Failure Modes and Flexural Ductility of Steel Moment Connections

K. C. Lin[1], K. C. Tsai[2] and H. Y. Chang[3]

[1] *Associate Research Fellow,Center for Research on Earthquake Engineering (NCREE), Chinese Taiwan*
[2] *Professor, Dept. of Civil Engineering, Taiwan University, Taipei, Chinese Taiwan*
[3] *Assistant Professor, Dept. of Civil and Environmental Engineering, University of Kaohsiung, Kaohsiung, Chinese Taiwan*
Email: hychang@nuk.edu.tw

ABSTRACT :
Ductility is the main attribute of steel moment connections and the failure modes control it. This study reanalyzes the results of 66 past experiments in Taiwan, comparing connection ductility across failure modes. In reference to FEMA 1995, the failure mode and root cause identification was firstly made for each specimen. 56 of the specimens were found to fail for the main three modes. In details, 43% of the connections failed for flange-HAZ (Heat Affected Zone) fracture, 27% failed for flange-weld fracture, and 16% failed for flange buckling. Following that, the connection ductility of the 56 specimens was reanalyzed and compared in detail. The plastic rotation, at which a connection reaches its maximum moment strength, was used as the main measure of connection ductility. Flange-HAZ fracture gave an average of 1.98%, and flange-weld fracture and flange buckling changed it to 2.47% and 2.95%, respectively. Furthermore, equivalent monotonic ductility was used as another measure, assessing the connection ductility without the effect of loading history. Flange-HAZ fracture gave an average of 11.32, and flange-weld fracture and flange buckling changed it to 11.56 and 11.81, respectively. Finally, a numerical example was given to illustrate that the obtained results have important implications for making a strategy for the performance assessment and earthquake retrofitting of steel moment frames.

KEYWORDS:
steel moment frames, connection ductility, failure modes, performance assessment, earthquake retrofitting

1. INTRODUCTION

To resist strong earthquake shaking, steel moment connections are required to have strength and ductility. In the 1994 Northridge and 1995 Kobe earthquakes, however, many steel moment connections failed in the mode of brittle fractures (EERI 1994; Tremblay et al 1996; AIJ 1997; Nakashima 2002). That means that current engineering practice still cannot ensure steel moment connections to have enough plastic rotational capacity. The ductility capacity of steel moment connections therefore has raised serious concern. In the 1999 Chi-Chi earthquake, no steel moment connections were found to fail. But before the earthquake, the laboratory tests in Taiwan have shown that over 20% of the connections may fail in the mode of brittle fractures (Chen et al 1989). This study therefore assessed the possible causes and effects by reviewing historical performance of steel moment connections in Taiwan. In the assessment, the ratio of ultimate strength to yield strength (strength ratio) was used as a measure to prequalify the results of the testing. Following that, the ultimate plastic rotation, at which a connection reaches its ultimate strength, was compared for failure modes and connection details. Initial cracks usually appear when connections reach the ultimate strength. The retained stiffness is not negligible, because many connections continue to support gravity loads, even at very large deformations. Considering that, the maximum plastic rotation, which a connection specimen can sustain in the test, was used as another measure for connection ductility. Furthermore, a recently reported similitude law (Kuwamura and Takagi 2004) was applied to yield a measure of deformation capacity without the effects of loading history (i.e. the equivalent monotonic ductility). The obtained results can be considered to have important implications for making a strategy for the performance assessment and earthquake retrofitting of steel moment frames. To illustrate that, a numerical example (Lin et al 2008) was given to end the paper.

2. FAILURE MODE AND ROOT CAUSE IDENTIFICATION

In 1987, the first full-scale beam-to-column connection testing in Taiwan was made for the new construction of the 50-story HASEGAWA building in the city of Kaohsiung. Following that, a substantial amount of experimental work has been done to investigate and enhance the seismic performance of steel welded beam-to-column connections (Chen et al 1989; Tsai and Wu 1993). That allowed establishing an experimental database herein. The analyzed 66 sets of connection testing were made in Taiwan University (NTU) and Taiwan University of Science and Technology (NTUST) during the period from 1990 through 1995, in which connection behavior was intensively studied.

