

# Low Cycle Fatigue Tests and Fracture Analyses of Bolted-Welded Seismic Moment Frame Connections – July 3, 2000

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## ABSTRACT

There was extensive damage to steel beam-to-column moment connections in several hundred buildings caused by the January 17, 1994, earthquake in Northridge, California. Similar damage was found during 1995 inspections of buildings that experienced the 1989 Loma Prieta earthquake in the San Francisco area.

Presented herein are stress analyses, strain gauge instrumented results, and metallurgical fracture surface analyses that demonstrate the assumed state of stress in the “pre-Northridge connection” was not correct. The analyses indicate that the stress was so unfavorable as to cause the pre-Northridge connection to fail in low cycle fatigue with four distinct areas defined as failure initiation sites.

It is important to note that low cycle fatigue was shown to be a major problem with this connection in 1992 by researchers in Japan.

## 1 INTRODUCTION

Extensive damage to steel beam-to-column Fully Restrained (FR) moment connections in several hundred buildings was caused by the 17 January 1994 earthquake in Northridge, California. These connections, used in Moment Resisting Frames (MRF), were fabricated with the beam flanges attached to the column flanges by full penetration welds and with the beam webs bolted to single plate shear tabs, which were welded to the column flange. Earthquake induced fractures originated in the MRF connection welds and/or in the beam and column base metal at joint rotations of 0.5% to 1.5% story drift, which the structural engineering design community considered unexpectedly low values (Kaufmann 1997). Post earthquake analyses of the test data used in developing the pre-Northridge connection indicated significant failure rates during the test programs (Bertero, Anderson, & Krawinkler 1994). The “abrupt” failures and “failures at low rotation” combined to create a failure rate of greater than 50%. In one series of tests, the first 6

specimens “failed very abruptly” in the weld or the heat affected zone (HAZ) of the base metal (Popov, et al. 1985). In some of the university laboratory tests the failure mode was described as identical to the field observed pre-Northridge divot failure with initiation of the crack at the weld root at the bottom beam flange and propagation into the column flange (Anderson & Linderman 1991). After a series of six full-scale tests performed at Lehigh University (Sarkisian 1985), which used large cantilever beam/column connections, the following statement was made: "Numerous specimens required weld repairs in the early test cycles. Fracture at welds dominated these failures. Laboratory repairs were relatively easy and inexpensive, but this would not be the case in field repairs."

It is important to note that none of the metal failure surfaces cited above were closely examined visually, optically, or by electron microscopy to determine the exact metallurgical explanation for the “premature failure.” Instead, the failures were improperly attributed to workmanship including

welding and inspection. The one exception is contained in a British Steel Report (Harrison 1995), which was written following an investigation of “pre-Northridge” beam/column connections, which failed prematurely in testing at the University of Texas, Austin, in 1994. In this report the British cite an appearance of the failure initiation in the column base metal and indicate further that the type of failure indicates a level of strain much greater than their current design theory would predict. Experimental studies performed at Smith-Emery Laboratories (Richard, et al. 1995) have shown measured flange tension strain values at the extreme fiber-in-bending that are five times higher than nominal flange strain. Additionally, compressive strains were measured on the obverse side of the tension flange. Moreover, subsequent experimental strain gauge results and finite element analyses have shown that this connection has large stress and strain gradients both across and through the welds in the beam flanges at the column flange face (Allen, et al. 1995). Under seismic loading, these large strain gradients induce very large dynamic ductility demands on the elements of the connection during the cyclic elastic and inelastic straining and force redistribution in the connection. These observations and conclusions are consistent with previous research made in Japan (Kuwamura & Suzuki 1992, and Kuwamura & Yamoto 1997).

