Panel zone detailing influences on SMRF cyclic behaviour considering unequal beam depth

Peyman Shadmanheidari
East Tehran branch, Islamic Azad University, Tehran, Iran

R. Ahmady & B. H. Hashemi
Structural Research Centre ,International Institute of Earthquake Engineering and Seismology(IIEES), Tehran, Iran

H. Kayhani & Pouya Shadmanheidari
Science and Research Branch, Islamic Azad University, Tehran, Iran

SUMMARY:

This is an ongoing research about the role of continuity plate arrangement and its detailing on PZ performance of SMRF system. PZ with unequal beam depths appear to be a special case of connection detailing, that has not received enough attention so far and could lead to complications in everyday engineering practice The results of recent studies have shown that the seismic performance of such frames can be improved by paying particular attention to the detailing of their panel zones (PZ) and beams. Four full-scale experiments of connections with unequal beam depths were performed, with different continuity plate arrangements (inclined and straight plates), and different corner clip lengths. The results have shown that the correct selection of inclined or straight continuity plates, with special detailing of the PZ, could remain the PZ behaviour within safe margins.

Keywords: continuity plate, corner clip length, doubler plates

1. INTRODUCTION

The Special Moment Resisting Frames (SMRFs) are ductile and deformable structures, and can therefore provide the structural characteristics which are needed for acceptable seismic design. The controlled inelastic deformation of such frames is dominated by the high ductility potential of the beams and the panel zone, so that the corresponding seismic design requirements focus on these elements (AISC (2005*a*)). For example, there are some limits on the minimum requirements for PZ doubler plates and on the length by which they can extend beyond the continuity plates, as well as on the continuity plate corner clip dimensions and the minimum requirements for continuity plate thickness, all of which can contribute to stable shear mode plastic deformation of the PZ.
studies on PZ behaviour were initiated in the late 1960's and early 1970's to comprehend the inelastic behaviour of joints in moment-resisting frames (Naka et. al (1969), Fielding and Huang, (1971); Krawinkler et al. (1971), Bertero et al. (1973), and Becker (1975)).These authors indicated that the contribution to the top displacement of the sub-assemblage was highly influenced by the panel zone distortion, also they pointed out the importance of considering the influence of joint deformations in frames in terms of stiffness and energy absorption. In addition, the effect of axial loads on the performance of connections subjected to shear was also carried out. Several years later, a number of test were performed by Slutter (1988) and Popov (1985) and (1987) in order to verify the extreme loading conditions on joints and to study the cyclic behaviour of large beam assemblies. These authors found that the column flange could contribute to the nonlinear behaviour of the PZ. They also showed that the PZ can have a high reserve of strength after yielding, high ductility and significant strain hardening properties.
These observations with regard to the high nonlinear capacity of PZ resulted in a decrease in the PZ demand that was specified in SEAOC (1987) and ICBO (1988). These codes therefore capped at 80% the transferred shear corresponding to the beam plastic moment, but large distortions of the PZ could lead to local kinking of the column flange at the corner of the joints, where the beam flanges are connected to the column flanges. Results of a study by Tsai and Popov (1988) indicated that panel

zones designed according to above-mentioned provisions could undergo large inelastic shear distortion before reaching their rated shear capacity. Weak PZ behaviour may have played a role in the failures which occurred during the Northridge earthquake (FEMA-267A (1997)). Participation of the panel zone to the inelastic response contributed to the reduction of the demands on the beams in terms of deformation (Lee C.H. et al., (2005)).Analogous research was carried out in Europe by Dubina et al. (2001) and Ciutina and Dubina (2006) to understand the cyclic performance of beam-to-column joints. In FEMA-355D (2000*a*), which originated from the SAC joint venture, the proposed method of design was substantially altered so that it became completely different to that specified in previous codes (AISC (1997)), being based on the idea that the framing beams and panel zone should yield at the same time in order to achieve balanced behaviour. It defined the yield point of the beam and of the PZ as the base-line for this balanced condition. Jin and El-Tawil (2005) were, however, of the opinion that this method was unsatisfactory since such balanced beam and PZ capacity could not guarantee controlled distortion of these elements, and that it would not be possible to establish simultaneous yield mechanisms in the beam and PZ.
This study aims (Ahmady and Hashemi(2012)) at investigating the seismic response regarding differences in connection detailing for this special case. In the present research, experimental studies were conducted to investigate the effect of different continuity plate arrangements and detailing for various configurations on PZ seismic performance.