The tests have many features in common and can be summarized as follows.

- The tests were conducted on specimens fabricated as cantilevers attached to column stubs, and almost all the specimens were tested by imposing cyclically increasing displacements at the cantilever end.

- The beam-to-column connection assemblages were composed of H-shaped beams (of A36 steel) and built-up box columns (of A572 Grade 50). In some cases, the bolted-web-welded-flange (BWWF) connection details were used with supplementary web bolts or welds.

- The tests examined 5 types of specimens, as shown in Figure 1. Type 1 connections use conventional BWWF details. Type 2 and Type 3 connections respectively increase the seismic capacity by adding cover plates and wing plates to the beam flanges. Type 4 and Type 5 connections reduce the seismic demands by perforating the beam flanges and changing the beam sections, respectively.

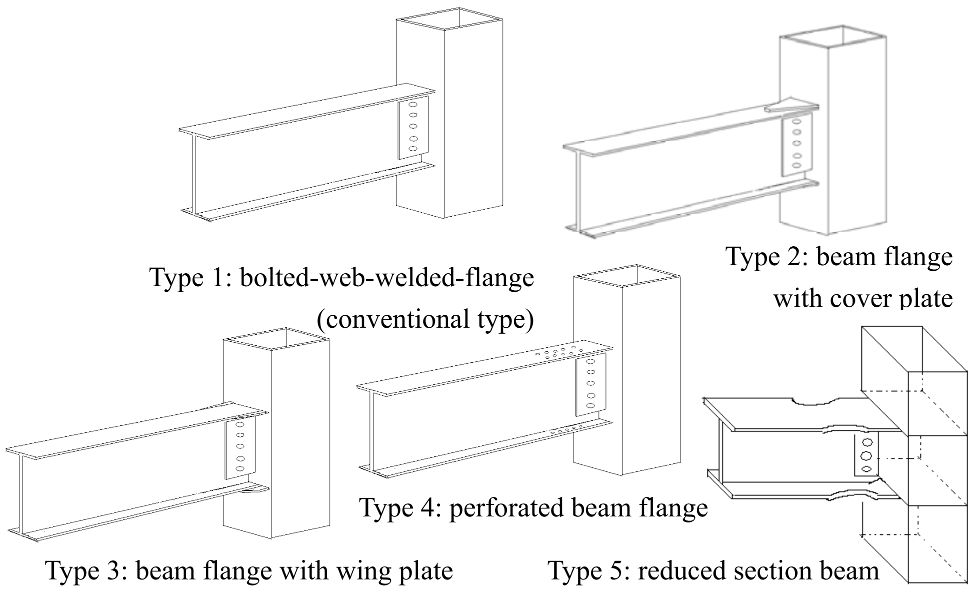


Figure 1 Connection types

In reference to FEMA 1995, the failure mode and root cause identification was made, as illustrated in Figure 2. Well over 66 specimens were identified, and 56 were found to fail for the main three modes. In detail, 43% of the specimens failed for flange-HAZ (i.e. Heat Affected Zone) (G3), 16% failed for flange buckling (G1), and 27% failed for the flange-weld fracture (W4).