#### *Finite Element Analysis of the “pre-Northridge” Connection*

Presented here are finite element model (FEM) results made to show the stress, strain and force distributions in the field-bolted, field-welded moment connection which now has become known as the “pre-Northridge” MRF connection. Use is made of the ATC-24 (Applied Technology Council) connection test protocol assembly that is shown in Figure 1. This moment frame subassembly, which is “pin” supported at the top and bottom of the column and loaded with a laterally supported beam tip load, was designated by the ATC to provide a reasonable simulation of seismic cyclic loading for the connection. Figure 2 is a close-up view of one of the finite element plate-model meshes used in the connection analyses. Several mesh sizes were evaluated to ensure that the very high stress and strain gradients in the beam flange and weldment

were properly obtained (Richard, et al. 1995). These high fidelity models that are required to capture the high stress and strain gradients in the connection, usually consisted of about 10,000 four-node plate bending elements having about 10,000 degrees of freedom. Solid element sub-models used to evaluate internal stress and strain distributions in the beam and column flanges and in the welds comprised 80,000 elements and 100,000 degrees of freedom. Flanges were modeled using six layers of hexahedrons whereas the webs and shear plate were modeled using four layers. The stress and strain results of the plate and solid models generally were in agreement to within ten percent. Bonded strain gauge data was utilized to confirm the FEM results.

Shown in Figures 3 and 4 are the top, middle, and bottom surface flexural stress and strain distributions, respectively, in the beam flange/weld of the tension flange of a W920X233 (W36X150) beam connected to a W360X433 (W14X311) column using a standard (pre-Northridge) connection. Twenty-five millimeter (1 inch) thick fully extended column continuity plates were used that matched the beam flange thickness. The nominal (strength of materials) flexural beam flange/weld stress at the column face is 344 Mpa (50 ksi). However, the linear finite element analysis in Figure 1 shows very large stress and strain gradients horizontally across and vertically through this tension flange. The maximum flexural stress is 1212 Mpa (176 ksi) tension at the top and center of the flange whereas at the bottom and center of this *tension* flange the flexural stress is 206 Mpa (30 ksi) compression. These stresses are based upon a linear analysis so that these stress and stress gradient intensities are much larger than those obtained by an elastic-plastic stress analysis. However, the opposite is true for the strain and strain gradients, which become much higher and much more severe as the beam flange becomes plastic. Both local and global ductility are required for stress, strain, and force redistributions in the MRF connection to prevent premature fracture modes and must be considered in the design of the connection. In any fracture study it is these FEM obtained stress/strain values, which must be employed so that the actual demand values of stress or strain are used in any calculations performed in a metallurgical failure/fracture analysis.

## Connection Ductility Demands

Due to the fact that the maximum tensile strength is exceeded by the demand placed upon the pre-Northridge joint design, it must be recognized that the true demand upon the structure must be discussed in terms of strain. Neuber's Theorem (Neuber 1961), is widely used to evaluate both the stress and strain demand distributions in specimens containing geometric/shape-caused stress concentration. This theorem postulates that the product of the stress and the strain concentration factors evaluated in either the elastic or inelastic range is equal to the square of the elastic stress concentration factor:

$$K_{\text{stress}} \times K_{\text{strain}} = (K_{\text{scf}})^2$$

For example, if the initial elastic stress concentration factor is 4.0 ( $K_{\text{scf}}$ ) and a final plastic stress concentration factor is 1.0 ( $K_{\text{stress}}$ ) due to material inelastic behavior and stress redistribution (e.g., uniform flange stress), then the actual strain concentration factor ( $K_{\text{strain}}$ ) at this plastic stress level is 16.0. From this it is apparent that a great deal of ductile behavior is needed to meet this strain requirement. If the loading is dynamic, this puts even more adverse demands on the ductility requirements; when the strains increase dramatically, so do the strain rates when the loading is dynamic. High strain rates increase the yield strength of ductile steels and can initiate brittle fracture in connections that would behave in a ductile manner under low strain rates (Collins 1993). Typical elastic stress ( $K_{\text{stress}}$ ) and strain ( $K_{\text{strain}}$ ) concentration factors in the tension beam flange/weld in the pre-Northridge, dogbone, and cover plate connections range from 4.0 to 5.0 (Allen, et al. 1995) and depend primarily upon the beam and column flange width and thickness ratios.

