Accumulated Seismic Connection Damage Based upon Full Scale Low Cycle Fatigue Connection Tests

Presented at the 2001 SEAOC Convention

Ralph M. Richard, Ph.D., P.E., Professor Emeritus
The University of Arizona, Tucson, AZ

Jay Allen, S.E., President
Allen Consulting Engineers, Inc., Mission Viejo, CA

James E. Partridge, P.E., President
Smith Emery Company, Los Angeles, CA

Abstract

Results of cyclic cantilever beam-to-column connection tests are used herein to evaluate both the fracture modes and the accumulated seismic connection damage of the pre-Northridge (pre-N), Dogbone (RBS) and Slotted Web (SW) connections. These tests were made using the ATC-24 test protocol with the exception that the tip displacements of the beam-column assemblies were cycled at constant displacements from the initiation of the test until the connections fractured. Results of these *strain-controlled* tests include a graph of "Interstory Drift vs. Cycles to Connection Fracture". Data from this graph are then used to demonstrate how accumulated seismic connection damage may be evaluated, and additionally, to compare the performance of the three connections using a steel moment frame building's composite seismic interstory drift history.

Introduction

Research in the modes and causes of fracture of unreinforced welded moment frame connections under simulated seismic loading was initiated by Popov in the late 1960s (Popov and Stevens, 1970). However, it was not until the research of Kuwamura in the early 1990s (Kuwamura and Suzuki, 1992) that these research efforts turned to low cycle fatigue as the cause of connection fractures. Subsequent research reported by Kuwamura and Yamamoto (1997), Partridge et al. (2000), and Barsom (2000) has shown that these connection fractures were caused by the initiation of fatigue cracks that propagated to critical size. These brittle fractures comprised three phases: (1) the initiation of a ductile crack, (2) the stable growth of this crack, and (3) the sudden propagation of this crack to connection fracture in a brittle manner. Kuwamura's research specimens included ATC-24 type tests using H beams welded to box columns that were cycled at constant amplitudes, welded T joints, and round notched bars. Fractographs by a scanning electron microscope were used to examine the fractured surfaces. Barsom's research used the results of the ATC-24 protocol tests sponsored by the FEMA/SAC program (Goel, et al., 1999; Fry et al., 2000; and Ricles, et al., 2000), and reported that fatigue cracks initiated and propagated in all the tested connections.

Results of cyclic cantilever beam-to-column connection tests are used herein to evaluate both the fracture modes and the accumulated seismic connection damage of the pre-Northridge (pre-N), Dogbone (RBS) and Slotted Web (SW) connections (FEMA-350, 2000). These tests were made using the ATC-24 test protocol with the exception that the tip displacements of the beam-column assemblies were cycled at constant displacements from the initiation of the test until the connections fractured. Results of these *strain-controlled* tests include a graph of "Interstory Drift vs. Cycles to Connection Fracture". Data from this graph are then used to demonstrate how accumulated seismic connection damage may be evaluated, and additionally, to compare the performance of these three connections based upon a steel moment frame building's composite seismic interstory drift history.