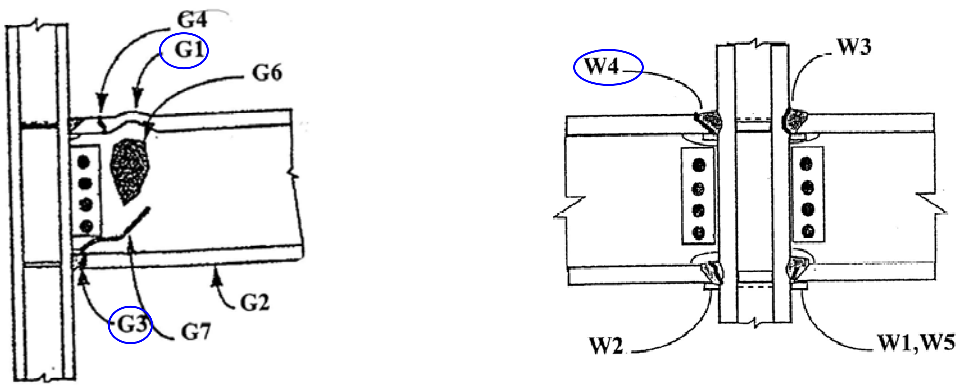
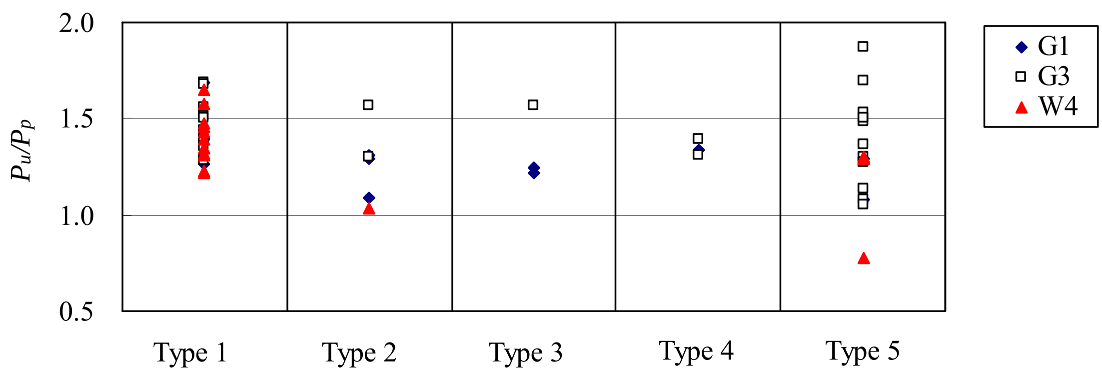


Figure 2 Main three failure modes; G1: flange buckling, G3: flange-HAZ fracture, and W4: flange-weld fracture

3. STRENGTH RATIOS AND CONNECTION DUCTILITY

Steel moment connections are required to have enough strength and ductility to resist earthquake shaking. Considering that, the ratio of ultimate strength P_u to yield strength P_p (i.e. strength ratio) has been used as a measure to prequalify the results of connection testing. As Figure 3(a) shows, most of the connections have a strength ratio P_u/P_p more than 1.0. In addition, RBS connections (Type 5 connections) have had a greater average and variation in P_u/P_p. On the other hand, conventional BWWF connections (Type 1 connections) are vulnerable to flange-weld fracture (W4). In contrast, RBS connections are more susceptible to flange-HAZ fracture (G3).

(a)



(b)

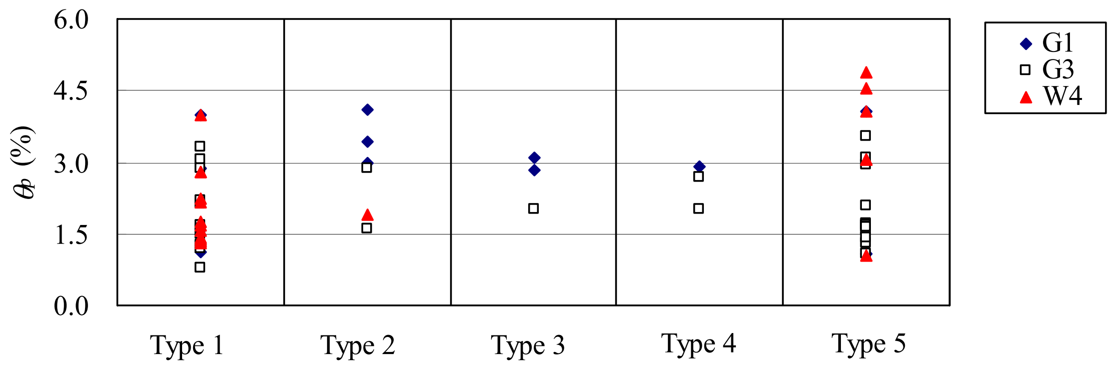
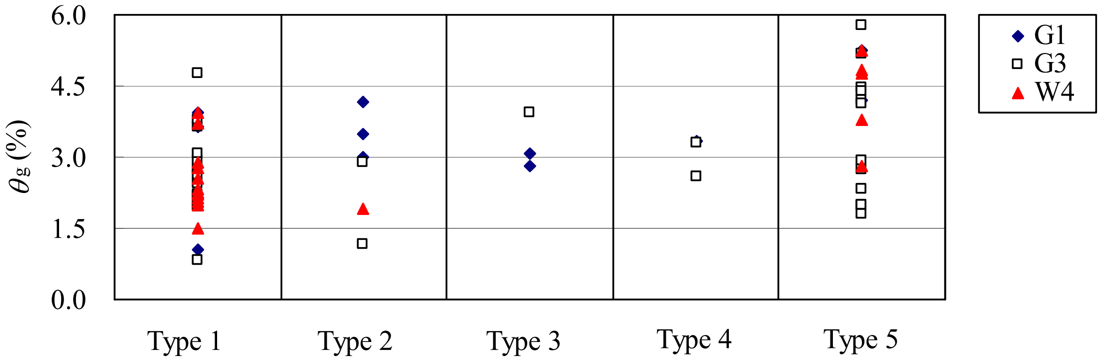


