INFLUENCE OF STRUCTURAL DETAILING ON THE SEISMIC PERFORMANCE OF STEEL MOMENT RESISTING CONNECTIONS VIA AN EXPERIMENTAL TERMINOLOGY

Hooman Farrokhi [1] and F. Ahmadi Danesh [2] Sassan Eshghi [3]

[1] *PhD Candidate KN Toosi University of Technology-Civil Engineering Department-Tehran-Iran*
[2] *Assistant Professor - KN Toosi University of Technology- Civil Engineering Department-Tehran_Iran*
[3] *Assistant Professor–International Institute of Earthquake Engineering and seismology-Tehran-Iran*

ABSTRACT:
Moment resisting Connections are meant to behave as one of the most sensitive elements in an earthquake resistant structure, in their major stress concentration intrinsically imposed by the interconnected elements namely beams and columns, due to the lateral seismic stimulations. In addition to the required rigidity level and ultimate strength demand to transfer structural moments from beam to column and vice versa, the moment connections have to present an appropriate ductility capacity to withstand probable dynamic nonlinear deformations imposed by a severe ground stimulation like a strong earthquake. Therefore, a qualified moment resisting connection may be the one which provides suitable stiffness, sufficient ultimate moment capacity and favorable ductility potential. Two common moment resisting connection types are seismically, investigated by means of full-scale experimental specimens, subjected to quasi-static cyclic loads. Finite element analyses are also performed to predict the deterioration mechanisms & the failure modes of the connection. Basically, failure mechanisms occurred adjacent to the connection face in the form of the crack propagation & brittle fractures in the weld root & the heat affected zone which developed the premature failure of the connection & significant loss of the moment capacity. Flange buckling in some cases boosted the crack initiation in the weld root & turn out to be unfavorable. A proposed solution were then presented & discussed to shift the stress concentration & hence nonlinear actions from the connection face through the beam member. Drilled holes were designed in cover plates in such a quality to absorb the stress flow & concentration from the connection face. The failure mechanisms therefore were altered to the expected yielding adjacent to the holes & necking of the plate sections which significantly modified the connection ductility capacity & structural reliability. Experimental results show that the local stress concentrations usually taking place in the similar reduced beam section details (e.g. dog bone connection) are omitted by means of the separated cover plates while favorable yielding nonlinear mechanism is still present.

KEYWORDS:
Steel Frames, Moment Connections, Experimental Testing, Finite element analysis, Earthquake resisting structures, Rotational Ductility

1. Introduction

Due to their major stress concentration intrinsically imposed by the interconnected elements, the moment resisting connections are considered to act the most vital role in an earthquake resistant frame structure. In addition to the required rigidity level and the ultimate strength demand to transfer structural moments, the rigid connections have to present an appropriate ductility capacity to withstand nonlinear deformations imposed by strong earthquakes.

The expected stiffness and ultimate strength of a moment connection are, traditionally estimated based on analytical simplifications, typically not valid after few load cycles above yield limit. Also the nonlinear rotation capacity of the connection is assumed to be independent of the detailing and dimensioning factors which is contrary to the fact. Poor knowledge about the connection characteristic may result in unpredictable seismic performance of the building system. [9], [10]. Before the Northridge 1994 earthquake in US (also Kobe 1995 earthquake in Japan), it was assumed that the standard directly welded connection will meet all the structural requirements to withstand strong ground motions. The Northridge earthquake damaged many steel moment resisting frame buildings [22]. The unexpected performance of the buildings effectively invalidated the building codes and the professional practice used prior to the earthquake. [6], [21]. Different behavior of similar details was observed which was typically due to hypothetical uncertainties or constructional imperfections. After the earthquake, the UBC removed the pre-qualified status of the prequalified connection which urged the development of research programs to investigate the connection behavior and to recommend new detail designs if needed. While many researchers individually performed valuable investigations on steel moment connections [1], [4], [11], [12] [13], others contributed in national research projects [15], [16], [22].

Thorough experimental evaluations of two common connection types are implemented and their nonlinear responses have been discussed. Finally, an alternative detail is proposed to eliminate unfavorable brittle mechanisms, which shall modify the reliability level of the moment resisting connection. The general specifications of the connection subassemblies are provided in table 1.