Kuwamura's ATC-24 Constant Amplitude Tests

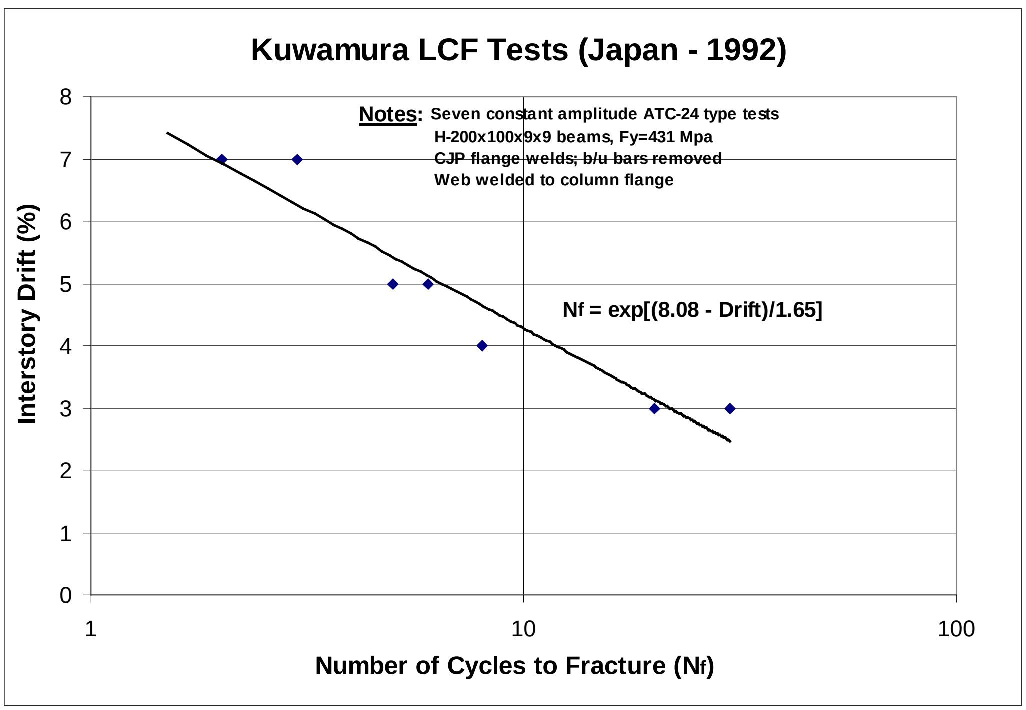
Kuwamura's (1992) low cycle fatigue tests comprised seven all welded beam-to-column connection assemblies using fabricated H-200x100x9x9mm beams. The flange welds were CJP whereas the beam web was fillet welded to the column flange. Both backup bars were removed and a reinforcing fillet weld was applied. Constant amplitude beam tip displacements of $\pm 3\delta y$, $\pm 4\delta y$, $\pm 5\delta y$, and $\pm 7\delta y$ were applied until the specimens fractured. All specimens collapsed due to fatigue fracture in same manner. Relations between tip deflection and total energy absorption before fracture were determined, which were then used to predict the seismic life of the connection.

Using the data from Kuwamura's seven tests, a plot of Interstory Drift (%) vs. Number of Cycles to Connection Fracture (N_f) was generated as shown in the figure below. From this plot it is apparent that an exponential equation accurately predicts the relationship between interstory drift and number of cycles to connection fracture for these tests, which provides an alternative method for the prediction of the seismic life of connections as shown in the following sections of this paper.

Description of the Partridge Beam-Column Assemblies and Connection Designs

All ten ATC-24 test specimens comprised Grade 50 W18x40 beams ($Z_{flg}/Z = 0.70$) and Grade 50 W14x135 columns with continuity plates. All of the complete joint penetration (CJP) beam flange welds were ground to 1/4" radius at the column face. The weld filler material was E71-T8 for all specimens (Partridge, et al., 2000).

The beam webs of Type I (SW) and Type II (RBS) connections were welded to the column flange using CJP welds extending the full depth of the shear plates. The 5/16" shear plates, which matched the thickness of the beam webs, used two 3/4" A325X erection bolts and were fully fillet welded to the beam webs. Additionally, both top and bottom back-up bars were removed, and the bottoms of the beam flanges were reinforced with 1/4" fillet welds. The RBS beams had a radius cut flange reduction of 40% and were designed in full accordance with the FEMA-350 RBS connection design rationale (FEMA-350, 2000) except for the 5/16" "heavy" shear plate that was not only full pen welded to the column flange but was also fully fillet welded to the beam web. Thus the beam flange and web attachments to the column flange were identical for the RBS and Slotted Web.



The Type III (pre-N) connection had both back-up bars removed with beam flanges reinforced with 1/4" fillet welds identical to the SW and RBS connections. The 5/16" fillet welded shear plates matched the beam web thickness, had four 3/4" A325X fully tensioned bolts in standard holes, and were reinforced with supplemental welds to the beam web (UBC, 1991).

All of the CJP welds on these test specimens were tested ultrasonically acceptable. These test specimens represented the _best_ possible designs in terms of weld material and their connection configurations.