A second linear analysis of the W360X433 (W14X311) and W920X233 (W36X150) assembly was made without the 25 mm (1") fully extended continuity plates in the column. This design modification changed the above maximum stresses at the center of the top and bottom surfaces of the tension flange to 1357 MPa (197 ksi) tension and 138 MPa (20 ksi) *compression*, respectively, as

shown in Figures 5 and 6. This relatively small increase in the maximum stress (about 10%) indicates that continuity plates in MRF columns are not as effective in preventing column flange flexing under seismic (lateral) frame loading. Note the very severe stress (and therefore, strain) gradient on the bottom of this tension flange from the edge of the 38 mm (1.5") weld access hole to the face of the column as shown in Figure 6. In this 38 mm distance the stress goes from 826 MPa (120 ksi) tension to 138 MPa (20 ksi) compression. This high-stress/strain explains the common beam flange fracture at the edge of the weld access hole where it meets the flange. It is also very apparent from the stress distributions shown in Figures 5 and 6 that strain gages would have to be located on both sides of the flange and very near the column face to detect these vertical and horizontal severe strain gradients. Such strain gauge measurements were made in 1994 (Richard, et al. 1994) when the weld metal was ground to the profile of the flange and bonded strain gauges were placed at 1/2" from the column face.

The severe stress and strain gradients, both horizontally across and vertically through the beam flange/weldment (Figs 3-6), are caused by the out-of-plane flexing of both the beam and column flanges and the presence of very significant shear loading in the flange. The resultant moment generated by this vertical shear stress gradient causes the beam flange to develop a prying action on the face of the column. This prying action can initiate the following connection fracture modes: a divot pullout in the column flange, a tensile weld fracture at the column face, a fracture of the beam flange near the weld or in the vicinity of the web weld access hole boundary, or a "k line" fracture in the beam web at the weld access hole. All of these fracture modes have been observed both in the field, and in laboratory tests of pre-Northridge connections (Engelhardt, et al. 1995). For example, a cover plate connection that has these similar large stress and strain gradients and flange distortions that result in a beam flange prying action, was tested at the University of Texas. This connection fractured with a divot pullout in the column flange. A metallurgical examination of this fracture (Harrison 1995) indicated the final fracture initiated in the heat-affected zone (HAZ) of the column flange. The

initiation region that was at the defined high stress *and strain* location was in the column flange and was characterized by a flat fracture appearance and consisted of a mixture of ductile microvoid fracture and fine grained cleavage. This type of failure indicates a significantly higher strain demand than predicted to be present at the time with pre-Northridge analytical methods.

### *Beam Shear Force Distributions*

A recent finite element study (Goel 1997) has shown that the shear force distribution in "pre-Northridge" connections differs drastically from that predicted by classical Bernoulli-Euler beam theory that lead to the popular design concept wherein "the flanges carry the moment and the web carries the shear." Shown in Figures 7 and 8 are top and bottom flexural stress distributions of the tension flange of a Reduced Beam Section (RBS or dogbone) for a W920X233 (W36X150) and W360X433 (W14X311) test assembly. These stresses are essentially identical in magnitude and distribution to those shown in Figures 5 and 6 for the "pre-Northridge" connection. Shown in Figures 9 and 10 are the shear stress and von Mises strain distributions, respectively, for the beam and column webs. Both the shear stress and von Mises strain at the center of the beam web are essentially zero. The result of this shear stress distribution is a dramatic increase in the vertical shear force carried by the beam flanges, which in turn overloads the beam flange welds. The finite element analysis shows that for this beam and column combination, 50% of the shear is carried by the beam flanges, while only 50% is carried by the shear tab. The previous assumption was that 100% of the shear was supported by the shear tab. This unfavorable distribution of shear has been known for many years (Yu 1959). Subsequent confirmation of this marked departure from classical beam theory at support locations was cited by Abel and Popov (Abel & Popov 1968). The effect of this flange shear in the causation of divot failures in the column flange has been long known (ASM, 10 1972). What remained unexplained was the linkage between the state of stress in the flange and the ultimate failure mechanism of low cycle fatigue, which is presented here.