Table 1. General specification of the connection subassemblies

Specimen	Description	Column Section	Beam Section	Connection type	Cover plate dimensions (mm)	Continuity Plates (mm)	Penetration Welding Material	Fillet Welding Material	Number of Drilled Holes	Diameter of Drilled Holes (mm)
SPC1	Benchmark	IPB240	IPE300	Direct Flange Weld	NA	100x200x12	E7018	E6013	NA	NA
SPC2	Evaluation	2IPE270	IPE300	Cover Plates	200x400x16	50x240x8	E7018	E6013	NA	NA
SPC3	Proposal 1	2IPE270	IPE300	Enhanced reduced Cover Plates	200x400x16	50x240x8	E7018	E6013	4	6
SPC4	Proposal 2	2IPE270	IPE300	Enhanced reduced Cover Plates	200x400x16	50x240x8	E7018	E6013	5	10

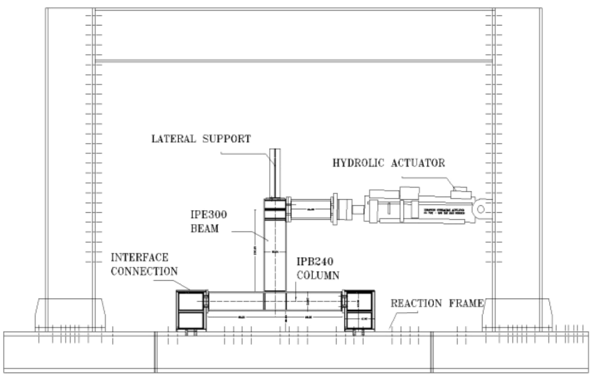

Figure1. Testing assembly

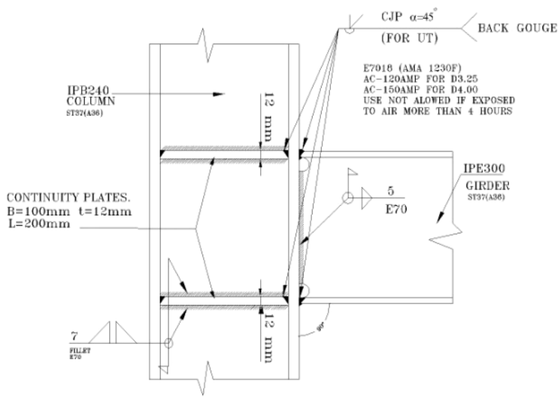

Figure2. SPC1- Direct flange groove weld

2. Experimental seismic evaluation of moment connections

Full-scale specimens are designed and constructed in accordance with the well-known design codes and recommendations to provide a comparative database. [2], [3], [19], [20]

The experiment setup typically consists of a T shaped beam to column connection while the connection is aligned horizontally connected to the reaction frame. The schematic view of the testing assembly is provided in figure 1.

Several material testing specimens are provided in accordance with the ASTM standard to reveal the stress-strain relationship of the material commonly used in numerical analyses and test calibration. Strain gauges and displacement transducers are instrumented at the decided locations based on the numerical investigations to shed a light over the material in terms of strains and displacements. Quasi-Static cyclic tests were conducted experimentally to investigate the global and local response and failure mechanisms of the connection types.

Figure3. Specimen 1 at final hysteretic displacement amplitude

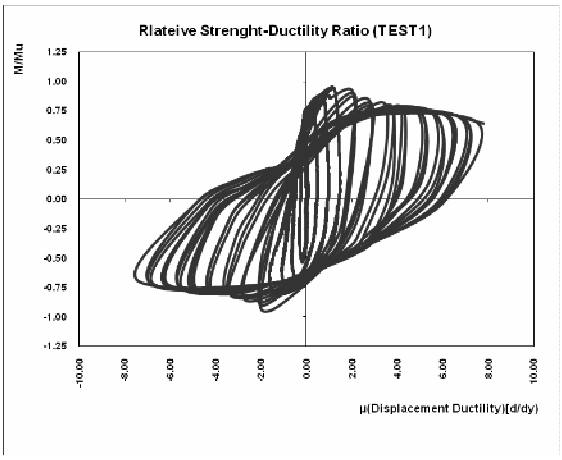

Figure4. Hysteretic relative moment rotation behavior of specimen 1

2.1. Seismic performance of moment resisting connections with direct flange groove weld (SPC1)

This type had become one of the most popular connection details used in US and other areas before Northridge earthquake 1994 [15]. The first specimen (SPC1) was proposed to relate the current study to the existing experimental knowledge.