2. DESCRIPTION OF THE EXPERIMENTAL STUDIES

Section Four tests were conducted in order to improve present understanding of PZ detailing with unequal beam depths, using plate girder sections. The sub-assemblages were full-scale simulations of SMRF at the connection region, which were extracted from the frame assuming that the inflection point was at the mid-point of all the elements. The dimensions of the column flange and web sections were 25×1.5 cm and 32×1.0 cm. respectively, whereas those of the deep beam flange and web were 15×1.0 cm and 48×1.0 cm, this beam being referred to as "Beam 50". The dimensions of the shallow beam flange and web sections were 15×1.0 cm and 28×1cm, this beam being referred to as "Beam 30". Based on these provisions, the doubler plates had dimensions of 70×32×1.2 cm, and the continuity plates had a thickness of 1 cm, except for the inclined continuity plates which had a thickness of 1.2 (AISC (2005*a*)), also this the PZ thickness satisfied the balanced condition of the connection and the PZ as stated in FEMA-355D (2000*a*). The thickness of the cover plate connection was calculated according to FEMA-267 (1995) and the AISC (2005*b*).

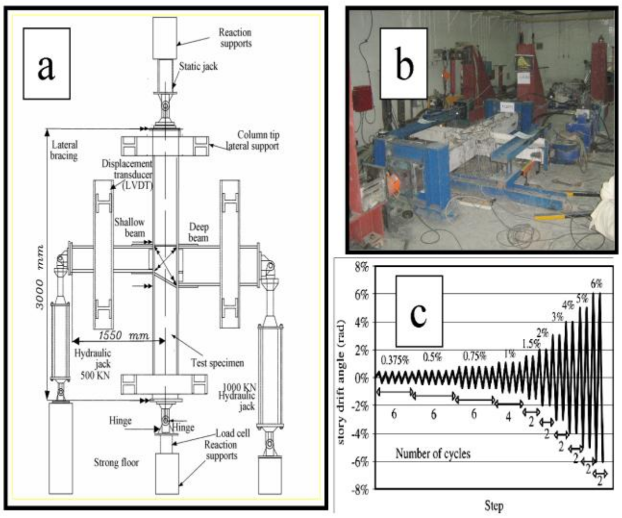

Figure. 1. (a) (b) Schematic and actual view of the test set-up (c) the load

As shown in Table 1, as well as in Figures 1 and 2, the test specimen U1-FW1 consisted of two unequal beams, with an inclined lower continuity plate which was fitted to meet the beam's bottom flanges. The corner clip length of the continuity plates was designed according to the AISC (2005 *a*), and amounted to 5 cm. Test specimen FW1 was similar to test specimen U1-FW1 except that the corner clip length of the continuity plates was not designed according to the AISC (2005 *a*), and amounted to only 2.5 cm. The test specimen U1-FW2 also consisted of two unequal beams, but had three straight continuity plates, whose corner clip lengths were designed according to the AISC seismic provisions (2005). Test specimen FW2 was similar to test specimen U1-FW2, but the corner clip lengths of the continuity plates were not designed according to the AISC (2005*a*), and amounted to 2.5 cm. The doubler plates were welded to the column flanges using fillet-welded joints. Due to the existence of a continuous fillet weld line for the column section fabrication on both sides, the doubler plate to column flange weld line could not be a CJP weld type.

