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# The structural detailing effect on seismic behavior of steel moment resisting connections

*Hooman Farrokhi[1], F. Ahmadi Danesh[1a] and Sassan Eshghi[2b]

[1]*Civil Engineering Department, KN Toosi University of Technology, Tehran, Iran*
[2]*International Institute of Earthquake Engineering and Seismology, Tehran, Iran*

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**Abstract.** Different types of moment resisting connections are commonly used to transfer the induced seismic moments between frame elements in an earthquake resisting structure. The local connection behavior may drastically affect the global seismic response of the structure. In this study, the finite element and experimental seismic investigations are implemented on two frequently used connection type to evaluate the local behavior and to reveal the failure modes. An alternative connection type is then proposed to eliminate the unfavorable brittle fracture modes resulted from probable poor welding quality. This will develop a reliable predefined ductile plastic mechanism forming away from the critical locations. Employing this technique, the structural reliability of the moment resisting connections shall be improved by achieving a controllable energy dissipation source in form of yielding of the cover plates.

**Keywords:** steel frames; moment connections; experimental testing; finite element analysis; earthquake resisting structures; rotational ductility.

## 1. Introduction

Moment resisting connections are considered to have the most fundamental function in an earthquake resistant frame structure. This is mainly due to the major stress concentration intrinsically imposed by the interconnected elements. A qualified moment resisting connection may provide suitable stiffness, sufficient ultimate moment capacity and favorable ductility potential. The expected stiffness and ultimate strength of a moment connection are estimated using methods based on analytical simplifications, which, area not valid after few dynamic load cycles above yield limit. In addition, the nonlinear rotation capacity of the connection is understood to act independently, considering the detailing and dimensioning factors which are divergent to the fact. These shortcomings are basically resulted from poor knowledge about the connection characteristic which may result in false prediction of seismic performance of the building system. In conclusion, numerical investigations coupled with enhanced experimental models are advised to be used for achieving a precise approximation of the connection behavior.

*Corresponding author, Ph. D. Candidate, E-mail: hooman_farrokhi@yahoo.com
[a]Assistant Professor
[b]Assistant Professor

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 | 100×200×12 | E7018 | E6013 | NA | NA |
| SPC2 | Evaluation | 2IPE270 | IPE300 | Cover Plates | 200×400×16 | 50×240×8 | E7018 | E6013 | NA | NA |
| SPC3 | Proposal 1 | 2IPE270 | IPE300 | Enhanced reduced Cover Plates | 200×400×16 | 50×240×8 | E7018 | E6013 | 4 | 6 |
| SPC4 | Proposal 2 | 2IPE270 | IPE300 | Enhanced reduced Cover Plates | 200×400×16 | 50×240×8 | E7018 | E6013 | 5 | 10 |

Before the Northridge (1994) earthquake in US and (Kobe 1995 earthquake in Japan), it was assumed that the standard directly welded connection detail will meet all the structural requirements to withstand strong ground motions. The Northridge earthquake damaged many steel moment resisting frame buildings (SAC 1996). The unexpected performance of the buildings effectively invalidated the building codes and provisions.

After the earthquake, the UBC removed the pre-qualified status of such connections which urged the development of research programs to investigate the connection behavior and to recommend new detail designs in case it is needed. Many researchers individually performed valuable investigations on steel moment connections (Ashtiani 1996, Astaneh and Nader 1991, Clifton *et al.* 2004, Elghazouli 2000). Others contributed in national research projects (FEMA 267 1995, SAC 1996). The outcome was the development of the general recommendations and interim guidelines to evaluate, repair and design of steel moment frame buildings.

The constructional limitations, structural geometries or economic purposes in different areas of the world may urge structural engineers to use different connection details. In most cases, the connections are designed by means of a force based terminology with no experimental proof which results in significant overestimation. This clarifies the viable demand to investigate the current state of the construction industry enjoying different connection types.

In this paper, thorough experimental and Analytical evaluations of two common connection types are implemented and their nonlinear responses have been discussed. Finally, an alternative detail is proposed to eliminate the 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.

## 2. Experimental approach for seismic evaluation of moment connections

In the current study, full-scale specimens are designed and constructed in accordance with the current design codes and recommendations (AISC-ASD 2005, AWS D1.1 2008, Mazzolani and Piluso 2004, Naeim 2004) to provide a comparative database. The experimental setup typically consists of a T shaped beam to column connection. The schematic view of the testing assembly is provided in Fig. 1.

