Behavior of Two-way Steel Moment Connections with Side Plates and Double-I Built-up Columns under Cyclic Loading

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ABSTRACT :

The common method of connecting I-beam to double-I built-up column in the case of moment connections has two kinds of problems, 1) brittle fracture of full penetration groove welds of beam flanges to the column and 2) flexibility of the column cover plate that connects two I-shaped sections. To modify the connection, we have proposed a new detail. In the new geometry of connection, there is a gap between beam end and column face. The moment transfers from beam to the column by two full depth side plates. The tensile force is not directly applied to the cover plate. Therefore the problems of cover plate excessive deformations and the brittle fracture of full penetration groove welds are completely eliminated. The finite element method was used for investigating the cyclic behavior of two-way moment connections with side plates. Results show that these connections have high strength and stiffness and cause the ductile behavior of structure with formation of plastic hinges in the beams.

KEYWORDS: Double-I Built-up Column, Side Plate, Moment Connection, Cyclic Behavior

1. INTRODUCTION

Double-I built-up columns are composed of two I-shaped sections connected to each other by means of two cover plates. In this type of column there is a distance between I-shaped sections. The cover plates are only welded to I-shaped sections of column along their longitudinal edges. Moment connections of I-beam to double-I built-up columns have two distinct types of problems:

1- General problems; which concern nearly all welded moment connections. The most significant problem of this group is brittle fracture of full penetration groove welds between beam and column flanges. In double-I built-up columns it concern the groove weld between beam flanges (or moment transfer plates), and the column cover plate.
2- Special problem of moment connections of I-beam to double-I built-up columns, is due to existence of the column cover plate in moment transfer path. Since the cover plates are only connected to I-shaped sections of column, along their longitudinal edges, they behave very flexible under tensile force of beam flange and undergo large deformations.

The behavior of moment connection of I-beam to double-I built-up column was experimentally investigated by Mazrooee et al. (1999). The results of their investigations show that the existence of column cover plate causes the semi-rigid behavior of connection. The problem of large deformation of column cover plate under tensile force of beam flange is very similar to deformation of box columns flange in moment connections without internal diaphragm. Several researchers have proposed innovative solutions to this problem. Blais (1974), from his experimental works concluded that the best way of moment transfer from beam to the box column is using side plates parallel to the column webs. These plates were in levels of beam flanges and connected the sides of beam flanges to the column webs. In his procedure there was a gap between beam end and the face of column. The moments were transferred from the beam ends to column only through the side plates. In a subsequent research Picard and Giroux (1977), continued this innovative concept of connecting I-beam to box column, but they used angles instead of plates. Atsou et al. (1996), proposed nearly similar details. They applied trapezoidal plates in levels of beam flanges to connect beam flanges to the corners of the box column. All of the explained methods were based on the separation between beam ends and face of column. The proposed solutions caused modifications in behavior of moment connections with box columns, but all of them have some defects. After Northridge earthquake, Houghton (2000), proposed a new type of connection to eliminate general problems of welded moment connections. Like the other above mentioned methods, his connection uses the concept of separation between beam ends and the face of column, but he used one full depth side plates in each sides of the beam, instead of two separated plates in each level of beam flanges.

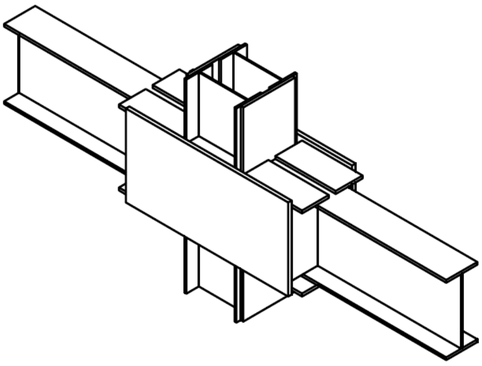


Figure 1 Moment connection with side plates and double-I built-up column

To eliminate both general and special problems concerning the welded moment connections of I-beams to double-I built-up columns, Deylami and Shiravand (2005), have proposed the use of two full depth side plates. In the present research we have investigated the cyclic behavior of two-way moment connections of I-beams to double-I built-up column with full depth side plates. The geometry of this connection is illustrated in Figure 1. In this connection the moment at the end of the beam is transferred to side plates by using top and bottom adjusting plates, and then side plates transfer the moment to the column. For simplicity of erection Deylami and Yakhchalian (2007), have proposed that the top adjusting plate be separated in two parts.