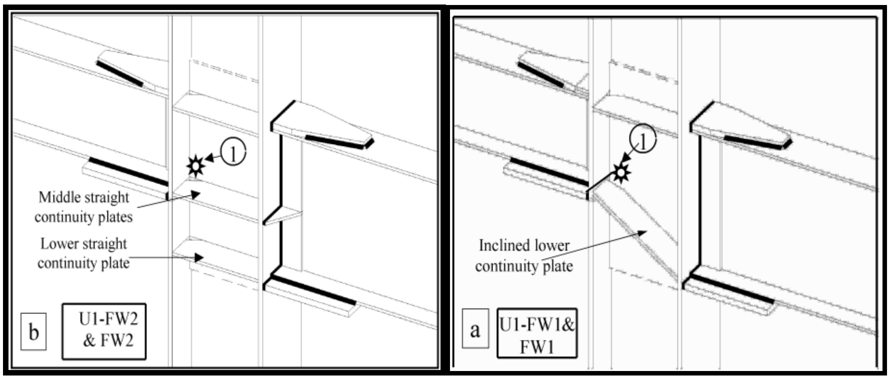

Figure 2. (a) test specimens U1-FW1 and FW-1, and (b) test specimens U1-FW2 and FW2, showing the locations of rosette strain gage(asterisk).

A constant axial load equal to 40 tons was applied and measured by a static jack and load cell to the top of the column (this load corresponded to a compressive stress value of $0.3f_a$, where f_a is the column's allowable compressive stress, in order to simulate the axial load effects. The bottom end of the column was pinned to the laboratory strong floor, and out-of-plane buckling was prevented by devices that were installed at the mid-point of the beam spans. The specifications of the test specimens, including their material properties obtained from coupon tensile tests, the continuity plate arrangements, and the corner clip lengths are presented in Table 1. Strain gauge No. 1 was fixed to the doubler plates, close to the latter's fillet weld line and the intersection of the column flange and – in the case of test specimens FW1 and U1-FW1 – the inclined continuity plate, or, in the case of test specimens FW2 and U1-FW2, the middle straight continuity plate, in each case on the shallow beam side.

Table 1.1 Investigated test specimens Ddata

Designations of the test specimens	Deep Beams		Shallow beam		coloumn		Lower continuity plate arrangement	Corner clip length
	Flange dimensions	web dims.	Flange dims.	web dims.	Flange dims.	web dims.		
U1-FW1	15×1	49×1	15×1	29×1	25×1.5	32×1	Inclined	5*
U1-FW2							Straight	5*
FW1							Inclined	2.5
FW2							Straight	2.5
1-The unit is cm (* refers to the AISC seismic provisions(2005) recommended values								

3. EXPERIMENTAL RESULTS

During the first 19 cycles, the behaviour of test specimens U1-FW1 and FW1 was in the elastic range.

The PZ of these two test specimens yielded earlier than the beams. Both test specimens then underwent inelastic behaviour, but at storey drift angles of 0.04 and 0.05 radians, corresponding to the 30th and 32nd cycles, no significant flange buckling of the deep beam had yet been observed. However, in the case of test specimens FW1 and U1-FW1, column web buckling in the proximity of the PZ geometric centre, i.e. intersection of diagonals of PZ’s trapezoidal shape, was observed, which amounted to 10 mm by the end of the test. At a storey drift angle of 0.05 radians a sudden rupture occurred in the bottom flange of the deep beam of test specimen FW1, close to the end of this beam's bottom cover plate, and the test was terminated. In the case of test specimen U1-FW1, a crack initiated along the weld line of the column flanges connection to the doubler plates, close to the bottom flange of the shallow beam, and ran upwards along the weld line (this crack is indicated by an arrow in Figure 3(a)). This crack began to appear at a storey drift angle of 0.04 radians, and propagated until a storey drift angle of 0.05 radians was reached.

During the first 20 cycles, test specimens U1-FW2 and FW2, also showed elastic behaviour. At the 21st cycle, the deep beams of each of the two test specimens underwent inelastic behaviour, but without any significant buckling of the beam flanges or webs, so that there was no degradation in the seismic behaviour. These test specimens failed due to flange fracture of the deep beam's bottom flange at a storey drift angle of 0.05 radians, and the tests were terminated.

In the test specimens with straight continuity plates (U1-FW2 and FW2), no cracks occurred in the PZ area during the experiment even though they had done so in the case of test specimen U1-FW1. It seems that the sensitivity of the PZ boundaries to detailing is less in the case of straight continuity plates than in the case of inclined continuity plates.