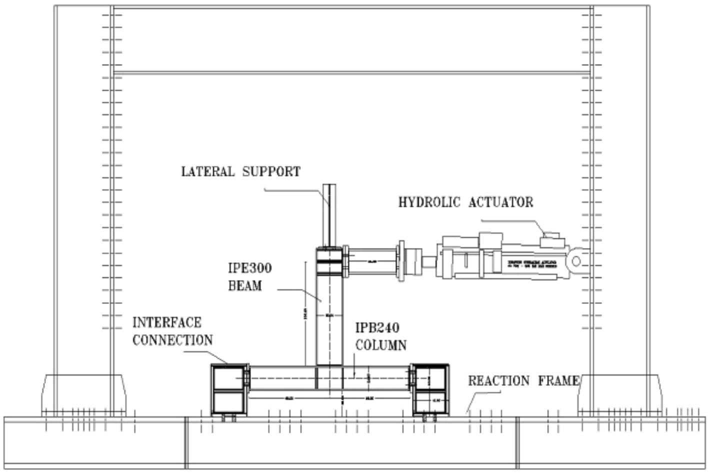


Fig. 1 Testing assembly

Several material testing specimens are also provided in accordance with the ASTM standard to reveal the stress-strain relationship of the material commonly used in numerical analyses and test calibrations. Strain gauges and displacement transducers are instrumented at the decided locations based on the numerical predictions. Quasi-Static cyclic tests were conducted to investigate the global and local response and to reveal failure mechanisms of the connection types. Calibrating the finite element program with the experimental results, the parametric studies on the sensitive parameters are then implemented to present a valuable feedback to the hypothetical basis.

## 3. Finite element approach for the seismic evaluation of moment connections

Nonlinear finite element models shall be used to predict structural seismic behavior of welded and bolted structures (Saravanan *et al.* 2009). In the current study, finite element models of the connections are generated using the FINNL3D finite element code, facilitating hierarchical solid and shell elements enjoying a non-associated flow rule to predict the nonlinear behavior. Large deformations are considered in numerical modeling of stiffness matrix to simulate the buckling modes. Also large strains are considered to simulate the significant yielding of plate elements.

Primarily, a monotonic incremental nonlinear analysis is performed to approximate the initial stiffness and the yield displacement.

A cyclic nonlinear analysis is then conducted using an increasing saw-tooth load pattern (ATC24 1992). Material models and numerical parameters are then calibrated with the experimental specimen to provide identical results. Afterwards finite element models are used in analyzing the sensitive parameters and finally to recommend probable modifications to improve the performance. Due to the general shortcoming of the finite element method in modeling the crack propagation, complete prediction of the specimen behavior is usually impossible or inaccurate. Smeared crack solutions generate significant approximations and discrete crack methods need the actual location and geometry of the crack, which is usually unknown. Fracture mechanic based methods are still being developed which is out of the scope of the current study.

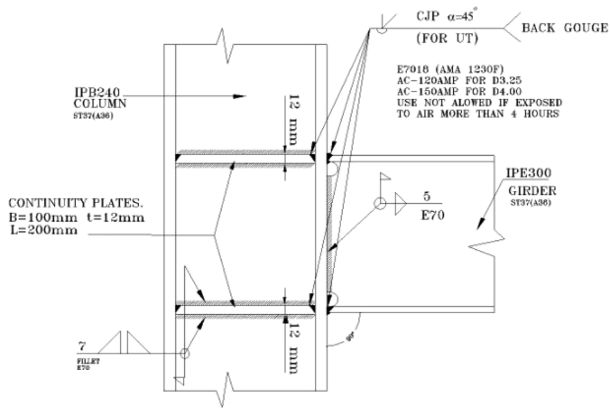


Fig. 2 SPC1- Direct flange groove weld

## 4. Moment resisting connections with direct flange groove weld (SPC1)

The moment resisting connections with direct flange groove weld were one of the most popular connection details in US and other countries before the Northridge earthquake in 1994. This specimen is proposed to narrate the current experimental knowledge.

The specimen dimensions and alignment elevations can be schematically seen in Table 1 and Fig. 2. The Geometry is generally similar to the specimen tested by Calado and Mele (2004). Material specimens were extracted from beam and column flanges and web to reach the exact stress-strain relationships of the elements. The results are shown in Fig. 8.

In addition, a finite element model is also generated using FINNL3D nonlinear finite element software to approximate the stiffness, strength, displacement capacity and the overall strain and stress distribution in the specimen. While the results are calibrated with the experimental measurements, the software may be used to predict other cases afterwards.

The finite element analysis results show accumulated stress 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. 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 performance.