2. DIMENSIONS OF MODELS

To evaluate the behavior of two-way moment connection we have considered four models of subassemblies. Details of different parts of connection are indicated in Figure 2. The characteristics of beams, columns and connection parts are presented in Tables 1 and 2. t_{sp} , t_{ad} , t_{con} and t_{sh} are the thickness of side plate, adjusting plate, continuity plate and shear plate respectively. Supposing that points of inflection of beams and columns are in the middle of their length, half of beams length and column height is considered in each side of connection for modeling of two-way connections. The height of column and length of the beams that were considered for modeling were 3 and 3.5 meters respectively.

3. MODELING

ANSYS finite element software (1997) was applied for three dimensional modeling of all specimens. We have used SOLID45 element. This element has 8 nodes and three degrees of freedom per node. This element has capability of modeling large deformations and local buckling. Bilinear model was selected to represent stress-strain behavior of steel. The first line of the model has the slope equal to steel modulus of elasticity, E= 2.1×10^6 kg/cm^2. The yield stress of steel was considered F_y=2400 kg/cm^2. After the yield point the second line continued until ultimate tensile stress F_u of steel (F_u=3700 kg/cm^2). The strain of this point was considered to be equal to 0.2. The Poisson's ratio is considered ν=0.3. Since it was expected that nonlinear deformations mostly occur near the connected parts, more refined mesh and nonlinear material behavior were assigned to these regions. Whereas for portions of models that were far from the connection, the material was assumed to behave elastically. The plasticity model was based on Von Mises yielding criterion, and kinematics hardening was utilized for modeling of steel behavior.

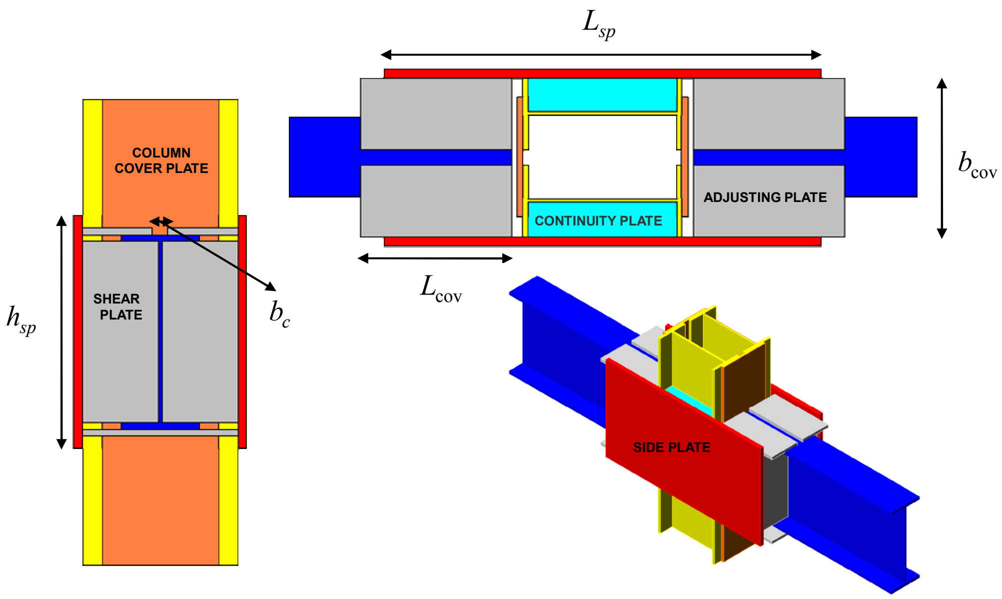


Figure 2 Details of moment connection with side plates and double-I built-up column

Table 1 Characteristics of beams and columns of the models (in mm)