From Figure 4(b) it can be seen that, in the case of test specimens FW2 and U1-FW2, at location 1, maximum microstrains (the measured strain direction is parallel to the main axes of the beams) of 16700 and 17200 were obtained, whereas in the case of test specimens FW1 and U1-FW1 maximum microstrains of 10300 and 7450, were reached at the same location, respectively. It can also be seen from Figure 4(a) that a large reduction in the strain measured at location 7 occurred suddenly, in the case of test specimen U1-FW1 and at a storey drift angle of 0.04 radians, which could indicate the occurrence of crack initiation at the weld line of the doubler plates to the column flange.

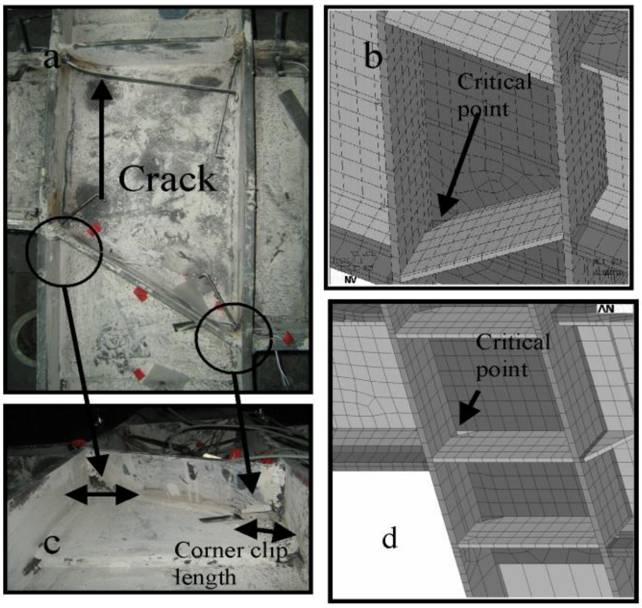

Figure 3. (a) crack initiation in test specimen U1-FW1 (front view, after the experiment) (c) the actual continuity plate detailing of specimen U1-FW1 (corner clip length 5 cm). (b)(d) The FE models and the locations of critical points for the test specimens FW1 and FW2.

4. ANALYTICAL STUDY

On the basis of the results of the described experiments, a comprehensive series of elastic and inelastic analyses was performed using ANSYS (1998) (Structural Analysis Software). Numerical modelling of the connection was carried out using eight-node first order SOLID45 elements for the material and the welds. Large displacement element formulations were used to simulate buckling of the PZ, the beam flange and the beam web, as well as local kinking of the column flanges. A multi-linear kinematic hardening plastic material model was used to simulate the cyclic inelastic behaviour. The material properties used in the analyses were based on the true stress-strain relationships obtained in the experimental research. A comparison of the responses of the analytical models and the experimental results has shown that the FE models can accurately predict the inelastic response of the test specimens. Fig 3(b) and (d) shows a FE model of test specimen U1-FW1 and U1-FW1. The FE models were used to estimate the local response (i.e. the seismic behaviour of the PZ), as well as the global response (i.e. the response of the beams) with relatively small errors (less than 7%). These verifications are shown for the PZ of test specimens U1-FW1 in Fig 4(a). In the case of all the test specimens, the observed and predicted resistance determined from the inelastic FE analysis correlated well with the experiment.

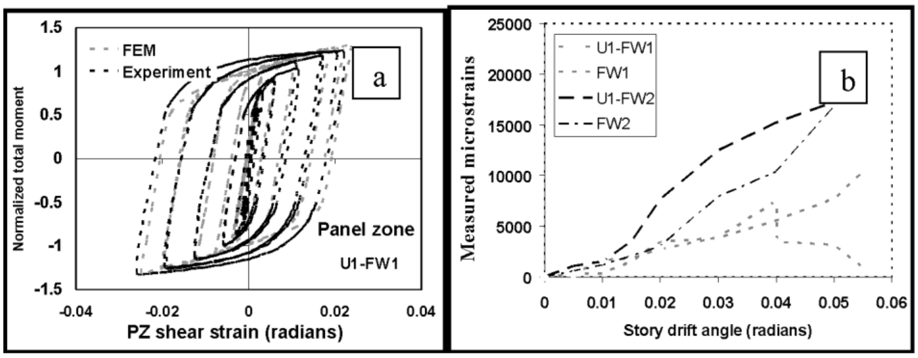

Figure 4. (a)Comparison of the analytical and experimental response of the PZ for test specimens U1-FW1 (b)