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 up to 25% while the final rotation capacity is reached. The hysteresis loops are typically convex and consistent with a gradual evolution of the hysteretic areas. The moment rotation diagram of the specimen can be seen in Fig. 3. The deformed shape of the tested specimen is shown in Fig. 4.

This design strategy features a rigid full strength moment resisting connection with a local rotational ductility ratio of about 7.9 and moment capacity of 217 kN.m. This, covers 92 percentage of the beam moment capacity. The connection is understood as a proper option to be used in ductile earthquake resisting frames with a mandatory focus on welding procedure, and quality control. The most sensitive factor affecting the performance of such connections is the completeness of the

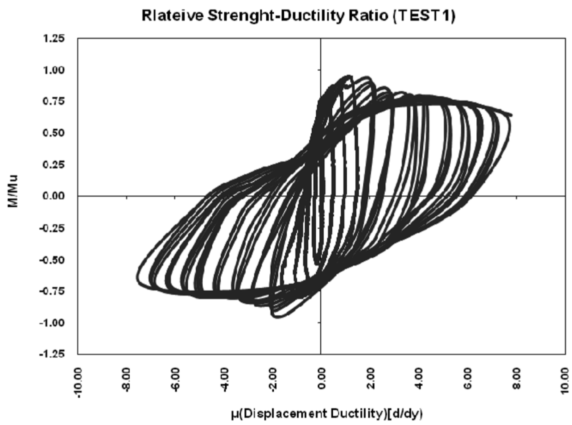


Fig. 3 Hysteretic relative moment rotation behavior of specimen 1

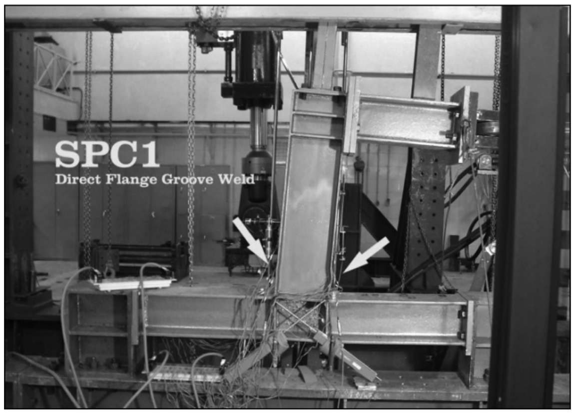


Fig. 4 Specimen 1 at final hysteretic displacement amplitude

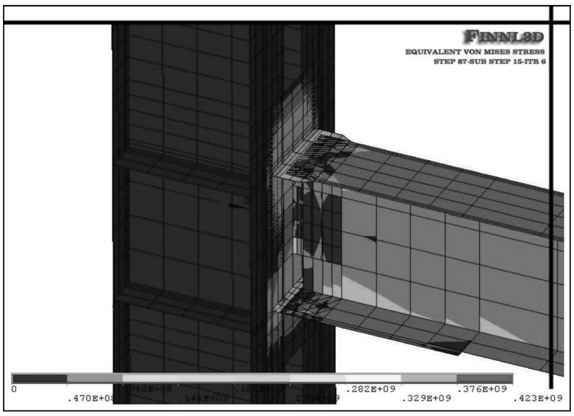


Fig. 5 Equivalent Von Mises stress contours of specimen 2 at final displacement amplitude

