THE EXPERIMENTAL STUDY ON BUILT-UP COLUMN SEISMIC RESISTANT MOMENT CONNECTIONS USING SIDE PLATES

A. Deylami[1] and M.R. Shiravand [2]

[1] *Professor, Dept. of Structural Engineering, Amirkabir University of Technology , Tehran, Iran*
[2] *Ph.D. Student , Dept. of Structural Engineering, Amirkabir University of Technology , Tehran, Iran*
Email :deylamia@ aut.ac.ir, shiravand@aut.ac.ir

ABSTRACT :

Moment Connection of I beams to double-I built-up columns, through the cover plate, impose some problems such as brittle fracture of full penetration groove welds of beam flanges and flexibility of the column cover plate. To modify the connection for use in seismic regions we have proposed using of full-depth side plates.
This new geometry eliminates the traditional welded connection between the end of the beam and the face of column and all other uncertainties concerning the cover plate flexibility.
A series of experimental and numerical models were developed to study the behavior of the connection under cyclic loadings. The results indicate that this new connection geometry has sufficient strength and ductility to be classified as a special moment connection (SMF) for use in seismic regions.

KEYWORDS: moment connection, Built-up columns, side plate, seismic Resisting

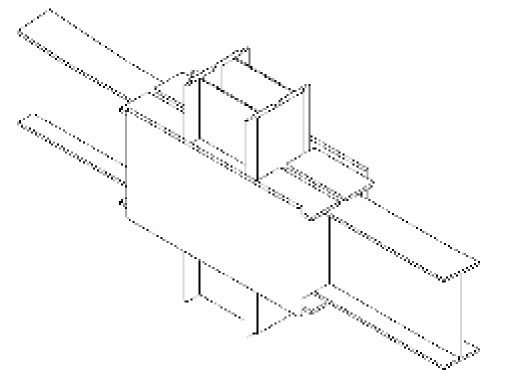
1. INTRODUCTION

Built-up columns have been used extensively in moment resisting frames. They are lighter as compared to the wide flange hot rolled sections. Double-I built-up columns are composed of two separated I-shaped sections connected to each other by means of two cover plates. Cover plates are only welded to I-shaped sections along their longitudinal edges. In the case of moment connection, when an I-beam connects to double-I built up column through the cover plate, the connection may impose two kinds of problems, 1) brittle fracture of complete-penetration groove weld that connects the beam flange directly to the column and 2) flexibility of the column cover plate that connects two I-shaped sections.
The problem of excessive deformation of cover plate in the case of double-I built-up columns under tensile force of beam flange is similar to deformation of box columns flange in moment connections without internal diaphragm. Several researchers have proposed innovative solutions to this problem. Blais (1974) proposed a new method for connecting I-beam to box column. He carried out a test on a model in which the upper and lower flanges of the beam were connected by means of side plates welded to the lateral walls of the box, instead of welding the beam flanges directly to the front face of the box column. Based on Blais work, Picard & Giroux (1976) and later Atsuo et al. (1996) and Engelhardt & Sabol (1998) carried out series of tests on moment connections for I-beams to square box columns. In all these researches, each side of beam flanges was connected separately to the columns (using four narrow side plates for each connection). A modification of the above mentioned methods was presented by Houghton (1998). Houghton used a pair of parallel full-depth side plates (instead of two narrow separated ones) to connect the beam to the column. This system is applied to wide flange shapes as well as rolled box sections columns. The innovative concept in all these methods was the separation between the end of the beam flanges and the column face. The ends of the beam flanges were not connected to the face of the box column and there were gaps between them. The beam never touches the column. The traditional T-joint full penetration groove weld between the end of the beam and the face of the column flange were eliminated.
Very few analytical and experimental investigations have been carried out on the behavior of moment connection of I-beam to built-up columns. No standard rules or provisions are available for design or evaluation of the behavior of such connections. None of the pre-qualified connection details proposed in FEMA 350 is applicable to the moment connection of steel I-beams to double-I built-up columns. To decrease the vulnerability of moment connection of I-beams to double-I built-up columns, any proposed method should be

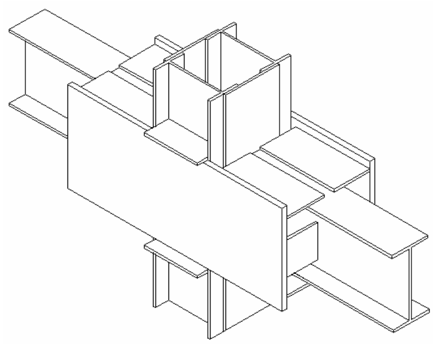
able to solve the two above mentioned problems simultaneously.
To eliminate both general and special problems concerning the welded moment connections of I-beams to double-I built-up columns, Deylami and Shiravand (2005) and Deylami and Yakhchalian (2007) proposed the use of two full depth side plates.