Figure 3 Experimental results; (a) strength ratio P_u/P_p; (b) ultimate plastic rotation θ_p

Initial cracks usually appear when connections reach the ultimate strength P_u. Since that, the plastic rotation θ_p, at which a connection reaches the ultimate strength P_u, has been accepted as a measure of connection rotational capacity. As Figure 3(b) shows, the ultimate plastic rotations θ_p distributes over a range of 1.5%-4.5%, having an average less than 3.0%. Compared to the other types of connections, RBS connections (TYPE 5 connections) have developed a little larger θ_p in the tests. On the other hand, flange-HAZ fracture (G3) limits the average to 1.98%, and flange-weld fracture (W4) and flange buckling (G1) change the average to 2.47% and 2.95%, respectively.

There is a dramatic loss in connection resistance and stiffness after initial cracks appear. In general, the retained stiffness is not negligible, because many connections continue to support gravity loads, even at very large deformations. Considering that, the maximum plastic rotation θ_g, which a connection specimen can sustain in the test, has been used as a measure for the collapse prevention limit state (e.g. Rodger 2003). As Figure 3(b) illustrates, the connection specimens give have an average θ_g near 3%. In addition, RBS connections (Type 5 connections) have sustained larger plastic deformations in the tests.

(a)



(b)

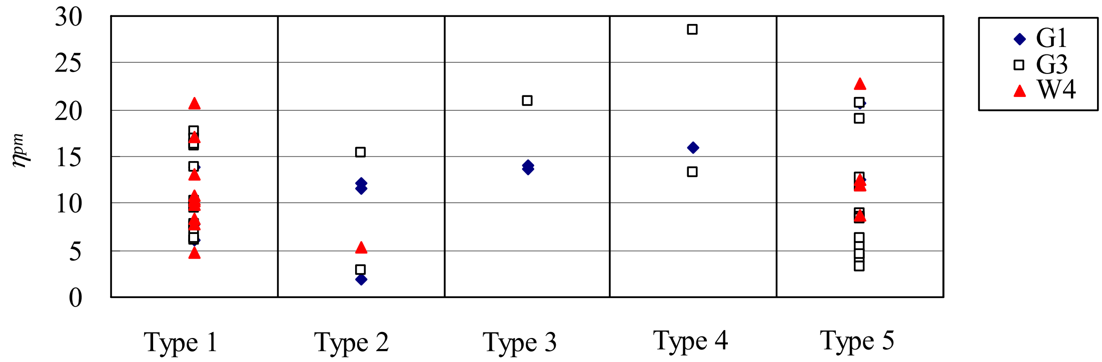


Figure 4 Ductility measures; (a): maximum plastic rotation θ_g, and (b): equivalent monotonic ductility η_{pm}

A recently reported work advocates a similitude law of prefracture hysteresis for steel members, establishing an invariant rule of the ductility amplitude, cumulative ductility, and number of cycles prior to fracture (Kuwamura and Takagi 2004). As shown by Figure 4(b), the similitude law has been applied to yield a measure of deformation capacity without the effects of loading history (i.e. the equivalent monotonic ductility η_{pm}). The invariant rule was originally developed for fractured steel members, and the applications to buckling steel members need further verification. Thus, the equivalent monotonic ductility η_{pm} for flange buckling (G1) in Figure 4 (b) was provided for reference only. Compared to the ultimate plastic rotations θ_p in Figure 3(b), the equivalent monotonic ductility η_{pm} in Figure 4(b) has provided a different view of the connection rotation capacity.