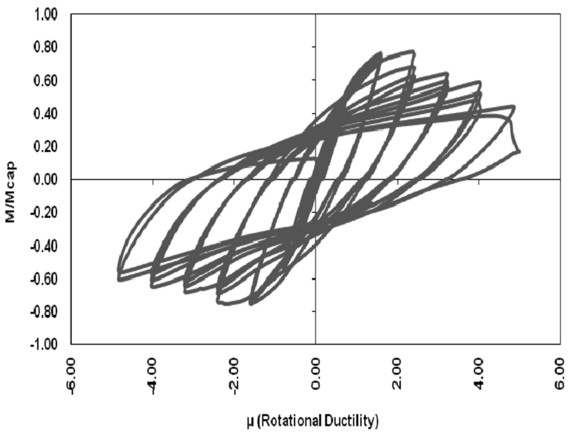


Fig. 6 Hysteretic relative moment rotation behavior of specimen 2

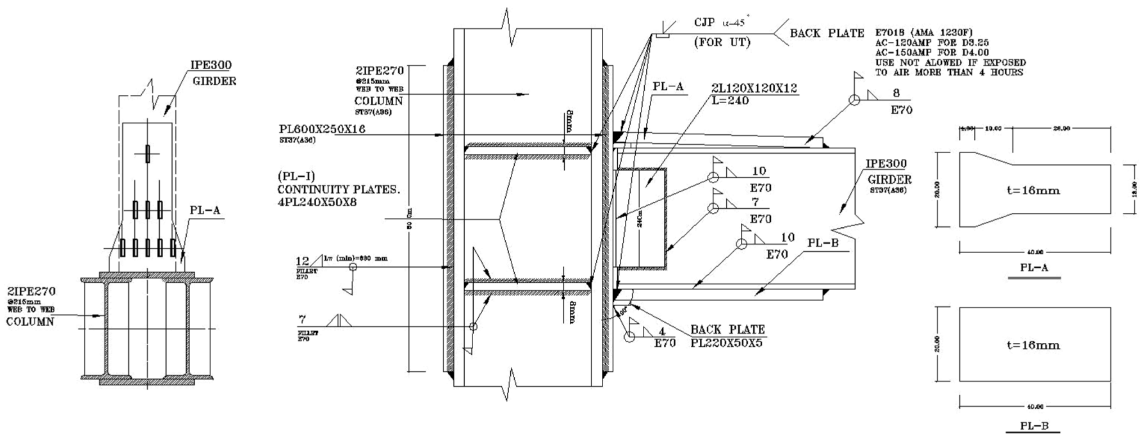


Fig. 7 Specimen 2- Cover Plates connected to an end plate welded to a Double profile Column

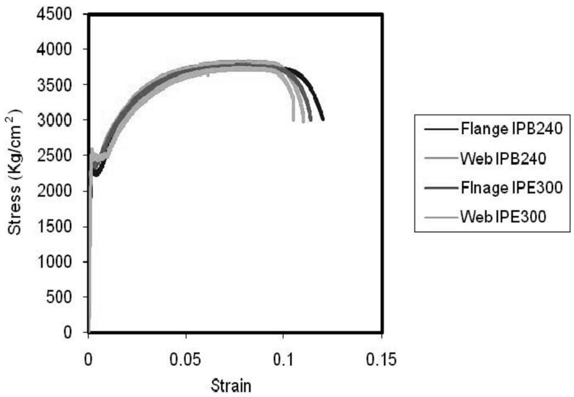


Fig. 8 Stress-strain curves of beam and column material

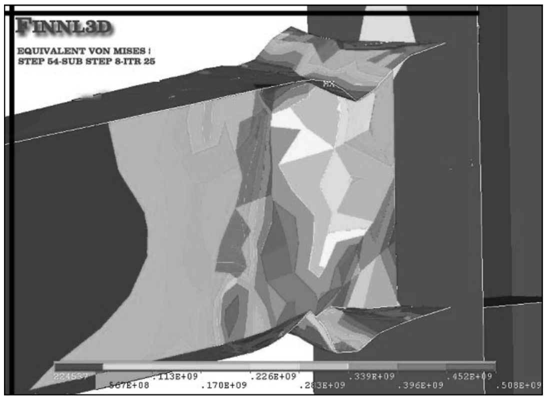


Fig. 9 Equivalent Von Mises stress contours of specimen 1 at final displacement amplitude

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.

## 5. Moment Resisting Connections with cover plates welded to a double profile column (SPC2)

Due to its constructional versatility and simple fabrication benefits, this type of moment resisting connections have 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 and fillet welds to the beam flange.

When using double I sections 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 induces forces with no major negative effect on the total performance.

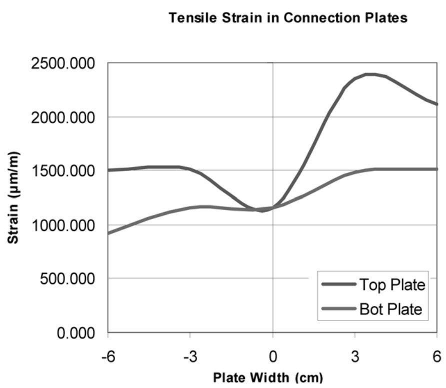


Fig. 10 Stress distribution against cover plate width-specimen 2

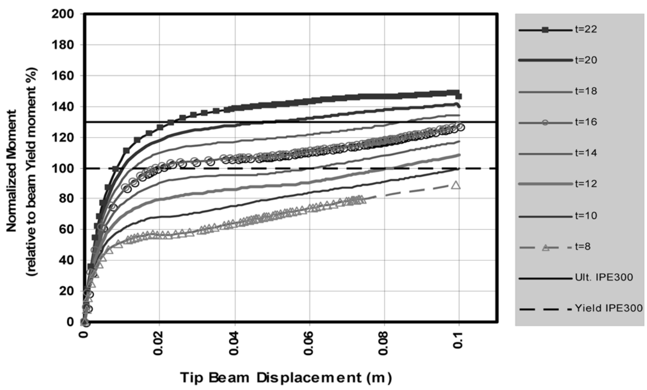


Fig. 11 Effect of the end plate thickness on the connection capacity

Structural engineers, usually consider full moment capacity and sufficient rigidity for the connection to be used in a ductile earth quake resistant frame which is not usually the case. 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 brittle fracture.