Testing Protocol and Accumulated Damage Assessment Rationale

All ATC-24 test assemblies were slowly cycled (typically of the order of one minute per cycle) at the specified values of constant beam tip displacements, i.e., constant story drifts, until the connections fractured. At the conclusion of this strain controlled test the fractured surfaces were cut from the beam-column assemblies for metallurgical evaluation. In every case it was found that sub-critical cracking, by a low cycle fatigue mechanism, preceded unstable fracture initiation and failure (Partridge, et al, 2000), which was consistent with the fatigue crack initiation and propagation reported by Kuwamura and Yamamoto (1997) and Barsom (2000). Typically, more than 50% of the fracture surface (or cross section of the beam flange) exhibited "beach markings" indicative of the low cycle fatigue mode of failure. Two of the RBS connections fractured in the flanges at the reduced section at 1.9% and 2.3% drift, due to lateral torsional twisting, and one near the beam flange weld at 1.5% drift. The pre-N connections all fractured in the beam flanges near the flange welds. All of the SW connections fractured in the beam flange over the slot.

The number of cycles to connection fracture vs. story drift curves for all three connection types are given in Figure 1. This constant amplitude testing procedure is equivalent to the ASTM Standards E-739 and E-606 "S-N" test protocols that are used to establish the number of cycles to failure (N) at given levels of stress (S) or strain, and to establish the endurance limit (fatigue life) of materials and/or mechanical components subject to cyclic loading. In these tests the beam tip displacement, which is proportional to story drift, was used to generate "Interstory Drift vs. Cycles to Connection Fracture" curves shown in Figure 1. These curves may then be used to evaluate the expected life of buildings that have a history of earthquake or wind induced story drift. Because the earthquake induced story drifts vary significantly in amplitude, the fatigue life evaluations of the connections in a given building are made using these curves and the Palmgren-Miner Rule for variable amplitude loading (Dowling, 1993). Research by Krawinkler and Zohrei (1983) also showed that this rule may be used to predict the number of cycles to failure in steel structures under variable amplitude seismic loading. Additional information on cycle counting can also be found in ASTM Standard No. E1049 and in the *SAE Fatigue Design Handbook*, 2nd Edition. It is noted here that the Palmgren-Miner Rule, which computationally converts a random process to a deterministic one, is widely used in the mechanical and aerospace industry to predict the fatigue life of welded joints, and engine, machine, and airframe components. Moreover, it is the fatigue life algorithm used in the American Institute of Steel Construction Specifications.

Quantification of Accumulated Seismic Damage

To demonstrate how these data may be used to evaluate the level of beam-to-column connection performance and/or accumulated seismic damage in a given building, consider the illustrative example given in Table 1. The composite seismic interstory drift history used in this demonstrative example is a numerically generated drift spectrum for a steel moment frame building that could result from a single seismic event such as the January 17, 1994, Northridge, California earthquake (M 6.7) and the after shocks that followed this event. In this spectrum the maximum interstory drift is 2.5% for three cycles. It is noted here that the onset of inelastic connection behavior, as evaluated by their plastic rotation curves obtained from ATC 24 protocol tests, is approximately the interstory drift percent minus one percent. Therefore, the maximum plastic connection rotation for this seismic spectrum is only 1.5%, which is equal to that chosen by Tsai and Popov (1988) as a reasonable maximum for steel moment frame connections subjected to severe seismic loading.

The first column of this table is the seismic drift spectrum, which gives the number of cycles, n, at the levels of interstory drift that the given building has been subjected. This spectrum, which could be obtained either from the instrumented building or through numerical simulation (Tsai and Popov, 1988), may be for a single seismic event or represent a composite history of these events over the life of the building. The next three columns are the cycles to failure, N_f, obtained from the test results for each of the three connections. The next three columns are the Palmgren-Miner quotients, n/N_f, for each connection that represents the accumulated damage for each of these levels of cyclic loading. Quotients that are greater than unity indicate connection fracture whereas quotients less than unity represent the accumulated damage or the "used up" life of the connection. As shown here, the pre-N (pre-Northridge) connection fractures at all of these levels of interstory drift, which is consistent with the connection fractures estimated to have occurred in numerous steel moment frame buildings in the proximity of the Northridge epicenter. The RBS (Dogbone) connection survives each level of these events but has accumulated 84% of its seismic life. In contrast to these two connections, the SW (Slotted Web) has accumulated only 28% of its seismic life.

