STEEL MOMENT FRAME DAMAGE PREDICTIONS USING LOW-CYCLE FATIGUE

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ABSTRACT :

For over 40 years low-cycle fatigue has been known to be a cause of structural failure in steel frames. The utility of these findings was limited due to the inability of existing fatigue damage models to properly account for the complexities of connection behavior. Recent research has reinforced the significance of low cycle fatigue through identification of the state of stress in standard moment connections. In addition, testing has shown that fatigue life is measurable, repeatable, and varies significantly based on connection type.

This paper presents a method for calculating the fatigue damage in steel moment frames. The results of nonlinear analysis are combined with experimentally obtained fatigue-damage curves to predict failure or the remaining useful life after an earthquake. Several example problems are presented where the response parameters of plastic hinge rotation and story drift are compared to FEMA-356 acceptance criteria and used to calculate damage using low cycle fatigue. Fatigue life predictions are presented for pre-Northridge, reduced beam section, and slotted web connections. The results indicate that fatigue life provides a more nuanced prediction of damage than hard deformation/deflection limits, and that the predicted damage correlates well with observed behavior.

KEYWORDS:
fatigue, damage, steel, connections, calculation

1. INTRODUCTION

Predicting structural damage due to seismic events is a crucial step in the design process and in developing new technologies and code requirements. Current code-based designs depend primarily upon defining equivalent static loads and ensuring that the structural elements remain elastic under those prescribed loads. Nonlinear analysis, both static (pushover) and dynamic, is allowed as an alternative technique. Guidelines have been developed for determining the model properties and damage parameters used for developing the nonlinear models and evaluating the results (ASCE/SEI, 2006). The decision about the suitability of a structural element to resist the imposed seismic loads is based on comparing the maximum response value obtained during the analysis to prescribed limits. The limits, although consisting of a single number, indirectly consider the cyclic nature of the loading.

Although most design techniques are based on considering the maximum response to a monotonic load, it has long been known that fatigue is a significant cause of failures in steel structures. Research on fatigue in structures dates back to the early 1900's. Renewed interest in research into fatigue developed in the 1960's and 1970's. Bertero and Popov (1965) performed a series of strain controlled tests on steel beams and found that cracks formed several cycles after flange local buckling was observed, and eventually leading to fracture of the flanges. The relationship between plastic strain amplitude and the number of cycles to failure followed the form of the classical fatigue relationship. Other studies (Kasiraj, 1972; Suidan and Eubanks, 1973) also found that failure in steel structures was well correlated with cumulative damage models.

The lack of practical analytical tools for implementing cumulative damage calculations effectively discouraged further development. As computer analysis methods became more practical, new methodologies for predicting earthquake structural damage were developed. Park et al. (1984) defined a damage index that included terms considering the maximum deformation and the hysteretic energy dissipation. Thus both peak response and cumulative measures are incorporated. The concept of equivalent cycles and damage prediction using low cycle fatigue concepts was discussed by McCabe and Hall (1989). Krawinkler et al. (1983) recommended testing protocols for determining the seismic capacity of structural elements that considered the full loading history in order to account for the cumulative damage expected during actual earthquakes.

More recent testing (Kuwamura and Suzuki, 1992; Kuwamura and Yamamoto, 1997; Partridge et al., 2000) has shown that connection fractures were caused by fatigue cracks that subsequently propagated to critical size. Taucer et al. (2000) tested two-story frames under cyclic loading. Low cycle fatigue failures were found in a majority of the failed connections, but the rotational capacity varied greatly by connection type. A series of connection tests sponsored by the FEMA/SAC program were evaluated to determine the cause of failure by Barsom and Pellegrino (2002). Fatigue cracks were found to have initiated and propagated in all the tested specimens. Stojadinovic (2003) tested a series of pre-qualified post-Northridge connections. It was found that adequate bracing and non-slender members, which prevented local beam buckling and lateral-torsional buckling, led to low cycle fatigue failures. However, properly designed connections were found to have adequate ductility to sustain typical earthquake loads before the onset of the eventual fatigue failure.