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. Although the geometry and thickness of the cover plates as well as the end plate, directly will affect the connection stiffness and moment capacity, still The final failure mode and the rotational capacity is controlled by the fracture in weld roots. The nonlinear behavior of the connection assuming different thicknesses for the end plate can be seen in Fig. 11.

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.

As it can be noticed in Fig. 12, the increase of the plate thickness enhances the out of plate bending stiffness of the plate. It also boosts the ultimate moment capacity of the connection. In other words in connections with thin end plates the whole nonlinear action is limited inside the end plate in bending while in connections with thick end plates the end plate is typically isolated from nonlinear deformations and the yielding is shifted to the cover plates.

The rotational rigidity is about 70 percent of the ideal rigid connection when designated plate thickness is used. It will increase up to 95 percent by increasing the thickness. The non-uniform tapered geometry of the top plate also disturbs the stress flow at the region. Based on finite element analyses performed, a severe stress concentration exists near the reduced section. Still, the most stressful region is the weld root, which is unfavorable. The stress distribution diagram through the top and bottom plate widths may depict the discussed stress concentration (Fig. 10).

Premature failure is predicted from weld roots propagating 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.

Cover plate thickness directly affects the ultimate capacity and stiffness of the connection but the increase rate will drop as the thickness grows. Dimensions chosen based on simple design guides

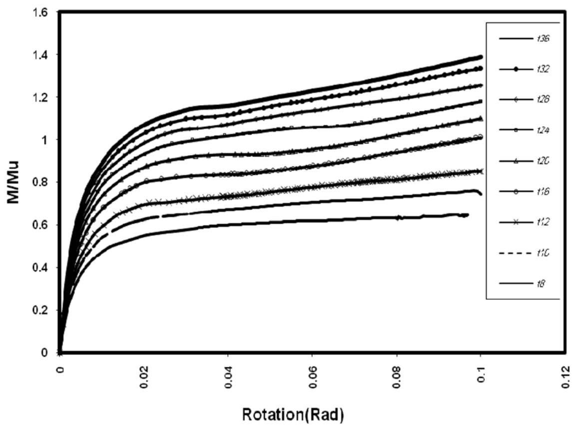


Fig. 12 Effect of cover plates thickness on connection capacity


Fig. 13 Specimen 2- Complete cracking of weld root adjacent the Copper Back weld plate


Fig. 14 Specimen 2- Bucking of the end plate and Crack propagation from the weld root

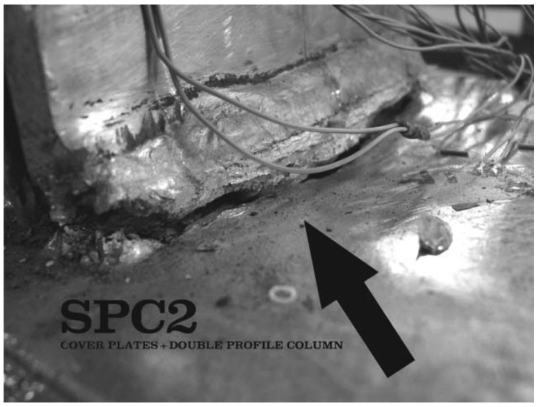


Fig. 15 Specimen 2- Through the thickness crack propagation in the end plate

seems to be acceptable. The cyclic test performed on the specimen accredited the finite element analysis.

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 resulted in total failure of the connection, the buckled zone of the end plate and the crack propagation through the plate thickness may be seen in Figs. 13, 14, 15 respectively.

The connection seems to be unreliable to be used in ductile earthquake resisting frames. Insufficient moment and ductility capacity compromising with rapid drop in stiffness and strength with no previous warnings, 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.

## 6. The proposal to the new enhanced detail (SPC3 and SPC4)

One of the most interesting concepts applied by many researchers to avoid premature connection failure is to shift the nonlinear mechanisms from the connection edge. One method to approach this goal 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 (Chou and Wu 2007) this may cause stress concentrations in the beam web. Some other, proposed cast connections with variable flange thickness (Fleischman *et al.* 2007) although it seems effective, it is very expensive and hard to fabricate. Also the use of channel or angle connectors welded to beam column flanges is proposed by other researchers which seems strongly case sensitive (Kumar and Rao 2006).