Commentary

Both finite element analyses and strain measurements have shown that the strain demands at the weld access holes and beam flange/welds of the pre-N and RBS connections are very high (Richard, 1995, 1997, 1998; Allen, 1995; Fry, 2000; Ricles, 2000). This is reflected in the results of these strain-controlled full scale connection tests. Elastic stress and strain concentration factors at flange/welds and weld access holes typically range from 4.0 to 5.0 due to the prying action of the large vertical shear in the flanges in both the pre-N and RBS connections, which is typically 50% of the seismic beam shear (Allen, 1996; Richard, 1998; Mahin, 2001). Although the redistribution of the stresses in these regions, as a result of material plastification, reduces the stress concentration factors to near unity, the strain concentration factors <u>increase</u> dramatically. For example, according to Nueber's Theorem (Nueber, 1961; Boresi and Sidebottom, 1985), the product of the stress concentration factor, Kstress, and the strain concentration factor, Kstrain, is equal to the square of the elastic stress concentration factor, Kstress, elastic:

$$\text{Kstress x Kstrain} = (\text{Kstress,elastic})^2$$

This equation holds for stresses above and below the elastic limit of the material. The results of both inelastic finite element analyses and tests confirm the validity of this equation, which quantifies the large ductility demands of the pre-N and RBS connections (Richard, 1998). For example, if the flange/weld access hole elastic stress and strain concentration factors equal 4.5, then after connection element (beam flange or web) plastification, the stress concentration factor becomes unity and the strain concentration factor becomes approximately 20. To reduce this ductility demand and to account for the vertical shear in the beam flanges and its prying effects at the flange welds and weld access holes, vertical flange fins have been successfully tested and used on both the pre-N and RBS connections (Zekioglu, 1997; Goel, 1997, 2000).

Even though the stress and strain distributions in the pre-N and RBS are the very similar at the column face, these tests show a better RBS performance. This results from this RBS connection design that uses beam web weldments to both the column flange and a "heavy" shear