MODEL	BEAM	COLUMN	CENTER TO CENTER	COLUMN COVER PLATE
SPD01	IPE300	2IPE270	200	250×10
SPD02	IPE300	2IPE300	200	300×15
SPD03	IPE400	2IPE300	200	300×15
SPD04	IPE400	2IPE400	220	300×15

Table 2 Dimensions of different parts of connections (in mm)

MODEL	t_{sp}	L_{sp}	h_{sp}	t_{ad}	L_{cov}	b_{cov}	b_c	t_{con}	t_{sh}
SPD01	15	910	370	10	350	335	40	10	10
SPD02	15	940	370	10	350	350	40	10	10
SPD03	20	940	470	15	350	350	40	17	12
SPD04	20	1140	470	13	400	400	40	13	12

Cyclic load was applied to the column tip in accordance with SAC loading protocol edited by Clark (1997). The loading history consists of stepwise increasing deformation cycles. The deformation parameter is the interstory drift angle (θ) which in two-way models is defined as column tip displacement (δ), divided to the height of column. The maximum interstory drift angle applied to the models was 0.06 radians. The loading history is shown in Figure 3. The beam ends were supported on rollers. The bottom of the columns was pin connected. For prevention of lateral-torsional buckling of beams adequate lateral support was provided.

Load Step	θ (rad)	n
1	0.00375	6
2	0.005	6
3	0.0075	6
4	0.01	4
5	0.015	2
6	0.02	2
7	0.03	2
8	0.04	2
9	0.05	2
10	0.06	2

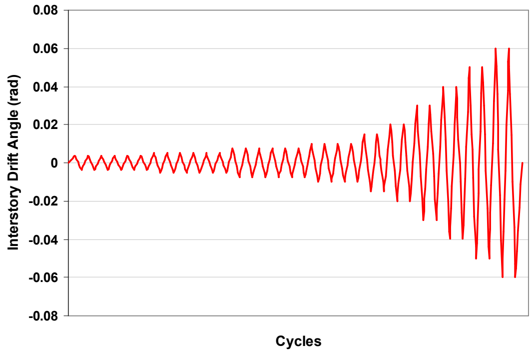


Figure 3 SAC loading history

4. STRESS AND STRAIN DISTRIBUTIONS

The Von Mises stress and equivalent plastic strain distributions for 0.06 radians interstory drift angle are shown in Figure 4 for all models. It can be seen that plastic hinges occur in beams. In order to have a better view of stress distribution in interior parts of connection, in Figure 4b, the front side plate is virtually removed. In model SPD03 that has smaller column/beam moment capacity ratio, the sides of column's flanges yielded slightly but this yielding does not cause significant strength degradation. In spite of this fact that in two-way models shear stresses in panel zone are approximately twice shear stresses in one-way models, in all models the panel zone remained elastic. As it can be seen, the local buckling of beam flange and web has occurred in 0.06 radians interstory drift. As indicated in Figure 4, due to separation between beam end and the face of column, stresses in column cover plate remain in elastic range.

5. MOMENT-ROTATION HYSTERETIC CURVES

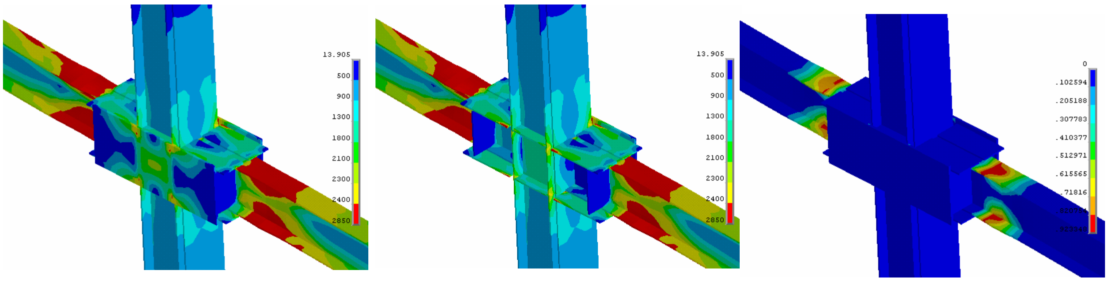
Moment-rotation hysteretic curves are shown for right and left beams for all models in Figure 5. Beam moment is measured at the face of column and rotation is considered equal to interstory drift angle. As it can be seen all models have suitable hysteretic behavior. Hysteretic curves show that strength of connection is reduced due to beam local buckling. With increasing of beam depth this strength degradation becomes more significant. But this strength degradation is not so important, since after the buckling, the strength of connection in all models, is still more than plastic moment capacity of beams. Therefore this connection can be classified as a full strength connection. As it can be observed from the hysteretic curves, all models have reached to 0.04 radians rotation, and the strength of connection at 0.04 radians rotation, is more than 80 percent of the beam plastic moment capacity, (0.8Mp). Consequently this connection satisfies the criteria of AISC Seismic Provisions (2005) for special moment frame systems.