A good alternative to create the weak point is the use of reduced cover plate sections. The yielded 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 might be replaced with new ones, while the other parts may remain undisturbed.

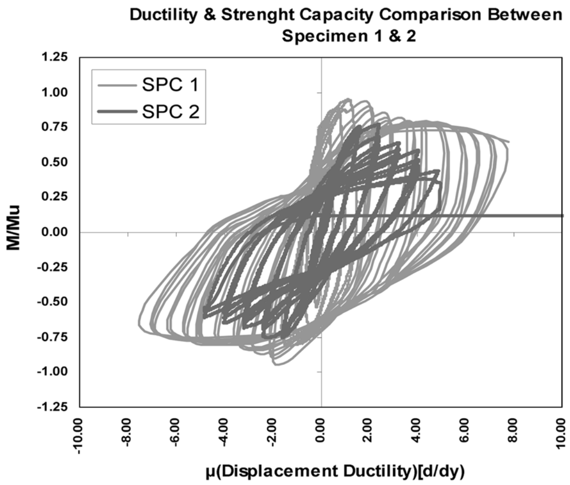


Fig. 16 Relative Moment Rotational Ductility Diagram of specimens

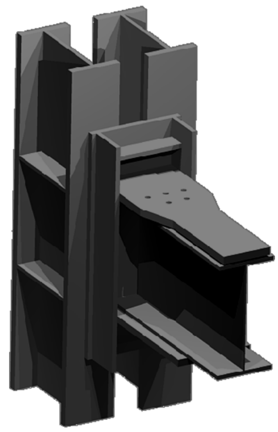

Fig. 17 3D view of the proposed connection

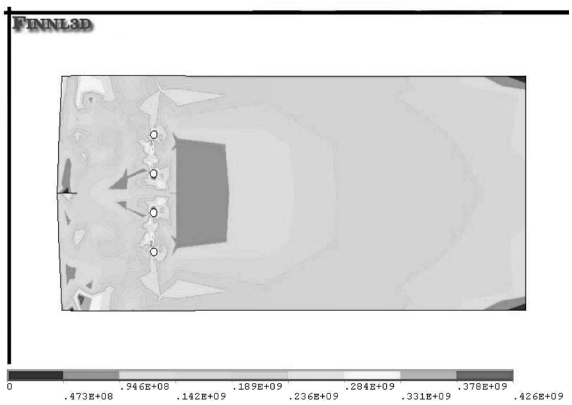

Fig. 18 Equivalent von misses stress contours in bottom cover plate (SPC3)

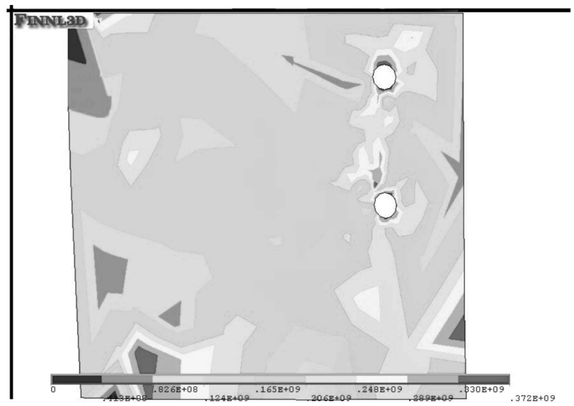

Fig. 19 Equivalent von misses stress contours demonstrating the stress concentration around holes (SPC4)

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.

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 which are imposed by cover plates. To reach this goal, 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

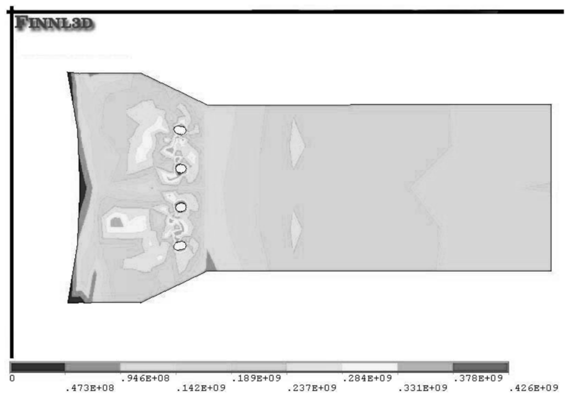


Fig. 20 Equivalent von misses stress contours in upper cover plate (SPC3)