Low cycle fatigue failures were found in a preponderance connection types across multiple testing programs. This suggests that a method for predicting connection capacity based on fatigue theory will allow engineers to evaluate the ultimate cause of failure. These calculations would be complementary to current design methods, which allow for designs that are able to withstand the imposed loads, but do not provide information about the ultimate failure mechanism. This paper outlines a methodology for calculating the fatigue life of steel frame connections that can be used to determine both the suitability for a single load and the estimated remaining life of the connection.

2. METHODOLOGY

2.1. Calculation Overview

Calculation of the fatigue damage to a steel frame structure consists of four basic steps:

1. Calculate the nonlinear time-history response of the structure to an earthquake loading.
2. Extract the response quantities of interest (ex. plastic beam rotations, story drift, etc.).
3. Convert the time history response to an equivalent series of loading cycles.
4. Calculate the fatigue damage of the equivalent cyclic responses.

The first two steps follow well-established structural engineering procedures. Steps 3 and 4 are less familiar, but standardized computational techniques exist and are detailed in the remainder of this section.

2.2. Damage Calculation

Calculation of damage due to repeated cyclic loads is a well established methodology in some fields of engineering. In particular, mechanical engineers regularly use fatigue damage calculations as part of the design process. With mechanical equipment the cycle amplitudes are generally constant and known and the fatigue limit is directly determined from experiments. However, seismic loads are not made up of complete, consistent cycles. In this case, the Palmgren-Miner rule (Miner, 1945) is used to predict the damage per cycle as

$$D_i = \frac{1}{N_{fi}} \tag{2.1}$$

where D_i is the damage for cycles of magnitude i and N_{fi} is the number of cycles to failure at level i. The total damage to a member over the complete cycling history is then estimated as

$$FDI = \sum_{i=1}^{n} \frac{N_i}{N_{fi}} \tag{2.2}$$

where FDI is the fatigue damage index, or total damage to the element due to the cyclic load, n is the number of different cycle amplitudes in the loading history, and N_i is the number of cycles at amplitude i. Values of FDI greater than or equal to 1.0 indicate a low-cycle fatigue fracture of the member.

Damage predictions made in this fashion give not only a pass/fail indication, but also provide an approximate measure of the remaining fatigue life in the element. Assuming no recovery between loading events, the remaining fraction of the fatigue capacity after an event is 1-*FDI*. Thus, if the *FDI* is equal to 0.60, approximately 40% of the element fatigue life remains available for future events. The FDI calculation procedure is illustrated in Table 2.1 using the connection test data for three different moment connections (Richard, et al., 2001). These data are given in the three columns "Cycles to Failure at Constant Amplitude" for the pre-Northridge (pre-N), the Reduced Beam Section (RBS), and the Slotted Web (SW) connections.

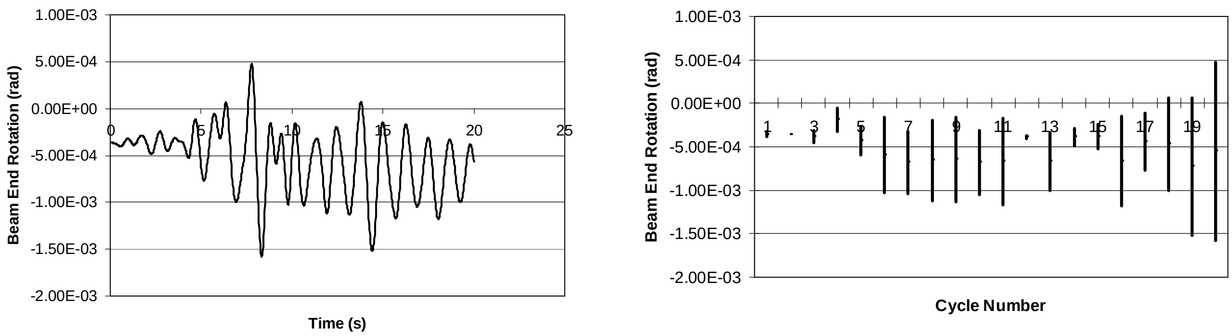
Table 2.1. Sample of fatigue damage index calculation for cycling based on typical measured loads.