The specimen dimensions and the connection detail can be schematically seen in Table 1 & Figure 2. The Geometry is generally similar to the specimen tested by Calado et al [7]. Material specimens were extracted from beam and column flanges and webs to reach the exact stress-strain relationships of the elements.

The strain gauge results show accumulated stress concentration at the vicinity of the connection edge at the weld roots. This modifies the potential for crack propagation in the region and the adjacent heat affected zone. In addition, significant nonlinear strains are anticipated in the beam flange due to buckling occurring in the loading amplitude about 4 times the yield displacement (figure 3). If the penetration welds are well designed and constructed, the connection may reach its expected strength and ductility capacity. Otherwise, a premature failure of the weld roots is experienced which will limit the overall deformation and ultimate moment capacity of the connection. In the tested specimen welding procedures, continuity plates, beam and column dimensions, web angles and other related details are designed precisely so that other unfavorable failure mechanisms. The buckling of beam flanges happens at a distance about the beam depth from the connection edge, which disturbs the strain distribution along the compressive flange. Moment capacity decreases slightly after buckling point 75 % of the total expected capacity while the final rotation capacity is reached. The hysteresis loops are typically convex and consistent with a gradual evolution of hysteretic areas. The moment rotation diagram of the specimen may be seen in figure 4.The connection shall be categorized as full strength moment resisting connection with a local rotational ductility ratio of about 7.9 and moment capacity of 217 kN.m which is about 92 percent the beam moment capacity. The connection is assumed favorable to use in ductile earthquake resisting frames with a mandatory focus on welding procedure and quality control. It might be concluded that the most sensitive factor affecting the performance of such connections is the completeness of the groove welds penetration. In case of a partial penetration, the gap between unwelded parts will act as an artificial crack (notch) from where the crack initiates and propagates through the material.

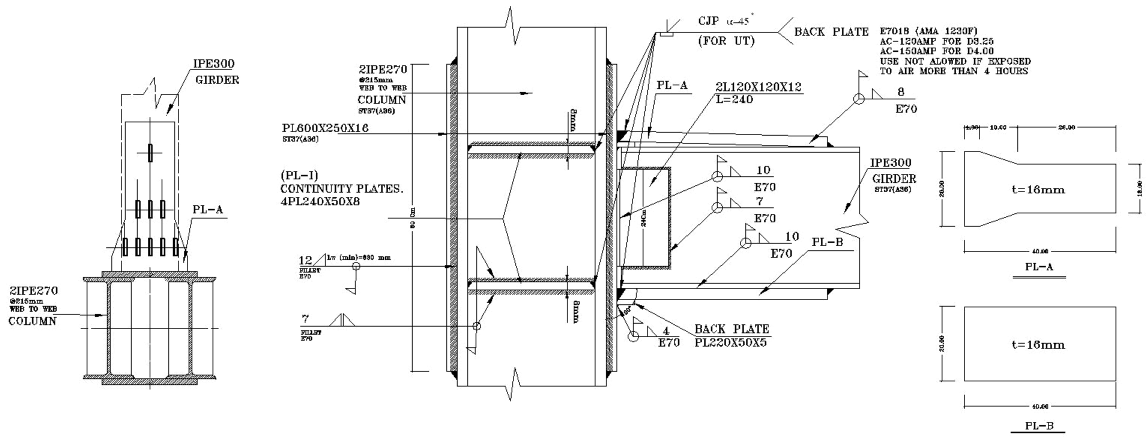

Figure5. Specimen 2- Cover Plates connected to an end plate welded to a Double profile Column

2.2. Seismic performance of moment resisting connections with cover plates welded to a double profile column (SPC2)

Due to its constructional versatility and simple fabrication benefits, this type has become one of the popular connection types used in residential and industrial structures. It typically consists of cover plates connecting the beam to the column by means of penetration welds to the column flange.