6. PANEL ZONE MOMENT-ROTATION HYSTERETIC CURVES

For investigation of panel zone behavior, panel zone moment-rotation hysteretic curves are shown in Figure 6. The panel zone rotation is defined as:

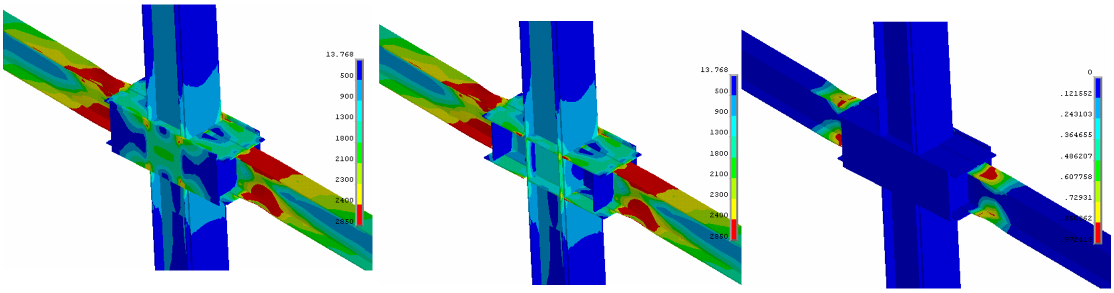
$$\gamma_{pz} = \frac{\sqrt{(a^2+b^2)}}{2ab}(\delta_1 - \delta_2) \tag{6.1}$$

Where a, b = initial dimensions of the panel zone, and δ_1, δ_2 = changes in length of the panel zone diagonals. As shown in Figure 6, panel zone rotation for all models has little value, because full depth side plates compel the region of energy dissipation to be out of the panel zone.