Interstory Drift History Number of Cycles at Amplitude = N_i	Cycles to Failure at Constant Amplitude = N_{fi}			Damage at Constant Amplitude = N_i/N_{fi}		
	Pre-N	RBS	SW	Pre-N	RBS	SW
3 cycles @ 2.50%	3	13	51	1.00	0.23	0.06
10 cycles @ 2.00%	9	36	98	1.11	0.28	0.10
30 cycles @ 1.50%	27	92	240	1.11	0.33	0.12
85 cycles @ 1.00%	74	-	-	1.14	-	-
Fatigue Damage Index, *FDI* (Accumulated Damage)				Fracture	0.84	0.28
Remaining Fatigue Life				-	16%	72%

2.3. Cycle Determination

Unlike mechanical vibrations, seismic loads do not impose constant amplitude, or even complete, cycles on a structure. This makes the concept of "cycles" problematic and complicates application of the Palmgren-Miner rule. As seen in Figure 1a, typical seismic response time histories exhibit varying amplitudes, mostly partial cycles, and no complete symmetric cycles. In order to use the measured seismic response to calculate fatigue damage, it is necessary to convert the time history to a series of varying amplitude cycles. The rainflow method is most commonly used for converting random measurements to cycles and has been standardized (ASTM E-1049, 2005). It has also been previously used for determining loading patterns for developing testing procedures to evaluate response to seismic loads (Krawinkler et al., 1983).

The rainflow procedure operates by defining half-cycles where significant changes in response direction occur. These half-cycles are then matched to produce full cycles. The output is a series of "cycles" with calculated mean values, not necessarily zero, and amplitudes. The results of a rainflow analysis on a typical response time history are shown in Figure 1b. Note that although the overall cyclic response is captured, the time sequencing of events is lost in the rainflow procedure. Therefore, if a fatigue failure is predicted there is no way to determine where in the time history the failure occurred.



a. Beam end rotation time history. b. Cycle output from rainflow analysis.

Figure 1. Seismic response time history and resulting cycle conversion.

2.4. Member Capacity

The calculation of earthquake response and converting the time history data to cycles follows well established principles. Likewise, using cyclic data to estimate the FDI is also common practice in many engineering disciplines. The main unknown in the analysis method is thus the fatigue capacity for various member sizes and connection types. Existing test data suggests that the fatigue life is predictable, reproducible, and varies by member size and connection type. Sample test data from one series of tests on three different beam connection types is shown in Figure 2.

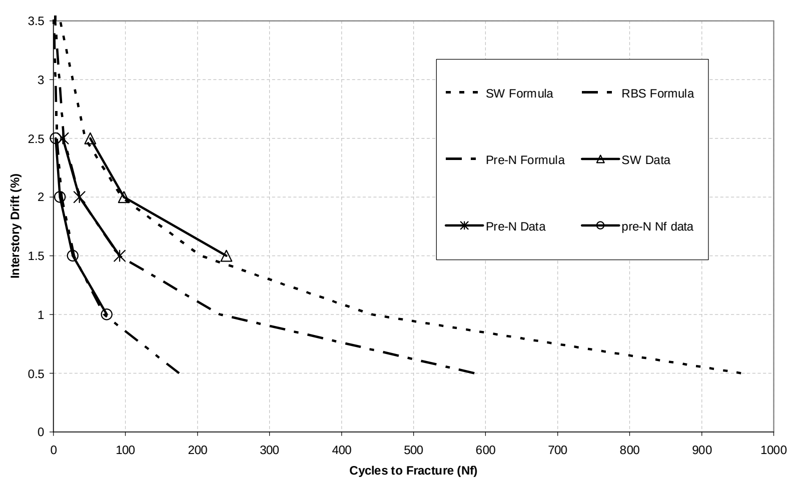


Figure 2. Sample fatigue life test data, W18x40 beams.