The results of the FE analyses correlated well with the local behaviour due to the consistent prediction of the location, initiation and extent of local yielding, as shown in Figure 4(a). Another parameter which was in good agreement with the results of the experiments is the equivalent plastic stress. Taking into account the presented figures, this parameter can be used to predict the yield distribution area with acceptable accuracy. It is defined by Eqn. 4.1:

$$\sigma_{eqv} = \left(\frac{1}{2} \left[(\sigma_x - \sigma_y)^2 + (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_y)^2 + 6({\sigma_{xy}}^2 + {\sigma_{xz}}^2 + {\sigma_{yz}}^2) \right] \right)^{\frac{1}{2}} \tag{4.1}$$

where σ_x and σ_y, etc. are the stress components. The equivalent plastic stress results are displayed using the stress contour plots shown in Figure 5(a) to 5(c). It is clear that the surface stress distribution may vary within the thickness of the steel, but a comparison has been made for the surface which demonstrates the greatest magnitude of the local stress state.

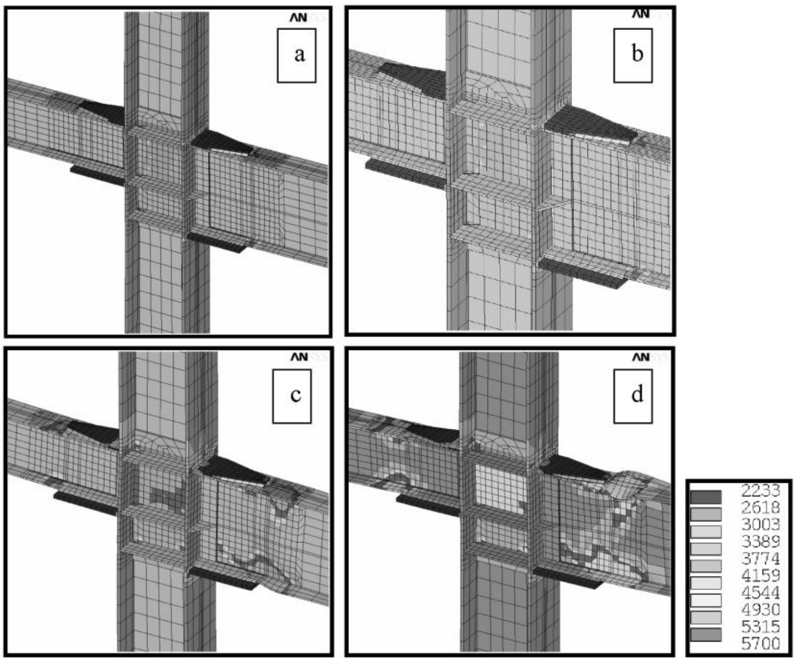

Figure 5. The equivalent plastic stress distribution of the analytical model corresponding to test specimen U1-FW2 at storey drift angle of (a)0.02 radians (b)0.03 radians (c) 0.04 radians(c) 0.05 radians.

5. EVALUATION OF DIFFERENT CONTINUITY PLATE CONFIGURATIONS USING RUPTURE INDICES

In order to evaluate and compare the different connection configurations for ductile fracture potential, a *rupture index* (usually abbreviated to RI) was used, and its values were computed for the different cases. This same methodology and approach has been used by other researchers (e.g. El-Tawil et al. 1998, Mao et al. 2001, and Ricles et al. 2002, 2003). The rupture index is defined as:

$$RI = \frac{\varepsilon_{eqv}^{pl} / \varepsilon_y}{\exp\left(-1.5.\dfrac{\sigma_m}{\sigma_{eff}}\right)} \tag{5.1}$$

where ε_{eqv}^{pl}, ε_y, σ_m and σ_{eff} are, respectively, the *equivalent plastic strain*, the yield strain, the hydrostatic stress, and the equivalent stress (also known as the Von Mises stress). A large tensile (negative) hydrostatic stress is often accompanied by large principal stresses, and generally implies a greater potential for either brittle or ductile fracture.