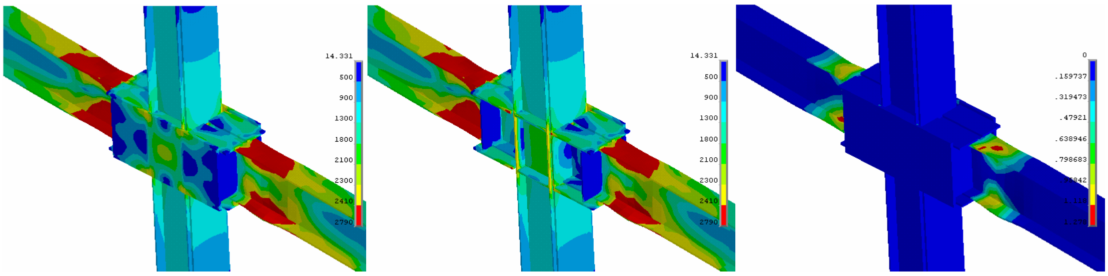
(a) **(b)** **(c)**

SPD01



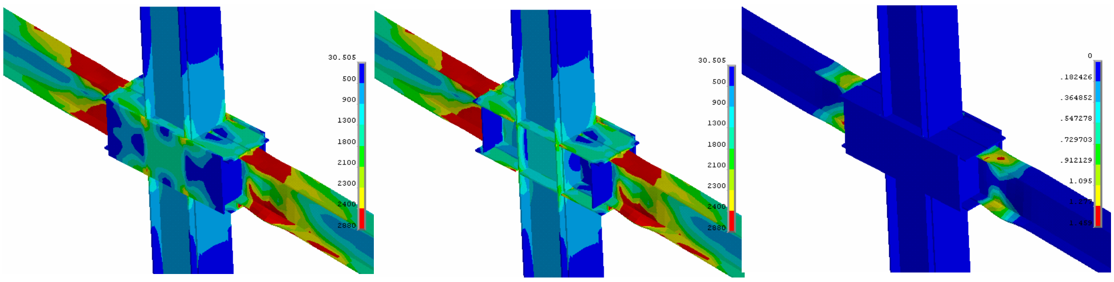
(a) **(b)** **(c)**

SPD02



(a) **(b)** **(c)**

SPD03



(a) **(b)** **(c)**

SPD04

Figure 4 Stress and strain distributions under cyclic loading in 0.06 radians interstory drift angle
a) Von Mises stress b) Von Mises stress (front side-plate removed) c) Equivalent plastic strain

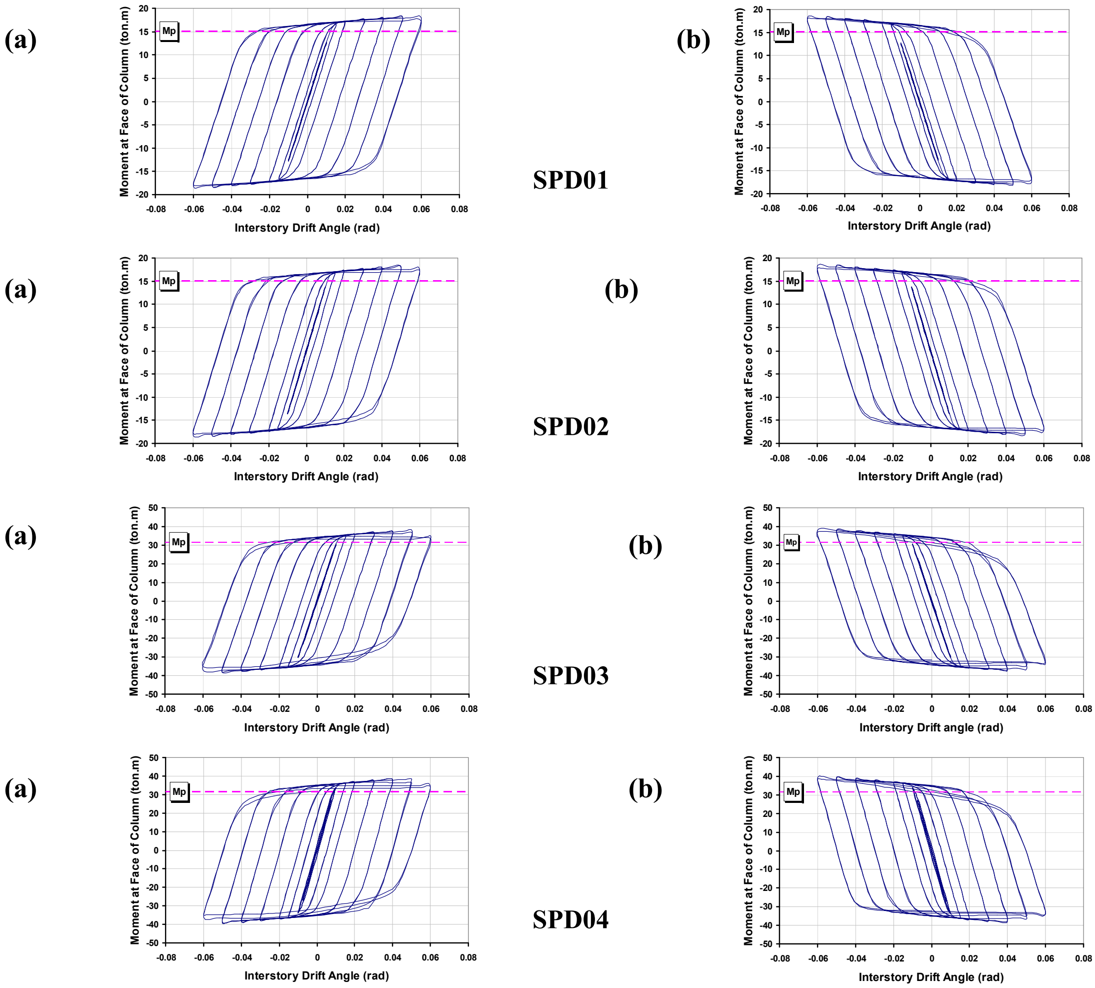
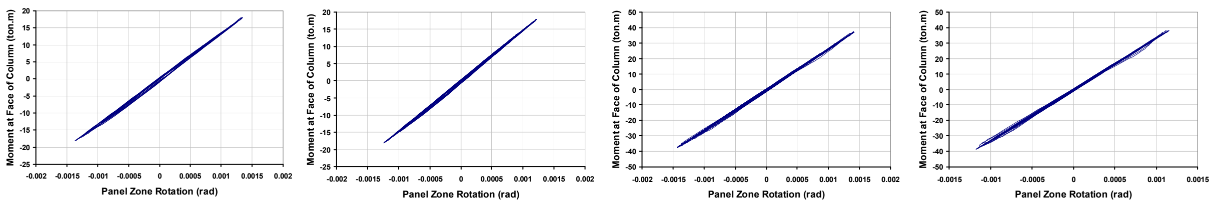