A major obstacle to implementing fatigue damage calculation on a large scale is the lack of fatigue life curve data for beam and connection sizes. Each curve requires multiple tests to failure making it cost prohibitive to experimentally obtain the necessary data. However, a recent study (Kuwamura and Takagi, 2004) documents several similitude laws that can be used to extrapolate limited data to full fatigue failure curves. The study determines that the number of cycles to failure can be expressed through a relationship that does not depend upon the beam size or connection type. One of the similitude equations is

$$N_f = \frac{\eta_{pM}}{\mu_p}\left(\frac{\eta_{pM}}{\mu_p} - 1\right)^{\frac{2}{3}} \qquad \frac{\eta_{pM}}{\mu_p} \geq 2 \tag{2.3}$$

where η_{pM} is the monotonic fracture ductility, and μ_p is the cyclic ductility. For the data in Table 1, the value of η_{pM} is 35, 20, and 4.5 for the slotted web connection, reduced beam section, and pre-Northridge connection respectively. The values of η_{pM} are comparable to those found during monotonic testing by Kuwamura and Takagi (2004) which ranged from 5.9 to 44.5 depending upon the steel material and connection details. The measured data and similitude relationships for the test results presented in this paper are shown in Figure 3. It thus appears that if the monotonic fracture ductility can be determined for a beam size and connection type, the fatigue life curve can be reproduced with only limited additional testing for validation. Work is currently ongoing to determine the relationship between η_{pM} for different beam sizes.

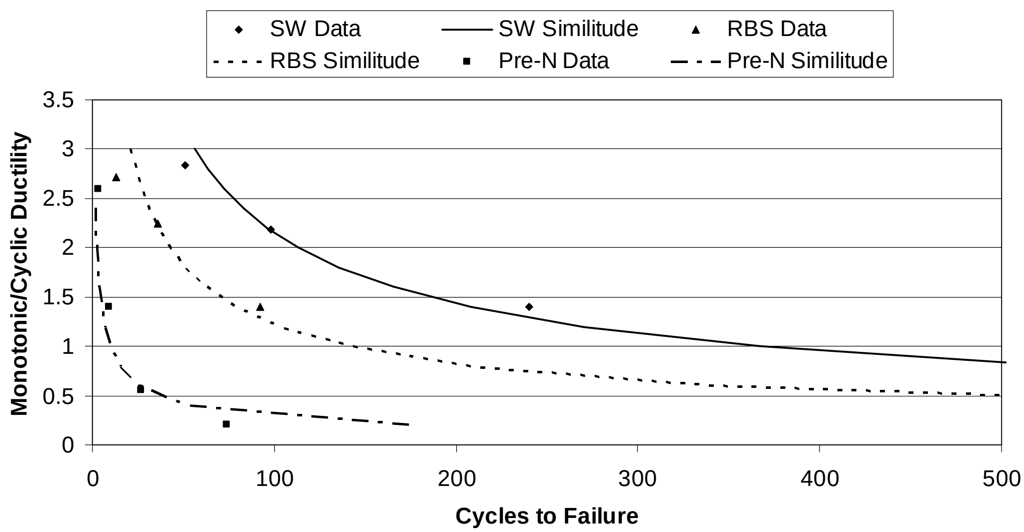
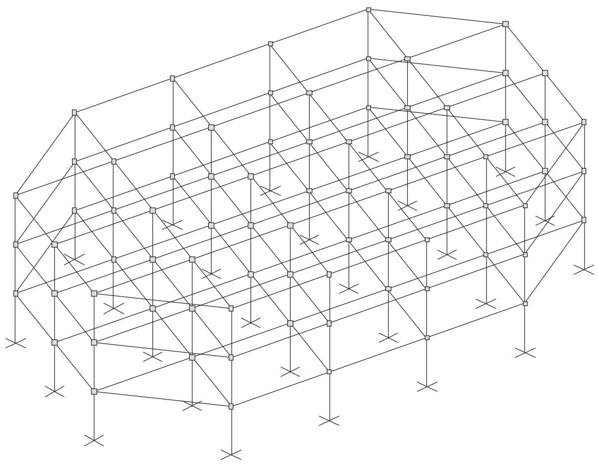