As double I sections are used to form the column, an end plate shall be used to create a suitable surface for the connection of cover plates. This end plate will undergo out of plate deformations, which may be greater than the yield limit. Therefore the plate should be dimensioned precisely to withstand induced forces with no major negative effect on the total connection behavior. Many sensitive parameters are neglected by the design experts. Out of plane bending deformations of the end plate compromising with the severe stress concentrations at the cover plates near to the beam edge, may end to an unsatisfactory failure mode in form of crack initiation.

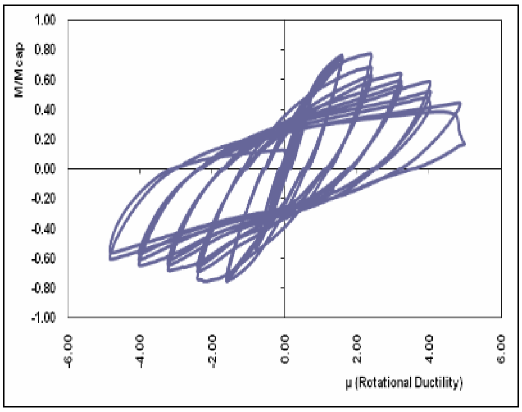

Figure6. Hysteretic relative moment rotation behavior of specimen 2

Figure7. Specimen 2- Complete cracking of weld root adjacent the Copper Back weld plate

The interesting fact about this type is that by using the cover plates, the buckling mode of beam flanges is postponed and the stress concentration is boosted at the weld roots. Cover plates transform the beam moment to a force couple acting on weld roots while they are strengthening the beam flanges preventing them from buckling. As it is anticipated in SPC2 no sign of buckling in beam flanges or yielding in cover plates is observed while the weld root are cracked and the end plate is torn through the thickness.

While stress distribution is generally steady in bottom cover plate, which is a rectangular plate, a stress concentration factor about 1.80 is estimated for the top tapered plate. Dimensions chosen based on simple

design guides seems to be acceptable. The specimen failed at the connection edge in form of crack propagation at the weld root growing in the end plate.
The connection reached the maximum 75 percent expected beam capacity (177 kN.M) and a ductility ratio hardly reaching 5 with a post failure moment capacity about 30 percent of the expected beam moment capacity. The crack initiated from the weld roots may be seen in figures 7 to 9. An unsymmetrical behavior is also observed in tensile and compressive plates due to the out of plate bending deformation, which is solely taking place in the tensile part. This releases the strains in the tensile cover plate and boosts the stress concentration in the end plate. Therefore, the maximum compressive stresses are somehow greater than the tensile ones. The connection seems to be unreliable to be used in ductile earthquake resisting frames.

Figure8. Specimen 2- Bucking of the end plate and Crack propagation from the weld root

Figure9. Specimen 2- Through the thickness crack propagation in the end plate

Insufficient moment and ductility capacity compromising with rapid drop in stiffness and strength with no previous warning, may cause total collapse of the structure and increased human fatalities. Fundamental modifications are recommended to alter the brittle fracture mechanism to a ductile mode and to shift the stresses from the column face vicinity.

2.3. The proposal to the new enhanced detail (SPC3 and SPC4)
One of the most interesting concepts employed by many researchers to avoid premature connection failure is to shift the nonlinear mechanisms from the connection edge. One method to reach this aim is to create an intentional weak point so that the nonlinear deformations are shifted to that point. Some researchers have proposed reduced flange sections to create the weak point [8] this may cause stress concentrations in the beam web. Some other, proposed cast connections with variable flange thickness [17] although it seems effective, it's very time & cost consuming. Also the use of channel or angle connectors welded to beam column flanges is proposed by other researchers which seems strongly case sensitive.[18]
A suitable alternative to create the weak point is the use of reduced cover plate sections. The cover plates prevent the flange buckling, while they absorb almost all the nonlinear deformations of the connection element. After the seismic stimulation, the cover plates may be replaced with new ones. The other parts may remain undisturbed while the cover plates behaved like the fuse of the earthquake resistant system.
As stated previously, direct welded flange connection (SPC1) featured a satisfactory seismic performance level. Thus, it is extremely sensitive to the penetration weld root quality. In presence of significant inclusions, the connection may experience weld root fracture prior to the flange buckling. This problem is boosted in cover plate type connection (SPC2).
Cover plates eliminate the buckling mode of beam flanges and concentrate the structural forces to the weld roots. This causes a premature failure of the connection that leads to a poor seismic performance. It is proposed by the author, that the unfavorable brittle fracture of the weld roots and the heat affected zone shall be prevented by releasing them from the stresses imposed by cover plates. To this aim, an intentionally weak point is created by means of drilled holes in cover plates balanced with a demand capacity to absorb the stress concentrations. Using this technique yielding mechanism is expected to occur prior to the weld cracking. The plate and holes might be designed and dimensioned in such a way to satisfy this goal. The design concept may be expressed as the follows:
An upper bound plastic mechanism of plate yielding around the holes must always occur prior to a lower