Figure 5 Moment-rotation hysteretic responses a) right beam b) left beam

In this type of connection, rotational behavior is completely independent of panel zone participation, so there is no more concerning about tearing between flange and web of column. As shown in Figure 6, in models with the same beam section the panel zone rotation reduces with increase of column section size.



SPD01 **SPD02** **SPD03** **SPD04**

Figure 6 Panel zone moment-rotation hysteretic curves

7. CONNECTION STIFFNESS CLASSIFICATION

The connections could be classified using moment-joint rotation curves. The joint rotation is considered as the summation of connection rotation and panel zone rotation. Panel zone rotation was computed in the previous section. The connection rotation is defined as:

$$\phi_C = \frac{((\delta_A - \delta_B) - (\delta_C - \delta_D))}{h_{sp}} \qquad (7.1)$$

$\delta_A, \delta_B, \delta_C, \delta_D$ = the horizontal displacement of points that are illustrated in Figure 7 and h_{sp} = depth of side plates. In Figure 7 points A and C are situated at the end of side plate and points B and D are situated at the intersection of side plate with column section. Moment-joint rotation curves are shown in Figure 8.

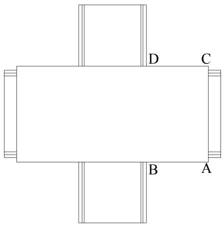
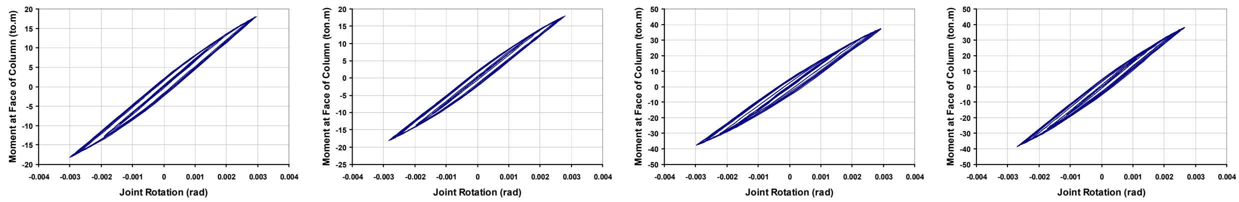


Figure 7 Points for computing connection rotation



SPD01 **SPD02** **SPD03** **SPD04**

Figure 8 Moment-joint rotation curves

Secant stiffness is computed using moment-joint rotation curves of models. Secant stiffness is defined as:

$$K_s = M_s / \theta_s \qquad (7.2)$$

$M_s = F_y \times S$. Where F_y is yield stress of steel and S is beam section modulus.
θ_s = joint rotation corresponding to M_s obtained from moment-joint rotation curves.