As can be seen in Figure 4(b), conventional BWWF connections (TYPE 1 connections) have η_{pm} quite uniformly distribute over the range of 5-20. In contrast, RBS connections (TYPE 5 connections) have large dissipation in η_{pm}. Some of the RBS connections have η_{pm} much less than 5, and some have η_{pm} more than 20. From this viewpoint, conventional BWWF connections are more reliable, when compared to RBS connections. The limited data also indicates that the other types of connections (i.e. Type2, Type 3 and Type 4 connections) can develop η_{pm} more than 15, having excellent rotation capacity against strong earthquakes.

η_{pm} was further compared across different failure modes. In detail, flange-HAZ fracture (G3) gives η_{pm} an average of 11.32, and flange buckling (G1) and flange-weld fracture (W4) change the average to 11.81 and 11.56, respectively. As mentioned before, flange-weld fracture (W4) and flange-HAZ fracture (G3) limit the average of θ_p to 1.98%, and in contrast, flange buckling (G1) increases the average to 2.95%. The equivalent monotonic ductility η_{pm} in Figure 4(b) has varied with the failure modes to a smaller extent, in comparison with the ultimate plastic rotations θ_p in Figure 3(b).

4. IMPLICATIONS OF OBTAINED RESULTS

The obtained results can be expected to have important implications for making a strategy for the performance assessment and earthquake retrofitting of steel moment frames. To illustrate that, a numerical example (Lin et al 2008) was given below.

Table 1 Connection details and connection plastic rotation

Connection details	Specimen number	Connection plastic rotation (%)	
		Median capacity	Standard deviation
Conventional BWWF	27	2.62	0.90
Improved (new) details	29	3.50	1.24

Table 2 Member sizes and steel grades

(1) 6-story MRF (period for the 1st mode vibration = 1.48 sec)

Story	Column(A572)	Girder_X(A36)	Girder_Z(A36)
6F~5F	Box500×500×19	H488×300×11×18	H386×299×9×14
4F~3F	Box500×500×22	H494×302×13×21	H500×304×15×24
2F~1F	Box500×500×25	H588×300×12×20	H582×300×12×17

(2) 20-story MRF (period for the 1st mode vibration = 3.18 sec)

Story	Column(A572)	Girder_X(A36)	Girder_Z(A36)
20F~19F	Box500×500×19	H588×300×12×20	H582×300×12×17
18F~16F	Box600×600×28	H594×302×14×23	H594×302×14×23
15F~13F	Box700×700×25	H700×300×13×24	H700×300×13×24
12F~10F	Box700×700×28	H708×302×15×28	H708×302×15×28
9F~7F	Box750×750×25	H708×302×15×28	H712×306×19×30
6F~4F	Box750×750×28	H800×300×14×26	H800×300×14×26
3F~1F	Box750×750×32	H800×300×14×26	H800×300×14×26