bound mechanism of the weld root. This can implement a required immunity factor to consider imperfections.

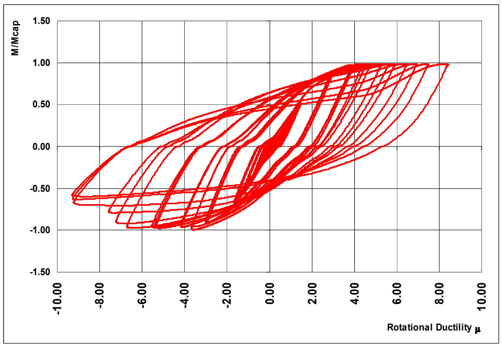

Figure 10.hysteretic behavior of SPC3

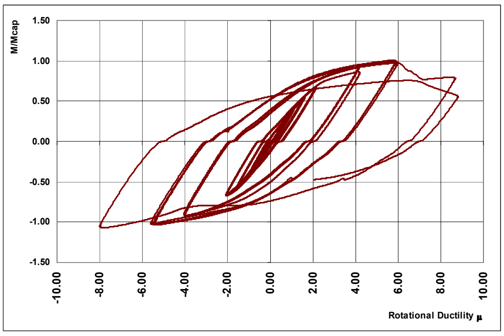

Figure 11.hysteretic behavior of SPC4

SPC3 and SPC4 are typically the same size as the SPC2 with some general modifications. Horizontal 10 mm thick stiffener plates are mounted and welded to the end plate to stiffen it in the out of plane direction.

Figure12.net section yield of the upper cover plate (SPC3)

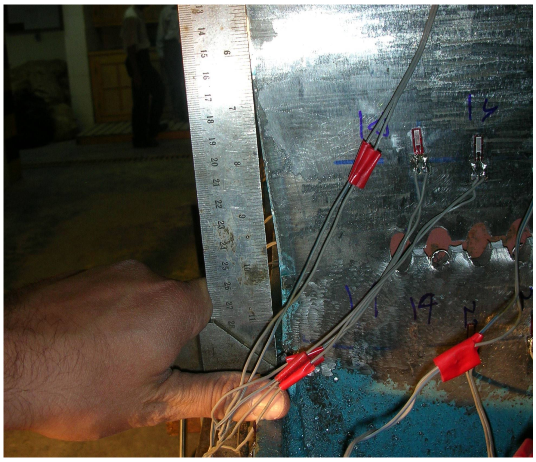

Figure13.Necking of the bottom cover plate (SPC3)

Drilled holes are located 15 cm away from the connection face to provide intentional stress concentrations and shift the nonlinear deformations. The only difference between SPC3 and SPC4 is in the number and diameter of holes, which are designated, based on the target capacity demand and safety factor. SPC3 contains four holes 6 mm diameters with equal 4 cm axe-to-axe distance, while SPC4 contains 5 holes 10 mm diameter with 3 cm axe-to-axe distance depicting reduced target moment capacity. Material and construction procedures are the same as prior specimens. SPC3 was proposed to experience the boundary between two failure modes. Therefore the specimen shall experience and interaction of this two extremes. The experiment accredited the discussed concept. Significant yielding was observed around holes in cover plates interacted with minor cracks forming around weld roots. (Figure 10)
The energy dissipation and the form of hysteretic loops are improved in comparison with SPC2 significantly but still the fracture controls the failure mode. The premature weld fracture prevented progressive yielding around holes. The SPC3 delivered 96 % of its expected moment capacity reaching a ductility ratio about 8.2 which is superior in comparison with the original connection (Figure 12, 13)
SPC4 was designed to demonstrate a secure reliable connection where almost no cracking is possible. This connection features a predictable ductile yielding in cover plates away from weld roots. Repeated yielding is observed in constant amplitude load cycles.