### *Strain Gauge Measurements*

In March of 1996, ATC-24 cantilever beam/column tests were performed using a heavy pre-Northridge connection design. For the first time, "post-yield" strain gauges were utilized, which would remain effective in obtaining data up to approximately 3% strain. One hundred and twenty (120) strain gauges were bonded to various areas of the beam/column connection. The flange gauges were located ½ inch from the column face. After completion of the tests, only two flange gauges were isolated and data plotted; one gage was the center top, the other was the center bottom.

Upon plotting only these two ATC-24 cyclic test results, it was found that after yield was reached in the beam flange weld at the column face, the gauge readings would not return to zero stress state upon reverse cycle loading of the cantilever beam (Fig. 20). It was found that the gauges near the center of the beam flange weld at the column face exhibited permanent compressive strain after unloading of the cantilever beam. Furthermore additional loading cycles produced additional incrementally increasing peak compressive strain and an ever-increasing strain range values at the bottom surface of the lower flange (Fig. 20). This observation indicates that successive cycles of loading serve to produce an ever-increasing accumulation of plastic strain in the weldment even though the global applied displacement was constant. This ever-increasing accumulation of plastic strain of the center of the beam flange region is referred to as "ratcheting."

Under ratcheting conditions the likely damage mechanisms are sub-critical fatigue crack growth or ductility exhaustion. As will be discussed in the following section, all the samples tested exhibited crack initiation and subcritical crack propagation by low cycle fatigue. After some amount of sub-critical crack growth (typically more than 50% of the beam flange cross-sectional area), crack extension by ductile tearing or brittle fracture occurred.

### *Fatigue Test Procedure*

The cantilever beam/column tests were run similar in test procedure to the ATC-24 protocol with the exception that the tip displacement of the cantilever

beam was made a constant value from initiation of the test until failure of the connection occurred. The displacement of the beam was defined in terms of story drift for ease of comparison with usual dimensions of structural calculations.

At the conclusion of testing, the failure surfaces were cut from the beam/column assembly, the fractures surfaces were protected, and the specimens were delivered to Aptech Engineering Services, Inc. for metallurgical evaluation. Tensile specimens were cut from beams and columns. Weld filler metal specimens were provided from a single separate weld-filler-metal mockup from which tensile and charpy tests were taken.

Each individual constant amplitude fatigue test is accompanied by its hysteresis loop which displays load displacement and all materials test results. The control of the test cycling was accomplished utilizing displacement actuated electric-hydraulic controls to insure that each maximum displacement was very accurately achieved and repeated; one plot of cyclic displacement is included. All deflection measurements were made using electrical displacement transducers with traceable calibration.

Figures 22-31 illustrate the test configuration for each of the 10 identical test specimens.

### Test Results

In an effort to substantiate the presence of fatigue, a series of constant amplitude pre-Northridge cantilever beam/column tests were run with the prospect of plotting an “S-N” curve to describe stress level vs. number of cycles to failure for the connection. The goal of such a plot is the determination of the safe design life of an assembly subject to seismic fatigue loading. Results of the first series of tests are presented in Figure 21 and were presented at the Structural Engineers Association of California (SEAOC) convention in October 1999 at Santa Barbara.

Nine additional tests were performed on connections that were made using tough weld consumables (E71 T-8, Lincoln NR232) with the backing bars removed and reinforcing fillet welds added to the weld roots. The configuration of the

tests and results are attached on Figures 22-31. A plot of the “S-N” curve for the illustrated connection is shown in Figure 32. The weld parameters, materials properties, failure mode, and hysteresis loops for each test are included on Figures 22-31. The results of the test are as follows:

Test Number	Amplitude, %	Story Drift Cycles at Failure
#1 w/ b/u	1.5	3-1/2
#2 w/ b/u	1.5	2-1/4
#3	1.5	29
#4	1.5	22
#5	1.2	61
#6	1.2	18
#7	1.2	60
#8	0.9	98
#9	0.9	82
#10	0.7	408

In an effort to determine anticipated connection cyclic life as well as safe design limit and to display classic low-cycle connection behavior, a plot of number of cycles to failure ( $N_f$ ) vs. story drift was made (Figure 32). It is common to present low cycle fatigue test results on a log strain range v. log  $N_f$  plot.