The critical Points as shown in Figures 3(b) to 3(d) correspond to the same location in the different continuity plate arrangements. These points are located in a high plastic stress demand region of the panel zone for the different continuity plate arrangements. Both geometrical and material non-linearities were considered in the analyses of the models.

Figure 6 shows values of Rupture index for the critical point of analytical models correspond to the test specimens; In the case of the model of test specimen U1-FW1, the rupture index reaches a value of 4.24 at a storey drift angle of 0.05 radians, whereas in the case of the models of the other test specimens about 50% smaller RI values are obtained. Taking into account this result, as well as the experimental data and computed analytical indices, it can be concluded that there is fair agreement between the experimental and analytical results, so that the potential for cracking can be anticipated properly.

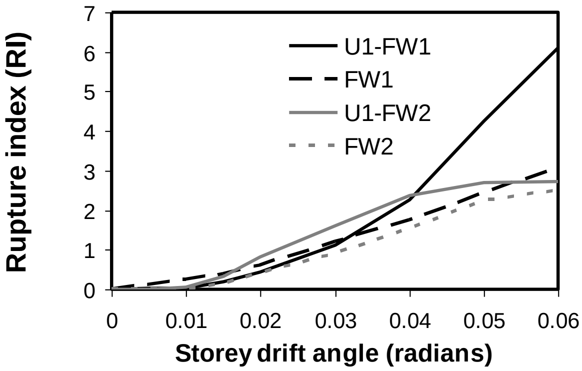

Figure 6. Computed values of Rupture index for the critical point vs. the storey drift angle.

6. CONCLUSIONS

The subject of this study has been neglected thus far in the codes for SMRF systems.The rupture of the doubler plates weld line to the column flange can be affected by many parameters, such as geometrical differences, the continuity plate arrangement and the corner clip length in this special case. Analytical models can be used to examine the sensitivity of each parameter. Regarding strain gauge data fixed to the PZs' corner and beams' flanges for all test specimens in the experiment, it appears that the yield sequence in the deep and shallow beams could be different, depending on the continuity plate arrangement. The interval between the onset of yielding in the beams and in the PZ was longer in the case of the test specimens with inclined continuity plates than in the case of those with straight continuity.

From the obtained experimental and analytical results, the proper corner clip length can be different for every case in unequal beam case and continuity plate arrangements, and there is no unique detailing for the reduction of crack initiation potential for all cases. It appears that the corner clip length which is recommended by the AISC 2005a, does not provide a safe margin for the inclined continuity plate arrangement with regard to the potential cracking of this weld line. The sensitivity of straight continuity plates, i.e. the middle and lower continuity plates, to corner clip length is less than in the inclined continuity plate arrangement, so that the AISC 2005a, detailing for this case is within safe margins.

REFERENCES

Ahmady Jazany R., Hosseini Hashemi B.2012.Effects of detailing on panel zone seismic Behavior in special moment resisting frames with unequal beam depths. *Canadian Journal Civil Eng.* **39(4):** 388–401.

American Institute of Steel Construction (AISC). 1997. Seismic provisions for structural steel buildings, *American Institute of Steel Construction*, Chicago (IL).

American Institute of Steel Construction (AISC). 2005a. Seismic provisions for structural steel buildings, *American Institute of Steel Construction*, Chicago (IL).

American Institute of Steel Construction (AISC). 2005b. Prequalified connections for special and intermediate steel moment frames for seismic applications. *American Institute of Steel Construction*, Chicago (IL).

American Welding Society (AWS). 2002. Structural welding code - steel. Miami (FL).

ANSYS.1998. User's manual, version 5.4. 201. Johnson Road, Houston, ANSYS Inc.

Becker R.1975. Panel Zone Effect on the Strength and Stiffness of Steel Rigid Frames. *Engineering Journal*, AISC, 12(1):19–29.

Bertero, V.V., Krawinkler, H., and Popov, E.P.1973. Further studies on seismic behaviour of steel beam-to-column subassemblages. *EERC Report No. 73-27*, University of California, Berkeley.