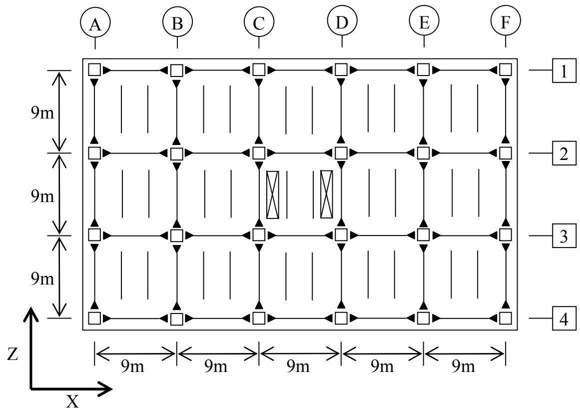


Figure 5 Floor plan of example buildings

In this example, the ultimate plastic rotation has been used as the main measure of connection ductility. The plastic rotational capacity of the connections and the variation were evaluated by experiment database, as shown in Table 1. The analyzed 6- and 20-story steel moment frames (SMRFs) have the same floor plan, as illustrated by Figure 5. The member sizes and steel grades are summarized in Table 2. To evaluate the demands, a series of three-dimensional nonlinear time history analyses were carried out for the SMRFs with a suite of 8 earthquake ground motions scaled to increasingly higher intensity. To consider the capacity uncertainty, the capacity was assumed to have a normal distribution, and the Latin sampling was applied.

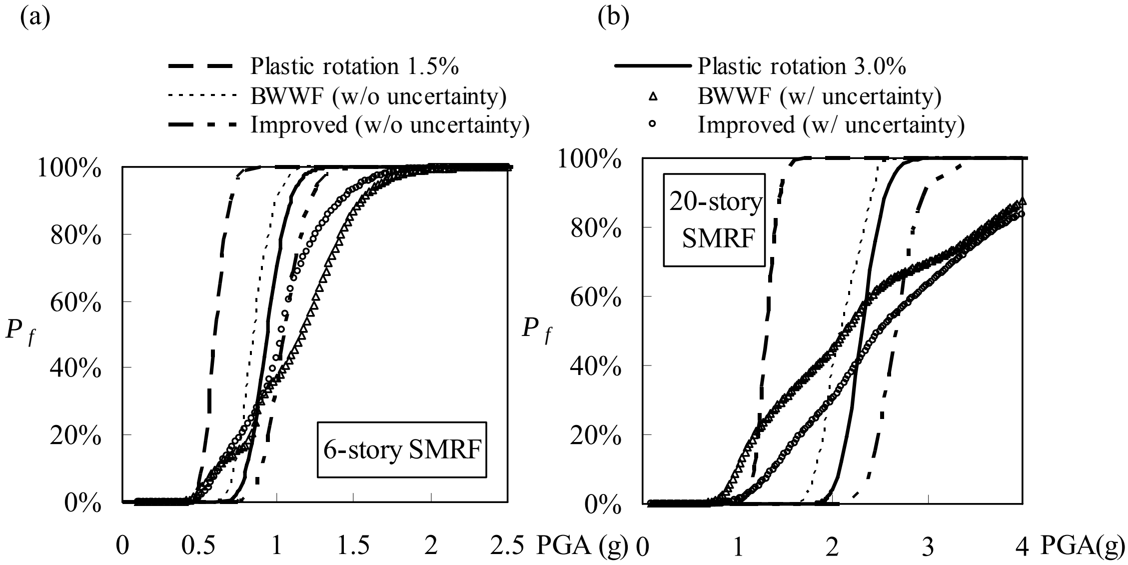


Figure 6 Effect of connection details in seismic reliability of 6- and 20-story SMRFs (Lin et al 2008)

Figures 6(a) and (b) compare the fragility curves for the 6- and 20-story SMRFs with connections using conventional BWWF and improved details. In the same figures, for reference, the fragility curves for the pre- and post-earthquake requirements were also given. As can be seen, the fragility curves shift to the right as the connection ductility change from 1.5% to 3.0%. In addition, the 20-story SMRF has a considerably larger change. That has suggested that high-rise buildings may benefit more by the increased connection ductility. Table 4 showed that the usage of improved details can increase the connection ductility.

Table 1 has showed that the usage of improved details can increase the connection ductility. As Figure 6 (a) and (b) illustrate, the fragility curves for BWWF details stay at the left side of those for improved details. That means that improved details can reduce the failure probability of SMRFs. Table 1 also indicated that the diversity of improved details can cause a large variation in the connection ductility. Figures 6 (a) and (b) further suggest that the capacity variation can differ and shift the fragility curves to the left. For the 20-story SMRF with connections using improved details, for example, the fragility curve without capacity uncertainty starts to rise at a PGA more than 2g. In contrast, the fragility curve with capacity uncertain starts to rise at a PGA less than 1g. In other words, the SMRFs can fail at a PGA much smaller than expected. The figures also show that the capacity uncertainty can reduce the difference between BWWF and improved details. For the 6-story SMRF, for example, the capacity uncertainty has caused the fragility curves to lose the difference.