Since in the present case the applied strain range at the locations of cracking is complex and cycle dependent, the results are presented in terms of story drift instead of strain range. It was found that the results presented in this fashion produced the following least squares fit relationship:

$$\text{Story Drift, \%} = 2.085 (N_f)^{-0.1597} \quad (1)$$

The lower bound (5<sup>th</sup> percentile) of story drift was found to approximately 78% of the least squares fit best estimate. For example, to use the beam to column configuration tested and be assured that the connections could sustain 1000 cycles, the structure would need to be designed for a maximum story drift of:

$$\text{Maximum Story Drift (1000 cycles)} = 0.78 \times 2.085 (1000 \text{ cycles})^{-0.1597} = \underline{0.54\%}$$

Recall that this is well below the story drifts experienced by structures in the Northridge earthquake.

## *Metallurgical Characteristics of Failures*

Six of the test specimens were examined in detail to determine where cracking initiated and the characteristics of crack growth. The test specimens examined were each fabricated using the basic pre-Northridge design, but were made with high toughness weld consumables, and had the backing bars removed and a 3/16" to 1/2" reinforcing fillet added to the weld root. In five of the six samples the run-on/run-off tabs had been removed and the ends of the welds ground to a smooth radius. The results of these evaluations are summarized in Table 1 on the following page. In every case it was found that subcritical cracking, by a low-cycle fatigue mechanism, preceded unstable fracture initiation and failure. Typically, more than 50% of the fracture surface (or cross-section the beam flange) exhibited "beach markings" indicative of low cycle fatigue.

The constant displacement amplitude tests presented at the SEAOC convention experienced failures in the weld metal or as divot pull outs of the column face (Bertero, et al. 1994). This particular fatigue failure mode dominates due to the fact that the welds in these tests were ground flat to the profile of the beam flange with but a small radius at the column face.

In the subsequent tests of February and March 2000, the welds were left unground and hence were slightly reinforced (i.e., 1/8" to 1/2" fillet welds) in profile. As a consequence, the welds cracked only slightly at the center of the beam flange near the column. In these test samples low cycle fatigue crack initiation occurred in three distinct locations (listed in order of frequency of occurrence):

- In the beam flange base metal at the surface of the weld access hole, approximately 1/4" from the beam flange surface
- From small, acceptable weld discontinuities located near the middle and/or ends of the beam flange to column flange weld
- At the weld interface at the toe of the reinforcing fillet weld at the root of the beam flange to column flange weld.

In a number of specimens LCF initiation occurred at more than one of the three locations described

above, only one of the cracks grew to an unstable size. For example, Test Sample #8 exhibited crack initiation from all three common locations (Fig. 11):

The dominant crack in Test Sample #8 was initiated at multiple locations within the weld deposit. The following weld discontinuities were present at the most obvious initiation sites (Fig. 12):

- A 1/64" x 7/32" slag inclusion
- A 1/64" diameter porosity
- A 1/64" diameter slag inclusion

These are all extremely small discontinuities and are characteristic of excellent weld workmanship.

A small secondary crack was observed in Test Sample #8 at the weld toe of the reinforcing fillet weld at the weld root (Fig. 13). Although this crack did not extend through the thickness of the beam flange it is believed to be a possible initiation location.

Test Sample #8 also exhibited a 0.6" long crack that initiated from the edge of the weld access hole at a location approximately 1/4" above the top surface of the lower beam flange (Fig. 11). This crack was transgranular (Fig. 14) and not influenced by a heat affected zone from flame cutting of the weld access hole (the weld access holes were ground prior to testing). This weld access hole crack was opened up by saw cutting it from the beam, cooling in liquid nitrogen and breaking it open. The fracture surface exhibited distinct "beach marks" typical of fatigue fracture. Examination in a scanning electron microscope confirmed that the cracking was characteristic of low cycle fatigue (Fig. 15). The spacing of the fatigue striations (i.e., the crack progression during a loading cycle) were measured and used to estimate the magnitude of the stress at this locations.