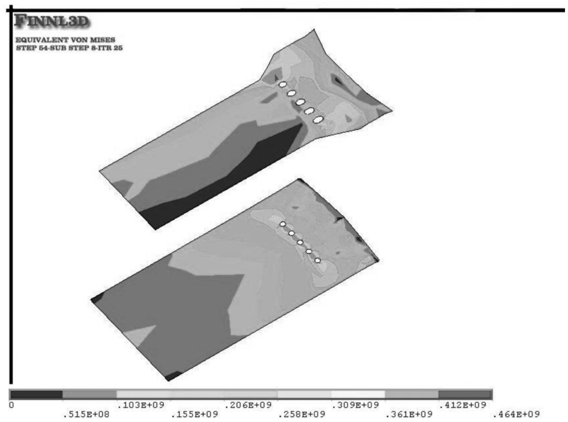


Fig. 21 Stress contours at cover plates in displacement amplitude $d$=2$dy$ (SPC4)

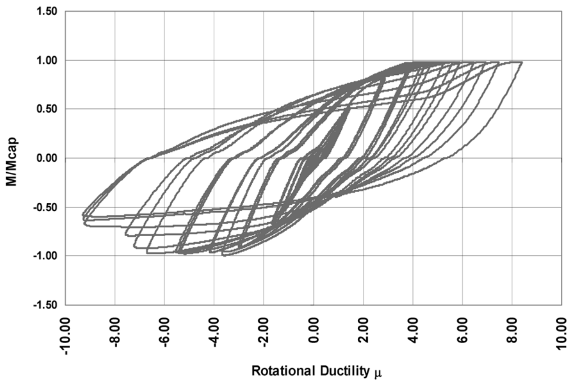


Fig. 22 Hysteretic behavior of SPC3

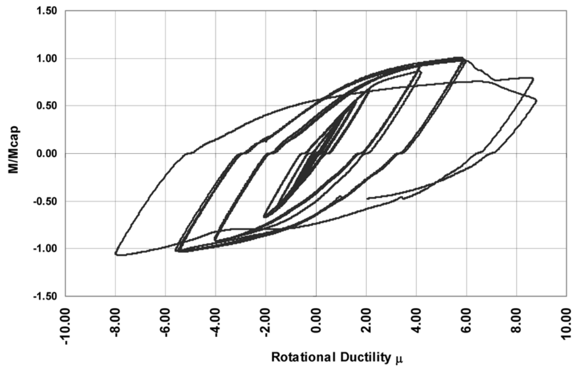


Fig. 23 Hysteretic behavior of SPC4

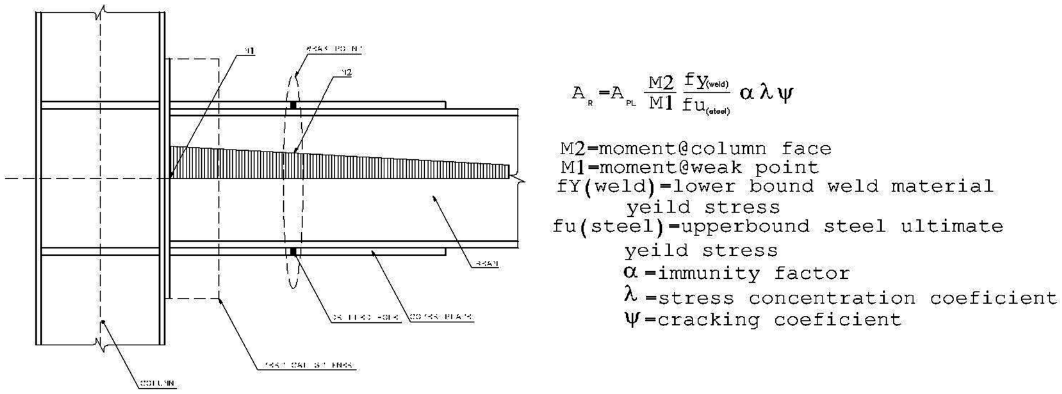


Fig. 24 Proposed design terminology

yielding mechanism is expected to occur prior to the weld cracking.

The plate and holes may be designed and dimensioned using this method to approach this goal. The design concept may be expressed as 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 fracture. This can implement a required immunity factor to consider imperfections. The mentioned terminology may be expressed in terms of the following simple formula shown in Fig. 24.

The factor $\alpha$ is calibrated with finite element analyses to simulate the required stress concentration which is needed to shift the critical action field. The stress concentration coefficient ($\lambda$) is meant to demonstrate the stress concentration effect intentionally created around the holes. It is calibrated based on finite element analyses conducted in different models. This coefficient is calculated to be 0.870 for the proposed geometry. The cracking coefficient ($\psi$) takes the cracking phenomenon into account and makes a reliability boundary to avoid premature weld fracture. This coefficient is equal to 0.795 for the specified welding procedure and material properties. Immunity factor is considered 1.00 and 0.80 for SPC3 and SPC4 respectively.