Ciutina, AL., and Dubina, D. 2006. Seismic behaviour of steel beam-to-column joints with column web Stiffening. *Steel and Composite Structures*, 6(6): 493-512.

Dubina, D., Ciutina, A., and Stratan, A. 2001. Cyclic Test of Double-Sided Beam-To-Column Joints.*Journal of Structural Engineering, ASCE,* 127(2):129–136.

El-Tawil, S., Mikesell, T., Vidarsson, E., and Kunnath, S.K. 1998. Strength and ductility of FR welded-bolted connections. *Report SAC/BD-98/01, SAC Joint Venture*, Sacramento ,CA.

Federal Emergency Management Agency (FEMA). 1995. Internal guidelines: Evaluation, repair, modification, and design of welded steel moment-frame structures. *Report FEMA-267*, prepared by the SAC Joint Venture for FEMA, Washington, D.C.

Federal Emergency Management Agency (FEMA). 1997. Interim Guideline. Advisory No.1, supplement to FEMA-267. FEMA-267A/Report SAC-96-03, *SAC Joint Venture*, Sacramento, CA.

Federal Emergency Management Agency (FEMA). 2000a. State of the art report on connection performance. FEMA-355D, prepared by the SAC Joint Venture for FEMA, Washington, D.C.

Fielding, DJ., and Huang, JS.1971.Shear in steel beam-to-column connections. *Welding Journal*,50(7):313–326.

International Conference of Building Officials (ICBO). 1988. Uniform building code, 1988 Ed., ICBO, Whittier ,CA.

Jin J., and El-Tawil, S. 2005. Evaluation of FEMA 350 seismic provisions for steel panel zones. *Journal of Structural Engineering, ASCE*, **131(2):** 250-258.

Krawinkler, H., Bereto,V.V., and Popov, E.P. 1971. Inelastic behaviour of steel beam to columns subassemblagement. *EERC Report No. 71-7*, University of California, Berkeley.

Lee, CH., Jeon, SW., Kim, JH., Uang, CH. 2005. Effects of panel zone strength and beam web connection method on seismic performance of reduced beam section steel moment connections. Journal of Structural Engineering, ASCE; 131(12):1854–1865.

Mao, C., Ricles, J.M., Lu, L.W., and Fisher, J.W. 2001. Effect of local details on ductility of welded moment connections. *Journal of Structural Engineering, ASCE*; 127(9): 1036–1044.

Naka, T., Kato, B., Watabe, M., Nakato, M.1969.Research on the behaviour of steel beam-to-column connections.4th world conference of earthquake engineering, Santiago, Chile.

Popov, E.P. 1987. Panel zone flexibility in seismic moment joints. *Journal of Constructional Steel Research*; 8: 91-118.

Popov, E.P., Amin, N.R., Louis, J.J.C., Stephen, R.M.1985. Cycle behavior of large beam column assemblies, *Earthquake Spectra*; 1(2):203-238.

Ricles, J.M., Fisher, J.W., Lu, L.W., and Kaufmann, E.J. 2002. Development of improved welded moment connections for earthquake-resistant design. *Journal of Constructional Steel Research* **58**: 565–604.

Ricles, J.M., Mao, C., Lu, L.W., and Fisher, J.W. 2003. Ductile details for welded unreinforced moment connections subject to inelastic cyclic loading. *Engineering Structures*, 25: 667–680.

Slutter, R.1981.Tests of panel Zone behavior in beam column connections. *Report No. 200.81.403.1*, Lehigh University, Bethlehem, PA.

Structural Engineering Association of California (SEAOC). 1975. Recommended lateral force requirements and commentary. Seismology Committee, Sacramento, California.

Structural Engineering Association of California (SEAOC). 1987. Recommended lateral force requirements and commentary. Seismology Committee, Sacramento, California.

Thomason, P.F. 1990. Ductile fracture of metals. Pergamon, New York.

Tsai, K.C., and Popov E.P. 1988. Steel beam-column joints in seismic moment resisting frames. *UCB/EERC Report No. 89/19*, University of California, Berkeley.