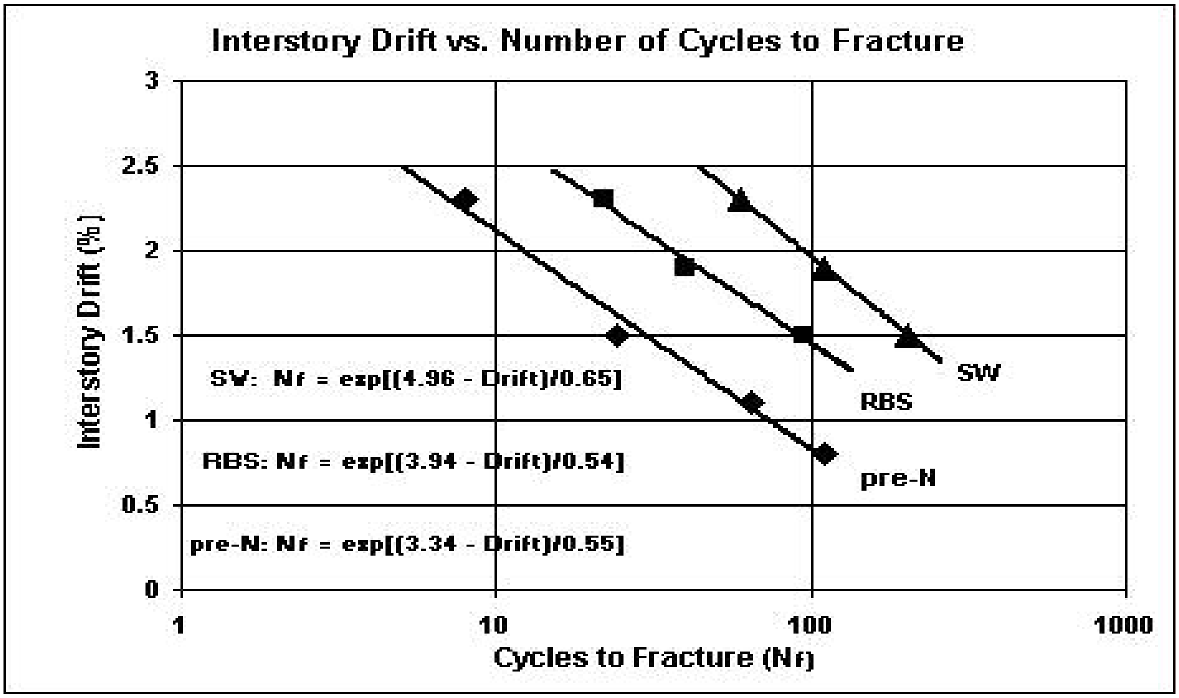


Figure 1. Interstory Drift vs. Cycles to Connection Fracture

Composite Seismic Interstory Drift History From Bldg. Data: No. of Cycles = n	Cycles to Fracture of the Connection: Cycles to Fracture = N_f			Accumulated Damage to the Connection: Damage = n/N_f		
	Pre-N	RBS	SW	Pre-N	RBS	SW
3 cycles @ 2.50% ± 0.25%	3	13	51	1.00	0.23	0.06
10 cycles @ 2.00% ± 0.25%	9	36	98	1.11	0.28	0.10
30 cycles @ 1.50% ± 0.25%	27	92	240	1.11	0.33	0.12
85 cycles @ 1.00% ± 0.25%	74	---	---	1.14	---	---
Connection Accumulated Damage Summary:				Fracture	0.84	0.28

Table 1. Computation of Connection Accumulated Seismic Damage

plate, which reduces the shear in the beam flanges to about 50% of the total connection shear. Additionally, the stiffness of the RBS test assembly is approximately 10% less than the pre-N assembly (FEMA-350, 2000). If a bolted web RBS beam with stiffness equal to the pre-N beam (e.g., a W18x46) had been used in these tests, the number of cycles to fracture for the RBS specimens would have been significantly reduced.

Summary and Conclusions

The four main attributes of the SW connection that give this connection very superior performance over the pre-N and RBS connections are:

1. The SW beam web resists the entire seismic beam shear whereas the welded web RBS connection resists approximately 50% of the seismic beam shear in the beam flanges. This flange shear is not accounted for in the current SAC/AISC design rationale (FEMA-350, 2000) for the RBS, WUF-W, and WUF-B connection designs.

2. The SW connection is kinematic so that the force distributions in the connection are statically determinate and independent of interstory drift. When the pre-N and RBS connections are subject to cyclic loading, their moment and shear distributions between the beam web and beam flanges change depending upon whether the connection is undergoing inelastic loading or elastic unloading.

3. The stress and strain distributions in the beam flanges and beam flange welds of the SW connection are nearly uniform with stress and strain concentration factors of 1.2 to 1.4. Conversely, both the pre-N and RBS connections have identical large stress and strain gradients horizontally across and vertically through the beam flanges and beam flange welds that result in stress and strain concentrations of 4.0 to 5.0.

4. Lateral torsional buckling is eliminated by the SW connection. By separating the beam flange from the beam web, the flanges and web buckle independently and concurrently. This eliminates the torsional shear stresses associated with this twisting mode of buckling that initiates at approximately 1.25% story drift in all unbraced non-slotted connections.

- By performing a series of constant-displacement low-cycle fatigue tests, it is possible to quantify the seismic fatigue life, reliability, and performance of existing and candidate beam-to-column connections in steel moment-frame buildings.

- The pre-N and RBS connection geometry and stiffness distributions result in large stress and strain concentrations. Both have moment and shear distributions that differ drastically from elementary beam theory that is used in their design rationale. Both of these characteristics significantly increase accumulated seismic damage and reduce the fatigue life of these connections.

- Although the pre-N and RBS connection stress concentrations reduce significantly as plastic behavior occurs, the strain concentrations, and therefore the strain rates, increase dramatically. This places severe ductility demands and force re-distributions in all the elements of the connection and reduces the fatigue life of the connection.

References

Allen, J., et al. 1995. "Ductile Connection Designs for Welded Steel Moment Frames", *Proceedings of the 1995 Structural Engineers Association of California Conference*, Indian Wells, CA.

Allen, J. 1996. Seismic Structural Design Engineers, Inc. Seminar Notes on the Design of Slotted Web Beam-to-Column Connections.

Barsom, J. M. 2000. "Development of Fracture Toughness Requirements for Weld Materials in Seismic Applications", SAC Steel Project Task 7.1.3.