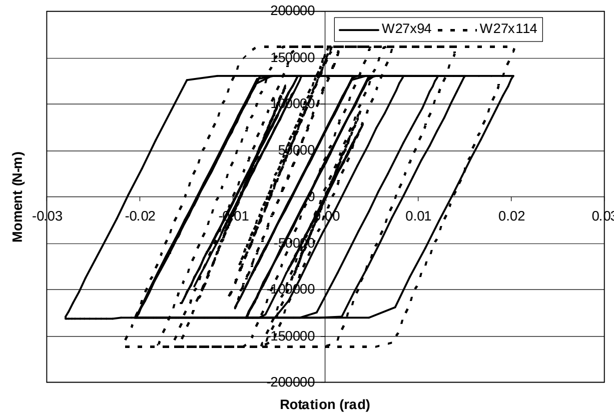
Figure 3. Similitude relationships and test data.

3. EXAMPLES

An example illustrates the method and interpretation of the results. The structure is a moment resisting frame (Figure 4a) with W27x94 girders, W14x159 columns, and doubler plates added to the panel zone to ensure their response remains elastic. The structure is subjected to gravity loads of the dead load plus a 1200 Pa live load followed by an earthquake with a peak ground acceleration of 0.632g. The nonlinear behavior and design limit states of the elements were modeled in accordance with the guidelines in ASCE-41 and the calculations were carried out in Perform-3D (CSI, Inc.).



a. Example structure.



b. Sample beam end moment-rotation responses.

Figure 4. Illustrative example structure and moment-rotation response history.

Usage ratios for the ASCE-41 limits states are obtained directly from Perform-3D for the Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) levels. The *FDI* numbers were calculated using a custom program for three connection types: pre-Northridge, reduced beam section (RBS), and slotted web (SW). The usage ratios and FDI for the most highly loaded beam connection were:

Table 3.1. Damage parameters for W27x94 beam example problem.

ASCE-41 Usage Ratios			Fatigue Damage Index		
IO	LS	CP	Pre-N	RBS	SW
4.2	0.7	0.53	1.24	0.40	0.13

The results illustrate the relationship between the damage measures. According to the ASCE-41 criteria, the connection has failed the IO level but passed the LS level. This suggests that there is some structural damage requiring repair, and that non-structural damage is likely. The same result is obtained for the pre-Northridge *FDI* which indicates that a fracture has occurred at this connection. The FDI level more closely corresponds to the LS limit state. The reason for the difference can be seen in Figure 4b. The LS limit state for end rotation is a single value exceeding a rotational ductility of 6. In this case, there are multiple cycles at ductilities of 3, 4, and 5. Each of these cycles damages the connection, but the cumulative effect is not considered in the ASCE-41 limits.

The beam sizes were changed to W27x117 in order to exam the differences between the two calculations. The resulting damage parameters, and the difference with the W27x94 beam calculations, are shown in Table 3.2. The ASCE-41 usage ratios show a decrease of 18-21%, corresponding to the lessening of the peak end rotation. The FDI decreases are significantly larger, up to 62%. Examination of Figure 4b provides the explanation. In addition to a lower peak rotation, the rotation at each cycle is also decreased. Since the majority of the fatigue damage occurs in these smaller cycles, due to the larger quantity, the overall FDI decreases much more than would be indicated by looking at only the peak values.