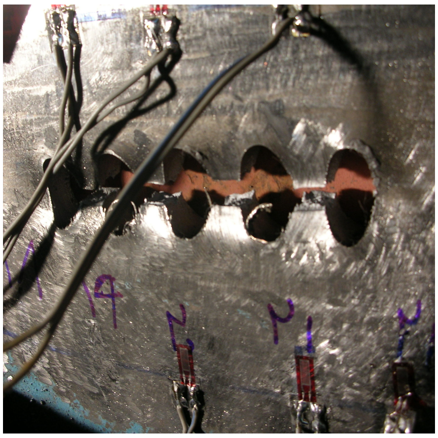

Figure14.yielding around bottom cover plate (SPC 4)

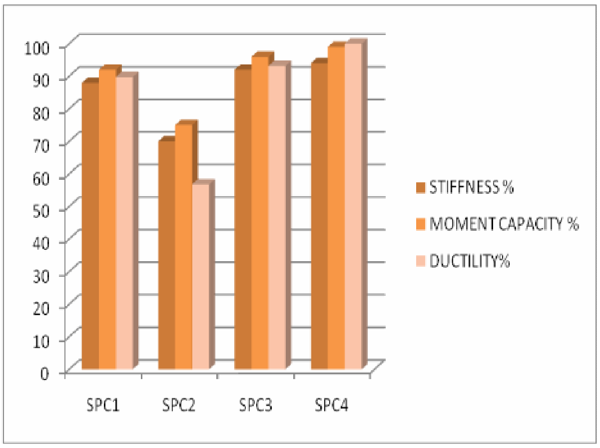

Figure15.Normalized comparison of connection subassemlies based on experimental results

The experiment validated the design terminology. No cracking were observed in weld roots and the adjacent areas while large plastic deformations exist around holes. Necking of the cover plates and net section yield were anticipated around holes. Hysteretic loops are wide and consistent.

Table 2. Test Results

Specimen No.	θy=Yield Rotation (Rad)	My=Yield Moment (kN.m)	Ki=Initial Stiffness (KN.m)/Rad	RG=Rigidity= Stiffness/ 12EI/L3 (%)	M= Maximum Moment Capacity (kN.m)	Mp= Beam Plastic Moment (kN.m)	M/Mp (%)	M res= Post Failure Residual Moment (kN.m)	M_{res}/Mp (%)	θu=Maximum rotation(Rad)	μ=Rotational Ductility Factor=θu/θy
SPC1	.0063	98.4	15619	88	217	236	92	170	72	.050	7.9
SPC2	.0076	94	12368	70	177	236	75	71	30	.038	5
SPC3	.0068	111	16323	92	226	236	96	177	75	.055	8.2
SPC4	.0070	116	16571	94	233	236	99	175	74	.062	8.8

3. Conclusions

Based on the numerical and experimental investigations performed on the full-scale connection models the following conclusions shall be made:

3.1. Moment resisting connections with direct beam flange welds feature sufficient stiffness and strength and a suitable rotational ductility capacity. It can be used as a rigid full strength ductile connection for steel building structures. The quality and complete penetration of the groove welds is the key factor, which controls the connection seismic behavior. In case of significant inclusions like micro cracks, porosity, partial penetration etc. premature failure in form of crack initiation is possible to occur. That, results in poor seismic performance. The reliability of this type of connection may be diminished by the unpredictable weld fracture induced by constructional imperfections.

3.2. Cover plates have valuable constructional benefits. They also omit the buckling mode of the beam flanges or shift it to the plate end, but in the other hand, they concentrate the internal actions to a force couple pointed to the most weakened region, the column face. High stress concentration compromising with significant imperfections resulted from welding, will end to the premature brittle fracture of the connection and hence the poor seismic performance. Ultimate moment capacity and rotational ductility capacity of the connection is decreased down to 25 % and 36 % ,respectively in comparison with SPC1 (figure 15).