Boresi, B.P., and Sidebottom, O.M. 1985. Advanced Mechanics of Materials, 4th Edition, John Wiley & Sons, Inc., pp. 594-598.

Dowling, N.E. 1993. N. E., Mechanical Behavior of Metals, Prentice Hall, Inc., pp. 382-399

FEMA-350, 2000. "Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings", July 2000, pp. 1-4, 3-38 to 3-42, and 3-70 to 3-72.

Fry, G.T., et al. 2000. *Supplemental Analysis and Testing of Reduced Beam Section Components.* SAC Steel Project - Task 7.1.3.

Goel, S.C., Stojadinovic, B.. and Lee, K.H. 1997. "Truss Analogy for Steel Moment Connections", *Engineering Jr.*, AISC, Vol. 34, No. 2, Second Quarter, pp. 43-53.

Goel, S.C., et al. 1999. *Parametric Tests on Unreinforced Connections,* SAC Steel Project - Task 7.023.

Goel, S.C, Lee, K.H., and Stojadinovic, B. 2000. "Design of Welded Steel Moment Frames Using Truss Analogy", *Engineering Jr.*, AISC, Vol. 37, No.1, First Quarter, pp. 31-40.

Krawinkler, H, and Zohrei, M. 1983. "Cumulative Damage in Steel Structures Subjected to Earthquake Ground Motions," *Computers & Structures*, Vol. 18, No. 1-4, pp. 531-541.

Kuwamura, H., and Suzaki, T. 1992. "Low Cycle Fatigue Resistance of Welded Joints of High Strength Steels", *Proceedings of the Tenth World Congress on Earthquake Engineering*, Madrid, Spain, pp. 2851- 2856.

Kuwamuara, H. and Yamamoto, K. 1997. "Ductile Crack as a Trigger of Brittle Fracture in Steel", *Jr. of Structural Engineering*, ASCE, Vol. 123, No. 6, June, 1997.

Mahin, S. 2001. SAC Regional Seminar: New Recommended Seismic Design Criteria for Steel Moment-Frame Buildings, pp. 3-7 and 3-8.

Nueber, H. 1961. "Theory of Stress Concentration Factors for Shear Strained Prismatic Bodies with Non-linear Stress-Strain Law", *Jr. of Applied Mechanics*, Vol. 28, Series E., No. 4, Dec. 1961, pp. 544-550.

Partridge, J. E., Paterson, S., and Richard, R.M. 2000. "Low Cycle Fatigue Tests and Fracture Analyses of Bolted-Welded Seismic Moment Frame Connections", STESSA 2000 Conference on Behavior of Steel Structures in Seismic Areas, 21-24 August 2000, Montreal, Canada. (www.ssda.net)

Popov, E.P., and Stevens, R.M. 1970. "Cyclic Loading of Full-size Steel Connections", Report No. EERC 70-3, College of Engineering, Univ. of California, Berkeley, CA.

Richard, R.M., et al. 1995. "Finite Element Analysis and Tests of Beam-to-Column Connections", *Modern Steel Construction,* AISC, October, 1995, pp. 44-47

Richard, R.M., Allen, J., and Partridge, J.E. 1997. "Proprietary Slotted Beam Connection Designs", *Modern Steel Construction,* AISC, March, 1997, pp. 28-33.

Richard, R.M. and Radau, R.E. 1998. "Force, Stress, and Strain Distributions in FR Bolted-Welded Connections", *Proceedings of the Structural Engineers World Congress*, San Francisco, CA, Technical Paper No. T199.

Ricles, J.M., et al. 2000. "Supplemental Testing of Unreinforced Connections with Integration of T-stub Weld Tests and Analysis", SAC Steel Project - Task 7.05.

Tsai, H.C., and Popov, E.P. 1988. "Steel beam-column joints in seismic moment resisting frames." *Report No. UCB/EERC-88/19*, Earthquake Engineering Research Center, Univ. of California, Berkeley, CA.

Uniform Building Code 1991. International Conference of Building Officials, Whittier, CA.

Zekiogla, S.E, et al., "Designing After Northridge", *Modern Steel Construction*, AISC, March, 1997, pp. 36-42.