Table 3.2. Damage parameters for W27x117 beam example problem.

	ASCE-41 Usage Ratios			Fatigue Damage Index		
	IO	LS	CP	Pre-N	RBS	SW
	3.3	0.55	0.41	0.47	0.15	0.06
Difference w/W27x94 (%)	21	21	18	62	62	54

4. CONCLUSIONS

This paper examined the use of low cycle fatigue to predict the damage to steel frame structures subjected to earthquakes. Each step of the calculation process was described, and the procedures that are not typically used in structural engineering were detailed. Example problems were presented to illustrate the comparison between *FDI* results and the damage predictions using ASCE-41. In addition, the effect of different connection types was examined.

The fatigue damage index calculation is based on solid engineering principles and is simple to implement. The calculation provides additional information beyond those available using peak response parameters to quantify damage. In particular, an estimate of the remaining usable life of a member is calculated, allowing the engineer to predict the cumulative effect of multiple earthquakes over the life of a structure. However, it should be noted that the *FDI* is not a replacement for traditional engineering damage measures. Rather, it gives the engineer an additional tool that can be used to better understand the performance of their design under seismic loads.

REFERENCES

ASCE/SEI Standard 41-06 (2006). Seismic rehabilitation of existing buildings. *American Society of Civil Engineers*, Reston, VA.

ASTM Standard E1049-85 (2005). Standard practices for cycle counting in fatigue analysis. *ASTM International*, West Conshohocken, PA.

Barsom, J. and Pellegrino, J. (2002). Failure analysis of welded steel moment-resisting frame connections. *Journal of Materials in Civil Engineering* **14:1**, 24-34.

Bertero, V. and Popov, E. (1965). Effects of large alternating strains of steel beams. *Journal of the Structural Division* **91:ST1**, ASCE.

Kasiraj, I. (1972). Fatigue failure of nonlinear multistory seismic structures, *Journal of the Structural Division* **98:ST3**, ASCE.

Krawinkler, H., Zohrei, M., Lashkari-Irvani, B., Cofie, N. and Hadidi-Tamjed, H. (1983). Recommendations for experimental studies on the seismic behavior of steel components and materials. *Report No. 61*, The John A. Blume Earthquake Engineering Center.

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.

Kuwamura, H. and Yamamoto, K. (1997). Ductile crack as a trigger of brittle fracture in steel. *Journal of Structural Engineering* **123:6**, ASCE.

Kuwamura, H. and Takagi, N. (2004). Similitude law of prefracture hysteresis of steel members. *Journal of Structural Engineering* **130:5**, 752-761.

McCabe, S., and Hall, W. (1989). Assessment of seismic structural damage. *Journal of Structural Engineering* **115:9**, 2166-2183.

Miner, M. (1945). Cumulative damage in fatigue. *Journal of Applied Mechanics*.

Park, Y., Ang, A. and Wen, Y. (1984). Stochastic model for seismic damage assessment. *ASCE Proceedings of the 5th Engineering Mechanics Division Specialty Conference*, 1168-1171.

Partridge, J, Paterson, S. and Richard, R. (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*, Montreal, Canada.

Richard, R., Allen, J. and Partridge, J. (2001). Accumulated seismic connection damage based on full scale low cycle fatigue connection tests. *SEAOC Annual Convention*, San Diegro, CA, USA.

Stojadinovic, B. (2003). Stability and low-cycle fatigue limits of moment connection rotation capacity. *Engineering Structures* **25**, 691-700.

Suidan, M. and Eubanks, R. (1973). Cumulative fatigue damage in seismic structures. *Journal of the Structural Division* **99:ST5**, ASCE.

Taucer, F., Negro, P. and Colombo, A. (2000). Low-cycle fatigue cyclic and PSD testing of a two-storey moment resisting steel frame with beam-to-column welded connections. *Journal of Earthquake Engineering* **4:4**, 437-477.