
t = 0.8	6.30	6.55	6.80	7.73	8.06	8.57	10.58	11.00	11.68
t = 1.0	5.22	5.43	5.64	6.40	6.68	7.10	8.77	9.12	9.67
t = 1.2	4.50	4.68	4.86	5.52	5.97	6.12	7.56	7.86	8.34
t = 1.5	3.91	4.07	4.22	4.80	5.01	5.32	6.58	6.84	7.26

Table. 3 Percentage of connection rigidity reduction for the frame of Fig. 6 at various conditions (With internal stiffener)

	Width of column: b = 15 cm			Width of column: b = 20 cm			Width of column: b = 25 cm		
	d = 25	d = 20	d = 15	d = 25	d = 20	d = 15	d = 25	d = 20	d = 15
t = 0.8	1.97	2.15	2.57	2.49	2.66	3.00	3.00	3.17	3.52
t = 1.0	1.64	1.78	2.13	2.06	2.20	2.49	2.49	2.63	2.91
t = 1.2	1.41	1.53	1.84	1.78	1.90	2.15	2.15	2.27	2.51
t = 1.5	1.23	1.33	1.60	1.55	1.65	1.87	1.87	1.97	2.19

of these analyses for more than 70 conditions are presented in Table 2 and Table 3. These tables indicate that in the case of RHS column with internal stiffeners, the amount of reduction in the connection rigidity is very small and negligible. Moreover, by increasing the width of column (b) the deformation of the column face and the reduction of connection rigidity will be increased. On the other hand by increasing the column wall thickness (t) or depth of beam (d), the column face deformation and the reduction in the connection rigidity will be decreased.

The same analyses are done for different types of the frames and different patterns of the loads. The results show that the effects of the frames shape (No. of stories and No. of members) and the loading types (vertical or lateral loads) on the flexibility of the beam-column connections are negligible, and the most important parameter is the column section properties. Afterwards, the applied graphs for calculation the connection rigidity reduction is drawn only based on the basic parameters such as the column section properties.

In Figs. 8~10 the graphs for calculation the reduction in the rigidity of the beam-column connections are presented. These graphs are the function of b, d and t. The percentage of reduction in the rigidity of connections can be calculated with regarding to fully rigid case by using these graphs. These percentages can be used for analyzing the semi-rigid frames and also for evaluating the actual bending moment in the members. By using the connection rigidity reduction factor, or by assuming the semi-rigid behavior for the connections, the calculated bending moments at the beam ends are less than those of the fully-rigid case. Also the bending moments at the mid-span of the beams are greater than those of the fully-rigid case. Then the design bending moments of the beams (end moments) will be reduced, and consequently the size and weight of the designed beams will be reduced; therefore the total weight of the designed structure is usually less than that of the fully-rigid case.

8. Conclusions

The rigid connection of I-beams to RHS columns, usually behaves as semi-rigid connection which is

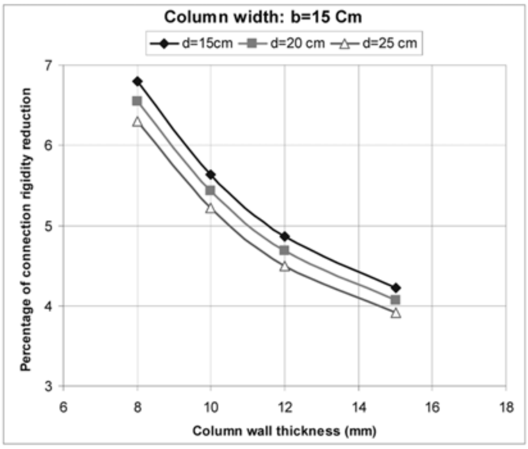


Fig. 8 Graph for calculation the connection rigidity reduction (Column width = 15 cm)

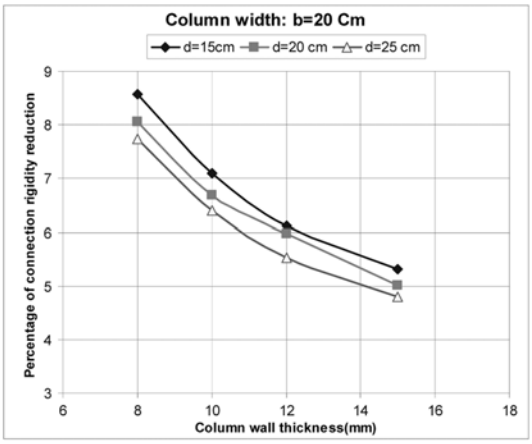


Fig. 9 Graph for calculation the connection rigidity reduction (Column width = 20 cm)

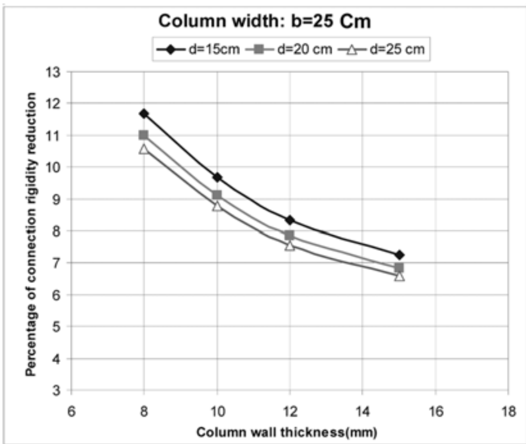


Fig. 10 Graph for calculation the connection rigidity reduction (Column width = 25 cm)

known under the influence of parameters such as: column width (b), column wall thickness (t), depth of beam (d), existence of internal stiffeners etc. The percentages of reduction in the rigidity of the connections, for RHS columns with internal stiffeners are negligible, and for the columns without stiffeners are noteworthy and it is about 7 to 20%. The percentages can be calculated from the graphs presented in this paper and they are useable for modeling the semi-rigid connections in the structural analysis or for redistribution the bending moments between the members.

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