2. EXPERIMENTAL STUDY

In the present research we have investigated the cyclic behavior of two two-way moment connections subassemblies of I-beams to double-I built-up column with full depth side plates (models SP01 and SP02). The geometry of this connection is illustrated in Figure 1. Beam top flange adjusting plates and bottom flange seat plate are used, to adjust the difference between widths of the beam flange and built-up column. In order to simplify the assembling procedure, the top adjusting plate is divided in two pieces. In this connection the moment at the end of the beam is transferred to side plates by using top adjusting plates and bottom seat plates, and then side plates transfer the moment to the column. Moment transferring from the beam to side plates and from the side plates to the column is achieved trough the fillet welds. To transfer the shear force, the connection was provided with vertical shear tabs. Full depth side plates provide very strong column panel zone compelling inelastic action into the beam. Details of the side plate connection used for models SP01 and SP02 are illustrated in Figure 1.



(a) SP01



(b) SP02

Figure 1. I-beam to double-I column connection using full depth side plates

The model SP02 was modified based on practical experiences obtained from the construction of the model SP01. In model SP02, the continuity plates were enlarged to provide the seat for the side plates. Also, the dimensions of connection details were modified in order to decrease welding processes and cost.

2.1 Model Characteristics

The IPE rolled shape sections were provided for beams and built-up columns. The built-up columns were composed of two IPE shapes with continuous cover plates welded on their open sides. The geometrical characteristics of beams and double-I built-up columns are listed in Table 1. The details and dimensions of connections are presented in Table 2. Beams, built-up columns and their details meet the requirements of AISC-LRFD Specifications for Structural Steel Buildings (2005).
Typical details of different parts of subassemblies of models SP01 and SP02 are indicated in Figure 2. In these models, the beam far ends were supported on rollers. The bottom of the columns was pin connected. Beams were laterally supported at the distance of 170 cm from the face of column to prevent their lateral buckling under cyclic loading.

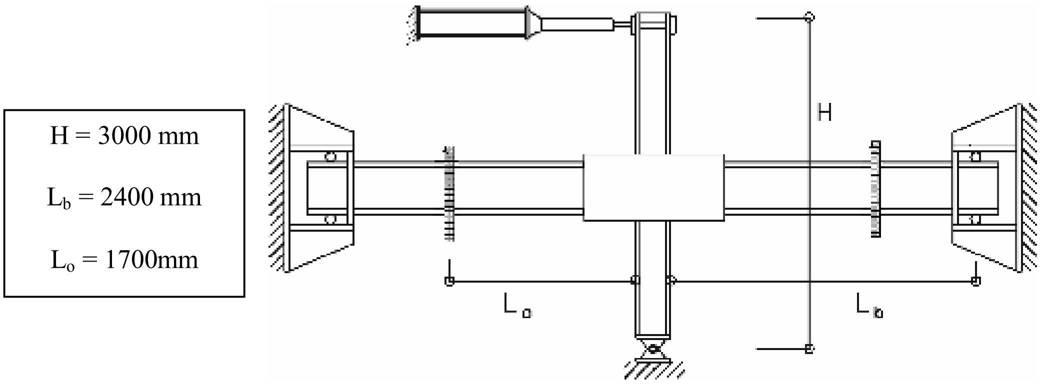


Figure 2. The two-way inter-story model

Table 1. Beams and Columns Specifications

Beam	Column section	Column cover plate	a (mm)	b (mm)	t_f (mm)	t_w (mm)
IPE300	2IPE270	250*10	200	150	8.3	6.6

Table 2. Characteristics of Models (beams and columns)

MODEL	t_{sp}	L_{sp}	h_{sp}	l_{cov}	t_{tcov}	b_{tcov}	t_{bcov}	b_{bcov}	t_{con}	t_{sh}
SP01	15	930	400	350	12	150	10	335	15	8
SP02	12	930	400	300	10	150	10	335	12	8