SPC3 and SPC4 are fabricated typically at 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.

Drilled holes are located 15 cm away from the connection face to provide intentional stress concentrations and to 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 of 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

Fig. 25 Initiation of cover plate buckling ($d = 1.5dy$)

Fig. 26 Cover plate buckling and start of yielding around holes

Fig. 27 Net section yield of the upper cover plate

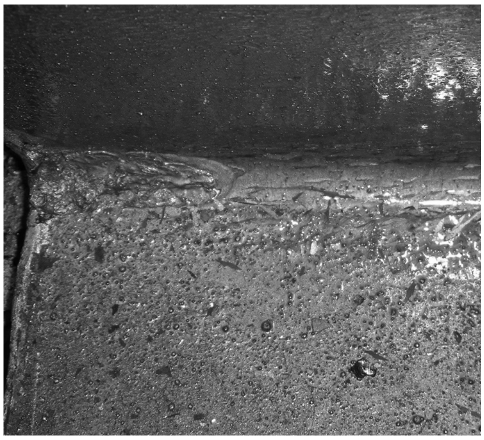

Fig. 28 The uncracked penetration weld root

Fig. 29 Necking of the bottom cover plate

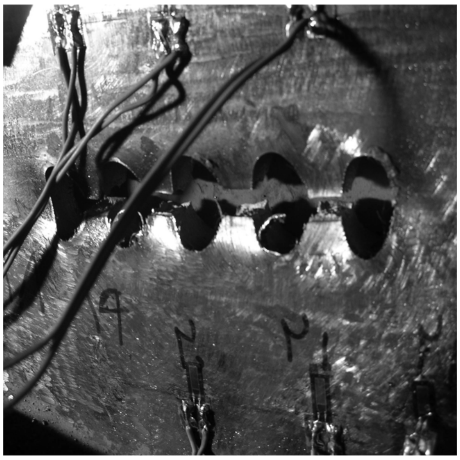

Fig. 30 Yielding around bottom cover plate

specimens. SPC3 was proposed to experience the boundary between two failure modes. In this state it is expected that brittle cracking mode is altered to ductile yielding. Therefore the specimen shall experience the interaction of these 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.

The energy dissipation and the form of hysteretic loop 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 (SPC2).

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

Table 2 Test results

| Specimen No. | $\theta_y$=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* (%) | $\theta$u=Max imum rotation (Rad) | $\mu$=Rotational Ductility Factor= $\theta u/\theta y$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 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 |

roots (Figs. 25, 26, 28). Repeated yielding is observed in constant amplitude load cycles. The experiment validated the design terminology. No cracking were observed in weld roots and the adjacent areas while large plastic deformations exist around holes (Figs. 27, 30). Necking of the cover plates and net section yield were anticipated around holes (Fig. 29). Hysteretic loops are wide and consistent. Therefore, excellent energy dissipation was resulted. The connection delivered 99% of the expected moment capacity demonstrating a ductility ratio about 8.8, which is quite satisfactory. The mechanism is reliable and will form regardless of constructional aims. The proposed terminology shall be used to design similar connections with different geometries.

## 7. Conclusions

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

(1) Moment resisting connections with direct beam flange welds feature sufficient stiffness, strength and a suitable rotational ductility capacity. It can be used as a rigid full strength ductile connection for steel building structures. In case of significant inclusions like micro cracks, porosity, partial penetration and etc., premature failure in form of crack initiation is possible to occur.

(2) Cover plates have valuable constructional benefits. They omit the buckling mode of the beam flanges or shift it to the plate end; on 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 conclude to the premature brittle fracture of the connection and hence the poor seismic performance. Ultimate moment and rotational ductility capacity of the connection is decreased down to 25% and 36%, respectively in comparison with SPC1 (Fig. 16).

(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 far enough from the connection face. In this region, a stable yielding mechanism is viable to occur.

(4) The proposed connection has a favorable and reliable performance with an expected plastic region within the cover plates. While the ultimate 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. 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.

(5) It is possible to define a reliability boundary coefficient for the proposed connection type, in terms of the ratio of the plate capacity near the holes to the total plate capacity, namely a plate void ratio. This coefficient shall be used in design of such connections with sufficient confidence that no cracking may occur and ductile yielding mechanism is expected regardless of constructional imperfections.

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