Test Specimen #7 exhibited the most common mode of cracking and failure observed in these tests. In this case crack initiation occurred only at the weld access hole. Following crack initiation, sub-critical crack growth occurred along the beam k-area and eventually propagated across the thickness of the beam flange (Fig. 16). More than 50% of the beam flange cross-sectional area

cracked by LCF prior to initiation of ductile tearing instability. (Fig. 17). Within the LCF portion of the fracture, distinct beach marks were observed.

<b>Sample</b> (Displacement/Story Drift/Cycles to Failure)	<b>Findings</b>
#3 – Bottom Flange (1.9"/1.5%/29 cycles)	Low Cycle Fatigue (LCF) crack initiation at the weld access hole at a location approximately 14" above the top surface of the beam flange. The first inch of the fracture propagated by LCF parallel to the top surface of the beam flange. At a crack depth of 1" the striation spacing was approximately 1/10". The second inch of the fracture propagated by LCF across the thickness of the beam flange. After crossing the beam thickness of the beam flange, the crack bifurcated at an included angle of approximately 60°. The LCF continued along the two cracks for another 1-1/2". The striation spacing along this portion of the cracks was approximately 0.2" to 0.3". The final 1-1/2" to 2" of fracture (on both sides of the beam flange) occurred by ductile (shear) fracture.
#4 – Top Flange (1.9"/1.5%/22 cycles)	Very similar to the LCF cracking from the weld access hole seen in Samples #3, #5, and #7 except that the crack immediately propagated across the beam flange thickness (rather than growing parallel to the flange surface). There was also a small secondary crack near one end of the weld.
#5 – bottom Flange (1.5"/1.2%/18 cycles)	Very similar to #3 except that the striation spacing at a depth of 1" was approximately 0.06". In addition, a secondary crack initiated by LCF from multiple, very small weld imperfections located near the beam-to- column weld runoff tab.
#6 – Bottom Flange (1.5"/1.2%/18 cycles)	LCF cracking initiated from small weld imperfections located at both the edge and the middle of the beam flange to column flange weld. Approximately 70% of the flange cross-section exhibits LCF. The remaining 30% exhibits cleavage fracture (with one pop-in region prior to final failure). There was also a small secondary crack that initiated at the toe of the weld root fillet weld.
#7 – Bottom Flange (1.5"/1.2%/60 cycles)	Very similar to #3 and #5. The LCF striations in this fracture are very distinct. The striation spacing versus crack depth is documented in Figure 1.
#8 – Bottom Flange	LCF cracking initiated from multiple sites near the edge of the beam flange to column flange weld (even though the edges were radiused and ground). The initiation sites included: (i) a 1/64" x 7/32" slag inclusion, (ii) a 1/64" diameter porosity, and (iii) a 1/64" diameter slag inclusion. The LCF from these sites grew as penny-shaped cracks, then linked up and propagated by LCF across approximately 1/2 the beam flange and then propagated by cleavage fracture. This sample also exhibited a 0.6" deep LCF crack at the weld access hole (see striation spacing, Kmax, and Peak Stress estimates in Figure 2).

Table 1. Summary of Metallurgical Evaluation of Test Specimens #3, #4, #5, #6, #7, and #8

The spacing of these fatigue striations were measured and used to estimate the magnitude of the stress which produced them. Approximately 42 striations were visible on this fracture. Recall that this sample failed in 60 cycles.

#### *Quantitative Fractography*

Measurements of the LCF striation spacings measurements in the weld access hole crack

locations of Test Samples #7 and #8 were made. These were used to estimate the magnitude of the stress present at this crack susceptible location. This estimation was made in a two step process. First, the maximum-stress intensity factor, Kmax, was estimated from the striation spacings. Next the peak stress amplitude was estimated from knowledge of the relationship between Kmax and peak stress at a given crack depth.