The variables in Table 1 and 2 are :
a = distance between center to center lines of IPE shapes in built-up column
b, t_f and t_w = flange width, flange thickness and web thickness of beam, respectively
t_{sp}, l_{sp} and h_{sp} = thickness, length and height of side plates, respectively
l_{cov} = length of top adjusting plates and bottom seat plate
t_{tcov} and b_{tcov} = thickness and width of top adjusting plates, respectively
t_{bcov} and b_{bcov} = thickness and width of bottom seat plate, respectively
t_{con} = thickness of continuity plates
t_{sh} = thickness of shear plates

2.2 Loading

Both models were tested under cyclic loading. Horizontal displacement imposed at the tip of the columns. Cyclic load were applied according to the AISC Seismic Provision (2005) loading protocol. The Final considered loading step was 0.06 radians

2.3 Material Properties

The mechanical properties of material used for beams and connection parts are listed in Table 3.

Table 3. Mechanical properties of models

Model	Specimen	Thickness (mm)	Yielding Stress (MPa)	Ultimate Stress (MPa)
SP01	Beam Flange	8.3	310.7	429.4
	Beam Web	6.6	345.1	456.4
	Top Adjusting Plate	12	354.0	477.5
	Bottom Seat Plate	10	312.0	464.9
	Side Plate	15	235	353.5
SP02	Beam Flange	8.3	310.7	429.4
	Beam Web	6.6	345.1	456.4
	Top Adjusting Plate	10	261.2	396.3
	Bottom Seat Plate	10	261.2	396.3
	Side Plate	12	343.0	461.3

3. RESULTS OF EXPERIMENTS

3.1 Model SP01 Behavior

The beam remained elastic at cycles of 0.00375, 0.05 and 0.075 radians. At the last cycles of 0.01 radians, yielding was occurred in top and bottom flanges of beams just after the adjusting and seat plates. The plastic zone was gradually developed in the webs of the beams during the cycles of 0.15 and 0.02 radians. At first cycle of 0.03 radians, the minor local buckling in beam flanges was happened but no strength degradation in load–displacement curve was observed. The flange buckling was increased at cycles of 0.04 radians and beam web slightly buckled. Strength degradation began at the end of cycles of 0.04 radians. At last cycles of 0.05 radians, flange and web local buckling and lateral torsional buckling at left beam were observed. The test was halted after cycles of 0.05 radians due to significant strength degradation of the model. No yielding in column, panel zone, side plates and cover plates and weld fracture were observed in this test.

3.2 Model SP02 Behavior

The beam remained elastic at 0.00375, 0.05 radians. Yielding, as indicated by minor flaking of the whitewash, began in beam flanges near the end of top adjusting plates and bottom seat plates during the cycles of 0.75% radians. By the cycles of 0.02 radians, the yielding in flanges was more pronounced and minor flaking on the beam webs was observed. Minor flange local buckling was observed during the cycles of 0.03 radians but no strength degradation was occurred. Web local buckling and more serious flange local buckling were observed during the last cycle of 0.04 radians. By 0.05 radians, the out-of-plane deformation of the beam flanges and formation of plastic hinge in beams were observed. The test was stopped at the last cycle of 0.06 radians due to excessive strength degradation. In this model, weld fracture and yielding in column, panel zone, side plates and cover plates were not observed either.

3.3 Hysteretic responses

Moment-beam rotation hysteretic response for models SP01 and SP02 are illustrated in Figure 3. The dashed line represents the $0.8M_P$ (M_P = plastic moment capacity) of beam. Regarding the beam specification the M_P is 155.5 (kN.m). We can observe expanding hysteretic behavior of I-beam to double-I built-up column connection provided with full depth side plates. As shown by hysteric responses, the beam moment strength decreases due to the local buckling.

Although, the local buckling causes a reduction in the flexural moment capacity of the beams but, their strength always remains more than $0.8M_P$. In both models, inter story drift angles reach to at least 0.04 radians, which satisfies the acceptance criteria proposed by FEMA350 to be classified as an Special Moment Frame "SMF" connection system.