3.3. Weld fracture modes are extremely unpredictable and may cause in rapid loss of stiffness and strength of the connection. They might be eliminated by releasing the weld root from stress concentrations. This may be done by stiffening the connection vicinity and weakening a predefined plastic point acting as a fuse to absorb the plastic deformations well far from the connection face. In this region, a stable yielding mechanism is viable to occur.

3.4. The proposed connection has a favorable and reliable performance with an expected plastic region depicting stable necking and finally the net section yield of the cover plates. While the ultimate demand moment capacity of the connection is delivered, the rotational ductility ratio is improved up to 11 % in comparison to specimens 1. The ductility ratio is nearly doubled in comparison with SPC2 (Figure 15). This detail has significant benefits in comparison with some similar details. It eliminates unfavorable cracking modes at the weld roots while it is easily replaceable after sever earthquakes.

4. Acknowledgements
The authors would like to acknowledge IIEES & KN Toosi University of technology for their valuable support for the experimental parts of the current study.

5. References

1-Ahmadi Danesh Ashtiani,F ;Seismic Behavior of Steel Frames With Semi-Rigid Connections-PhD Dissertation imperial College of science Technology and medicine University of London 1996
2-AISC-ASD Code for steel structural design
3-AWS D1.1- Structural welding code-steel-American welding society
4-Astaneh A, Nader M.N; Cyclic Behavior of Frames with Semi Rigid Connections-Proceedings of the second International Workshop Connections in Steel Structures II 1991
5-ATC-24 Guidelines for cyclic seismic testing of components of steel structures 1992
6-Bertero, V.V and Anderson, J.C and Krawinkler,H; Performance of steel Building structures During the Northridge Earthquake-Report No.UCB/EERC94/04,EERC,University of California, Berkeley
7-Calado, L and Mele.E; Experimental Behavior of Steel Beam to Column Joints: Fully Welded VS Bolted Connections-proceeding of the 12th World Conference on Earthquake Engineering (12WCEE)-Paper No2570
8-Chang che ehou & Chia Ching wu; Performance evaluation of steel reduced flange plate moment connections-Earthquake engineering & structural dynamics Vol.36 Nov2007
9-Chen W.F.;Joint Flexibility in Steel Frames-Elsevier Applied Science-Purdue University , West Lafayette, Indiana 1936
10-Chen W.F. and Lui,E.M Beam to column Moment Resistant Connections- Elsevier Applied Science Publishers, London
11-Clifton, G.C. and Butterworth,J and Scholtz,W.; Performance of Rigid Welded Beam to Column Connections Under Severe Seismic Conditions-proceeding of the 12th World Conference on Earthquake Engineering(12WCEE)-Paper No 1601
12-Elghazouli, A.Y Ductility of Frames with Semi Rigid Connections-Eleventh World Conference on Earthquake Engineering (11WCEE)-Paper No 1126
13-Elnashai, A.S. and Elghazouli A.Y; Seismic Behavior of Semi Rigid Steel Frames -Journal of constructional Research Vol29 No1-3
14-Faella, C and Mazzolani,F.M; Cyclic Behavior of steel Beam to Column Joints-Experimental Research,Vol2
15-FEMA267 Interim Guidelines: Evaluation, Repair, Modification and Design of welded Steel Moment Frame Structures-1995
16-FEMA 351 Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment Frame Buildings-July 2000
17-Fleischman,Robert and Xuejun Li,Yong Pan and Sumer, Ali; Cast modular panel zone node for steel special moment frames- paper no.1393 & 1404, Journal of structural engineering, ASCE-Vol. 133,oct 2007
18-Kumar, Satish S.R. & Prasada Rao; RHS beam to column connection with web opening, experimental study & finite element modeling- Journal of constructional steel research, Elsevier-Vol. 62-2006
19-Mazzolani, F.M and Piluso, V; Theory and Design of Seismic Resistant Steel Frames 1996
20-Naeim, F. Seismic Design Handbook-Van Nostrand Reinhold 2004
21-Popov E.P. and Yang, T. S.; Analytical Studies of Pre Northridge Steel Moment Resisting Connections-11th World Conference on Earthquake Engineering (11WCEE)-Paper No661
22-SAC Joint Venture; Technical Report: Experimental Investigations of Beam-Column Sub assemblages-Report No. SAC 96-01 Part 1 and 2-1996