5. CONCLUSION

This paper reanalyzed the results of 66 past experiments in Taiwan, comparing the connection ductility across different failure modes. The failure mode and root cause was made for each connection. 43% of the connections were found to fail for flange-HAZ fracture, 27% for flange-weld fracture, and 16% for flange buckling. Then, reanalysis was made on the 56 sets of connection testing that showed the above three failure modes. The plastic rotation, at which the ultimate strength is reached, was adopted as a measure of connection rotation capacity (i.e. ultimate plastic rotation). Flange-HAZ fracture gave an average of 1.98%, and flange flange-weld fracture and buckling respectively changed it to 2.47% and 2.95%.

The equivalent monotonic ductility was used as another measure in this study. That allowed further comparing the connection ductility without the effects of loading history. Flange-HAZ fracture gave an average of 11.32, and flange-weld fracture and flange buckling changed the average to 11.56 and 11.81, respectively. Compared to the ultimate plastic rotations, the equivalent monotonic ductility has varied with the failure modes to a smaller extent. The equivalent monotonic ductility was also compared for different connection details. RBS connections were found to have larger variation in the connection ductility and were therefore considered be less reliable. The equivalent monotonic ductility has provided a different view of the connection rotation capacity.

The obtained results can be expected to have important implications for the performance assessment and earthquake retrofitting of steel moment frames. To illustrate that, a numerical example was also given. A reliability assessment was made for a 6- and 20-story SMRFs with connections using conventional BWWF and improved details. The experimental database indicated that improved details can increase connection ductility, but the diversity of the details, e.g. added cover plates and reduced beam sections, can also lead to a large variation in the connection ductility. The reliability assessment helped confirm that the improved details can improve the seismic performance of SMRFs. It also showed that the capacity uncertainty may greatly reduce the difference between the improved and conventional BWWF details.

REFERENCES

Earthquake Engineering Research Institute (EERI). (1994). Northridge Earthquake, January 17, 1994: Preliminary reconnaissance report, Oakland.

Architectural Institute of Japan (AIJ). (1997). Report on the Hanshin-Awaji Earthquake Disaster Building Series Volume 3 Structural Damage to Steel Buildings Structural Damage to Shell and Spatial Structures Structural Damage to Storage Tanks and Their Supports, Tokyo (in Japanese).

Tremblay, R., Bruneau, M., Nakashima, M., Prion, H.G.L., Filiatrault, A., and DeVall, R. (1996). Seismic design of steel buildings: lessons from the 1995 Hyogo-Ken Nanbu earthquake, Canadian Journal of Civil Engineering, 23:3, 727-756.

Nakashima, M., Roeder, C. W., and Maruoka, Y. (2000). Steel moment frames for earthquakes in United States and Japan, Journal of Structural Engineering, ASCE, 126:8, 861-868.

Chen, S. J., Lin, H.Y., Lin, and Yang, R.K. (1989). Research on Steel Beam-to-Column Connection. Structural Engineering, 4:3, 39-48 (in Chinese)

Tsai, K. C. and Wu. S. (1993). Behavior and Design of Seismic Moment Resisting Beam-Column Joints, Report No. R82-10, Center for Earthquake Engineering Research, Taiwan University, Taipei, Chinese Taiwan.

Roeder, C. W. (2003). Prequalification of Steel Moment-Frame Connection Performance. Earthquake Spectra, 19:2, 291-308

Kuwamura, H and Member, Takagi, H. (2004). Similitude Law of Prefracture Hysteresis of Steel Members, Journal of Structural Engineering, ASCE, 130:5, 752-761.

Lin, C. C., Lin, K. C., Tsai, K. C., YEH, S. Y., LIN, C. H. and Chang, H. Y. (2008). Effect of connection ductility in the seismic performance of steel moment frames, *Proceedings of 11th East Asia-Pacific Conference on Structural Engineering & Construction* (EASEC-11), Taipei.