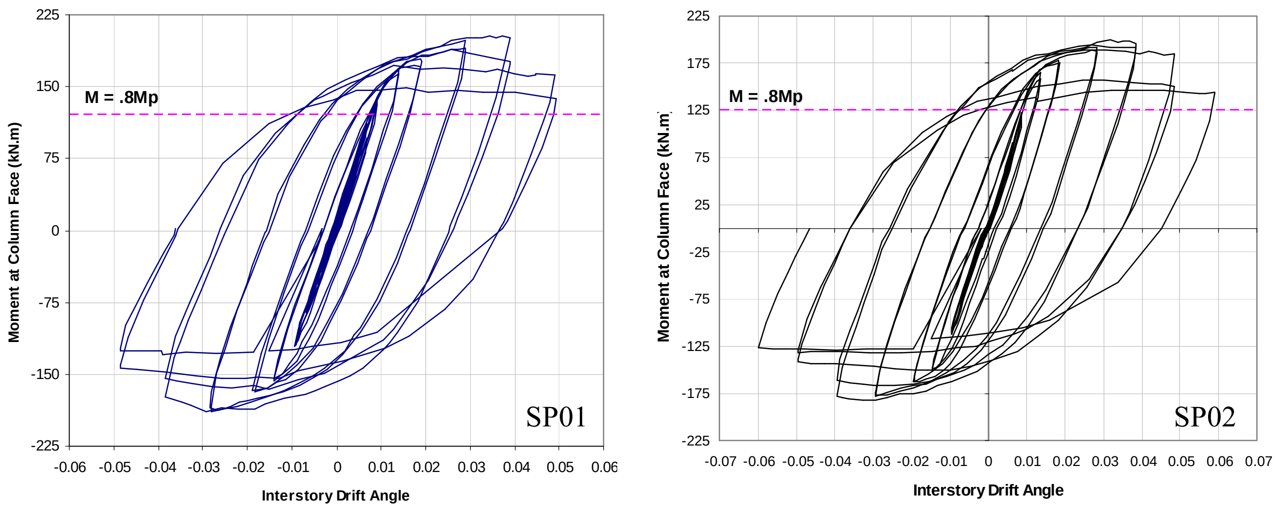
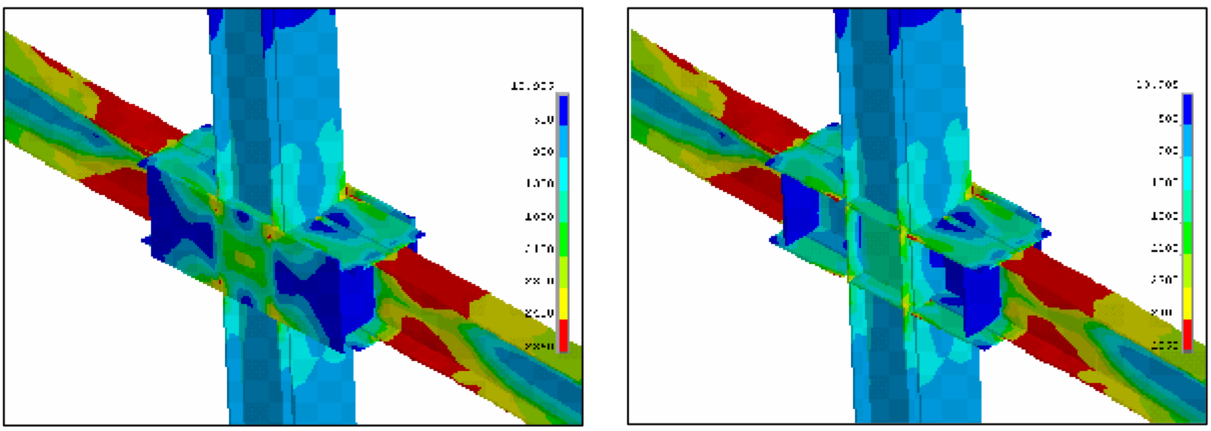


Figure 3. .Hysteretic responses

4. FINITE ELEMENT STUDY

The model SP01 was selected for finite element modeling. A kinematics bilinear model was selected to represent the stress-strain material properties. The 8-node brick solid elements with possibility to employ material nonlinearity and large deformation were utilized for modeling. The elastic modulus of steel was considered 210 (GPa). The mechanical properties of steel, boundary conditions and cyclic loading protocol were considered the same as for the tests.

Von Mises stress distributions for finite element model SP01 is shown in figures 4a and 4b. To have a better view of the stress distribution in the interior parts of the connection and panel zone, we have displaced the front side plate in figures 4b.



(a) (b)

Figure. 4. Effective stress distribution under cyclic loading
a) Von Mises stress b) Von Mises stress (front side-plate removed)

It can be seen that the plastic hinge occurs in the beam and the panel zone remains in elastic state of stresses. The connection was capable to move the plastic hinge from the face of the column into the beam to prevent the brittle fracture in connecting zone or formation of plastic hinge in the panel zone.