The maximum stress intensity factor, Kmax,

was estimated using the empirical relation for structural steels given by (Barsom & Rolfe 1987):

$$da/dN = 2 E-10 \Delta K^3 \quad (2)$$

where:

$da/dN$  = crack growth per cycle, inch/cycle

$\Delta K$  = stress intensity factor range =  $K_{max} - K_{min}$ , ksi-sqrt(inch)

This empirical relationship correlates the average fatigue growth for a given applied stress intensity factor range. It is expected that this relationship will somewhat overestimate  $K_{max}$  for very low cycle fatigue, especially for loading with high stress ratios,  $R$  ( $R = \sigma_{min}/\sigma_{max}$ ). In the current case the stress ratio is negative  $R \approx -1$  and the relationship above should provide reasonable estimates of  $K_{max}$ .

A second relationship was used to estimate the peak stress from knowledge of  $K_{max}$  and the crack depth associated with the striation spacing:

$$K_{max} = F \sigma_{max} \sqrt{\pi a} \quad (3)$$

Where:

$F$  = stress intensity factor correction factor  $\approx 1.12$  for shallow cracks surface connected cracks

$a$  = crack depth, inch

The estimated maximum stress intensity factor and peak stress values for the weld access hole cracks in Test Samples #8 and #7 are shown in Figures 18 and 19 respectively. It was found that for crack growth along the beam k-area, the estimated value of  $K_{max}$  is relatively constant and that the stress decreases with distance from the surface of the weld access hole. The table on Page 9 summarizes these estimates:

## 2 CONCLUSIONS

Based on the findings documented herein, the following conclusions are made:

- The field-bolted, field-welded, "pre-Northridge" beam-to-column connection is fundamentally flawed and should not be used in new construction.
- The field-bolted, field-welded, "pre-Northridge" beam-to-column connection has large stress and strain gradients horizontally across and

vertically through the beam flange/welds at the column face.

- The field-bolted, field-welded, "pre-Northridge" beam-to-column connection geometry and stiffness results in both moment and shear distributions drastically differ from the assumptions usually made wherein "the flanges resist the moment and the web resists the shear". Typically, in this connection the shear is equally shared by the beam flanges and the beam web, which overloads the beam flange welds.
- Although the stress concentrations reduce significantly as plastic behavior occurs, the strain concentrations and therefore the strain rates increase dramatically and place severe ductility demands and force re-distributions in all of the elements in the connection.
- With cyclic seismic loading the connection is prone to ratcheting. This will produce an ever-increasing strain range even under constant displacement loading creating poor low cycle fatigue performance.
- By performing a series of constant displacement (i.e., story drift) low cycle fatigue tests it is possible to quantify the long term reliability and performance of candidate beam to column connections and structures.
- The dominant detriment of low cycle fatigue performance is the magnitude of stress. Achievement of desired cyclic seismic fatigue life for anticipated story drift will require improvements to the connection design to minimize the stress/strain concentration factors.
- Designs which transfer loads, stress, and strain more smoothly through the connection are likely to provide better seismic performance than designs which "weaken the beam" or "strengthen" (and stiffen) the connection.
- Fatigue failure will occur in either the weld metal, column face or beam web, or flange at the weld access hole, depending the stress/strain concentration factors at each location and upon the cyclic response of the weld and base metals.
- Detailed finite element analysis can be used to determine the likely crack initiation locations.
- Detailed finite element analyses in combination

with appropriate constitutive equation modeling can be used to determine the low cycle fatigue life.

### 3 ACKNOWLEDGMENTS

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Test Specimen	Story Drift	Nominal Beam Flange Stress, ksi	Estimated Peak Stress at the Weld Access Hole	Effective SCF at the Weld Access Hole
# 8	0.9%	Yield	150 ksi	4.2
#7	1.2%	Yield	200 ksi	4.2

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