According to AISC Specifications for Structural Steel Buildings (2005), if $K_s L / EI \geq 20$ the connection can be considered as fully restrained. Where L and EI are length and bending rigidity of the beam respectively. Values of secant stiffness and $K_s L / EI$ are presented in table 3 for all models. The value of L in this table is considered equal to length of beam in the frame between two columns which is twice the beam length in each side of column in selected subassemblies.

Table 3 Stiffness classification of connection

MODEL	M_s	θ_s	K_s	I	L	EI/L	K_sL/EI
	Kg.cm	rad	Kg.cm	cm^4	cm	Kg.cm	
SPD01	1.34E+06	0.0019	7.04E+08	8360	671	2.62E+07	26.89
SPD02	1.34E+06	0.0018	7.43E+08	8360	667	2.63E+07	28.22
SPD03	2.78E+06	0.0019	1.47E+09	23130	667	7.28E+07	20.12
SPD04	2.78E+06	0.0016	1.74E+09	23130	657	7.39E+07	23.54

As shown in Table 3 all values of $K_s L / EI$ are greater than 20, therefore this connection can be classified as a fully restrained (FR) connection.

8. CONCLUSIONS

The main results of this study are:

1- In this connection plastic deformations significantly take place in the beam.
2- Only in one of the models sides of column flanges yielded slightly but this yielding did not cause significant strength degradation.
3- Due to use of two full depth side plates, the panel zone in all models remains elastic.
4- As shown in hysteretic curves this connection is a full strength connection.
5- This connection can be used in special moment frame (SMF) systems.
6- Due to use of two full depth side plates, in moment connection with double-I built-up column, panel zone rotation is approximately 2% to 3% of total rotation, therefore rotational behavior is completely independent of panel zone participation.
7- All values of $K_s L / EI$ are greater than 20, therefore this type of connection is a fully restrained connection.

REFERENCES

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

AISC, American Institute of Steel Construction. (2005). Specification for Structural Steel Buildings, Chicago.

ANSYS User's Manual (Revision 5.4). (1997). Swanson Analysis Systems, Inc. USA.

Atsou, T., Hiroshi, M. and Akiyoshi, I. (1996). Statistical Characteristics of WF Beam to RHS Column Connections Using Vertical Stiffeners. *In Proceeding of the 11th World Conference on Earthquake Engineering (11WCEE)*, Acapulco, Mexico.

Blais, R. (1974). Joints Soudes Pour Profiles Tubulaires, MSc. Thesis, Civil Eng. Dep., Laval University, Quebec.

Clark, P. (1997). Protocol for Fabrication, Inspection, Testing, and Documentation of Beam-Column Connection Tests and Other Experimental Specimens, SAC Joint Venture, Sacramento, California.

Deylami, A. and Shiravand, M.R. (2005). Moment Connection of Steel Built-up Column Using Side Plates. *In Proceeding of the 4th International Conference on Steel Structures*, Shanghai, China.

Deylami, A. and Yakhchalian, M. (2007). Cyclic Behavior of Two-way Moment Connections with Side Plates and Built-up Column. *In Proceeding of the 5th International Conference on Seismology and Earthquake Engineering (SEE5)*, Tehran, Iran, (in Persian).

Houghton, D.L. (2000). Steel Frame Connection Technology of the New Millennium Satisfying Heightened Performance Expectations with Simplicity and Reliability at Low Cost. *In Proceeding of the 12th World Conference on Earthquake Engineering (12WCEE)*, Auckland, New Zealand.

Mazrooe, A., Simonian, W. and Nikkhah Eshghi, M. (1999). Experimental Evaluation of Rigid Welded Connections Used in Iran, BHRC, Tehran, (in Persian).

Picard, A. and Giroux, Y.M. (1977). Rigid Framing Connections for Tubular Columns. *Canadian Journal of Civil Engineering* **4**, 134-144.