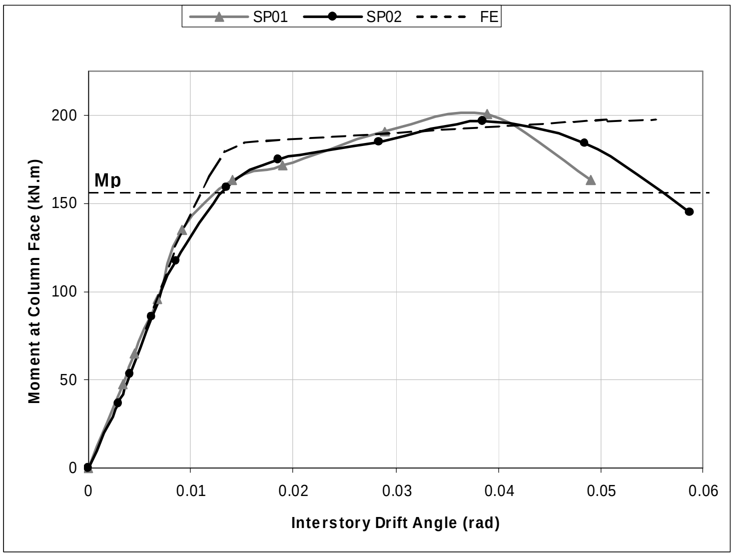


Figure. 5. Comparative M-θ Diagrams

5. ANALYSIS AND COMPARISON BETWEEN THE EXPERIMENTAL AND FE RESULTS:

The experimental results for moment-drift angles for models SP01 and SP02 and the finite element results for model SP01 obtained from hysteretic responses are shown in Figure 5. Comparing the diagrams, it can be seen that in elastic zone they are nearly the same. Models SP01 and SP02 have almost the same behavior in plastic region up to 0.04 radians. The rotation capacity (interstory drift angle) in two models reaches at least to 0.04 radians without any degradation. As it can be observed in figure 5, the local buckling of beam flange and beam web occurred under full loading. The buckling causes the anti-symmetric distribution of stresses in plastic hinges and decreases peaked moment resisting strength of the connection

Strength degradation due to initiation of buckling is observed in both diagrams. The strength degradation in model SP01 was happened earlier and was more significant than SP02 because of local and lateral tortional buckling occurred in model SP01 during the test. The degradation strength in SP02 is only due to local beam buckling so the degradation is smoother.

Comparing the finite element and experimental results diagrams for model SP01, it can be seen that in both elastic and plastic region the FE model verifies the test results with good accuracy. After the 0.04 radians, the test results differs from the FE model because of anti-symmetric behavior due to local and lateral tortional buckling which happened in the test model . Therefore, we can trust FE results to the pint that buckling affects the beam behavior

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6. CONNECTION CLASSIFICATION

The most important behavioral characteristics of the connection can be represented by a moment-rotation (M-θ) curve. Figure 6 shows M-θ curves for models SP01 and SP02. The connection response is defined by the contributions of not only the connection, but also the connecting elements and the column panel zone. The classifications account directly for the stiffness, strength and ductility of the connections according to AISC Specifications (2005) and AISC Seismic Provision (2005).

6.1 Connection Stiffness.

The secant stiffness K_S at service loads is taken as an index property of connection stiffness. Specifically, $K_S = M_S/\theta_S$ where M_S and θ_S are the moment and rotation, respectively, at service loads. L and EI are the length and bending rigidity, respectively, of the beam. According to AISC Specifications. If $K_SL/EI \geqslant 20$, the connection will be fully restrained (FR). If $K_SL/EI \leqslant 2$, the connection is considered as simple. Connections with stiffness between these two limits are partially restrained (PR). As it can be observed in Figure 6, the secant stiffness for models SP01 and SP02 connections are greater than 20EI/L (shown with fine dashed line). Then, it can be concluded that models SP01 and SP02 connections could be classified as fully restrained connection.

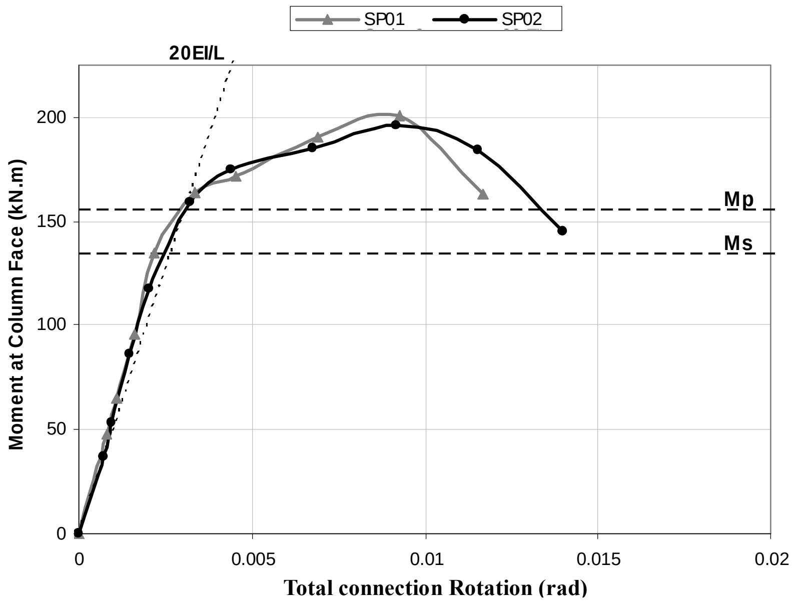

Figure.6. Classification of moment-rotation curve for models SP01 and SP02

6.2 Connection Strength

The strength of a connection is determined by the maximum moment that it is capable to carry out. The plastic moment capacity of beams (M_p) is shown in figure 6. If the connection strength substantially exceeds the fully plastic moment of the beam, then the ductility of the structural system is controlled by the beam. In this case, the connection could be considered fully strength (FS). Figure 6, shows that for models SP01 and SP02 connection strength are greater than fully plastic moment of the beam (M_p). Hence, models SP01 and SP02 connections are classified as fully strength.

6.3 Connection Ductility.

The ductility required of a connection will depend upon particular applications of structural system. The available rotation capacity should be compared with the rotation required to the strength limit state, as determined by an analysis that takes into account the nonlinear behavior of the connection.
Beam-to-column connection used in an Special Moment Frame (SMF) seismic load resisting systems shall

satisfy following requirements (a) the connection shall be capable of sustaining an interstory drift angles at least 0.04 radians and (b) the moment strength of connection, determined at column face shall equal at least 0.8 M_p of the connected beam at an interstory drift angles of .0.04 radians. As mentioned in previous sections (figure 3), for both models, interstory drift angles reaches at least to 0.04 radians, while the moment is still more than 0.8 M_p. This condition satisfies the acceptance criteria of AISC Seismic Provisions (2005) and these connections can be classified as SMF system connections.

7. CONCLUSION

The important results obtained from the experimental and FE are summarized below:
1. This new geometry eliminates all uncertainties in double-I built-up column moment connections.
2. The plastic deformations take place in the beam and panel zone remains elastic.
3. We can obtain the SMF connections by using full depth side plates for double-I built-up column in seismic region.

REFERENCES

AISC, American Institute of Steel Construction, (2005). *Load and Resistance Factor Design Specifications for Structural Steel Buildings*, Chicago.

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

Atsuo, T., Hiroshi, M., and Akiyoshi, I. (1996). Statical Characteristics of WF Beam to RHS Column Connections Using Vertical Stiffeners, *Proceedings of 11th World Conference on Earthquake Engineering* ,Paper No. 186, Acapulco, Mexico, Elsevier Science.

Blais, R. (1974). Joints Soudes Pour Profiles Tubulaires, *MSc Thesis*, Civil Engineering Department, Laval University, Quebec.

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).

Engelhardt, M.D., and Sabol T.A. (1998). Reinforcing of Steel Moment Connections with Cover Plates: Benefits and Limitations, *Engineering Structures* , Vol. 20 ,Nos. 4-6, April-June, 199 8, pp. 510-520

FEMA -350 (2000), Federal Emergency Management Agency, *Recommended Seismic Design Criteria for New Welded Steel Moment Frame Buildings,* , SAC Joint Venture, Sacramento, California.

Houghton, D.L. (1998) The Sideplate Moment Connection System: A Design Breakthrough Eliminating Recognized Vulnerabilities in Steel Moment-Resisting Frame Connections, *Proceedings of the 2nd World Conference on Steel Construction*, San Sebastian, Spain

Picard, A. and Giroux, Y.M. (1976). Moment Connection between Wide Flange Beams and Square Tubular Columns, *Canadian Journal of Civil engineering*, Vol.03, pp 174 -185