A Framework for Forensic Examination of Earthquake-Induced Steel Fracture Based on the Field Failures in the 2011 Christchurch Earthquake

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SUMMARY:
A detailed forensic analysis of field fractures of Eccentrically Braced Frames (EBFs) is presented. These fractures, which occurred during the February 22, 2011 Christchurch, New Zealand earthquake are the first field fractures observed in EBFs worldwide. Material from the fractured frames, including the fracture surfaces were obtained as part of this study. Thus, detailed assessments could be made of the various factors that contributed to the observed failures. The study encompasses frame simulations of the constructed frames, continuum finite element simulation of the details and material tests to establish fracture toughness of the material. The data from these tests and simulations is synthesized to ascertain the relative effects of various factors (frame design, detailing, material properties, and ground motion intensities) and to propose strategies for mitigating future occurrences.

Keywords: Eccentrically Braced Frame, earthquake, fracture, forensics, Christchurch

1. INTRODUCTION

The Christchurch, NZ earthquake (M = 6.3) of February 2011 created widespread damage in the city of Christchurch and surrounding areas. The epicenter was within 10 km of the city at a shallow depth of approximately 5 km, made the earthquake especially devastating. As a result of the intense shaking, the first observed field failures worldwide of steel Eccentrically Braced Frames (EBFs) were observed in the St. Asaph Street parking structure adjacent to Christchurch Hospital shown in Figure 1. Though the building did not collapse, the unprecedented field failures of these frames pose significant concerns regarding design philosophy, expected capacity and ductility, and detailing requirements for EBFs in current codes and guidelines.

Figure 1. St. Asaph Street parking garage, Christchurch, New Zealand

Two failures were observed and shown in Figure 2: the first a fracture located in the collector beam connecting to the shear link propagating up from the brace-beam connection weld (a); the second a shear localization in the link, resulting in significant plastic deformation throughout the beam web (b).

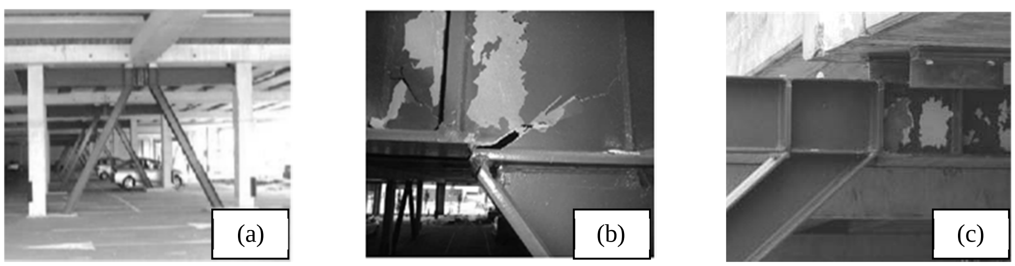

Figure 2. Eccentrically Braced Frames (EBFs) in St. Asaph Street parking structure (a), fracture (b), and shear localization (c)

Commonly assumed to be structural systems with excellent ductility (Popov and Engelhardt 1988), the failures in these frames are unanticipated and warrant careful investigation. Upon visual inspection of the fractured frame shown in Figure 2 (b), it can be observed that the brace flange was not welded directly beneath the web stiffener in the beam above, indicating that inadequate detailing may have been the primary cause of failure. The cause of failure due to the localized shearing in the frame shown in Figure 2 (c), however, cannot be determined through simple examination. This paper presents the robust methodology in which these failures are processed, demonstrating a rigorous 'end-to-end' simulation in the context of the proposed framework. Local demands resulting from actual recorded time histories from the site are obtained through high-fidelity three-dimensional frame simulations at the structural scale then inputted into a detailed local finite element (FE) model of the failed components. In addition, global and local simulations are supplemented by material property data obtained from material tests conducted on coupons extracted from the failed components. Parametric studies and probability analysis are used to pinpoint the root cause of failure. This paper will demonstrate the aforementioned framework in three distinct sections: (1) material testing (2) structural modeling and nonlinear time history frame simulations, and (3) local FE modeling and simulations, followed by a discussion of limitations.

2. MATERIAL TESTING

An important and distinguishing feature of the study is the acquisition by the research team of undamaged steel components from each of the failed Eccentrically Braced Frames (EBFs). Steel samples from failure locations were shipped to the University of California, Davis to undergo examination, 3-D scanning, and material testing. From these, specimens were extracted from locations shown in Figure 3(b) and were tested to obtain material toughness and constitutive properties; thus reducing the likelihood of introducing systematic errors in simulations attributed to inaccurately characterizing the material properties of the as-built steel. Calibrated parameters included: (1) parameters for continuum fracture and fatigue toughness models (Kanvinde and Deierlein 2006, 2008) and (2) material constitutive parameters for finite element simulation. Specimens consist of smooth tensile coupons and circumferentially notched tensile/compressive cyclic coupons. The specimens were extracted from various parts of the beam to capture material heterogeneity that occurs between base metal, weld metal and heat affected zones (Myers *et al.* 2009). These tests were supplemented by Charpy V Notch energies (provided by Forell Elsesser Engineers Inc. and Holmes Consulting Group). Results from Charpy tests and a sample tensile test are shown in Figure 4. The tests indicate the steel material is extremely ductile with upper shelf Charpy Values of approximately 250 ft-lbs.

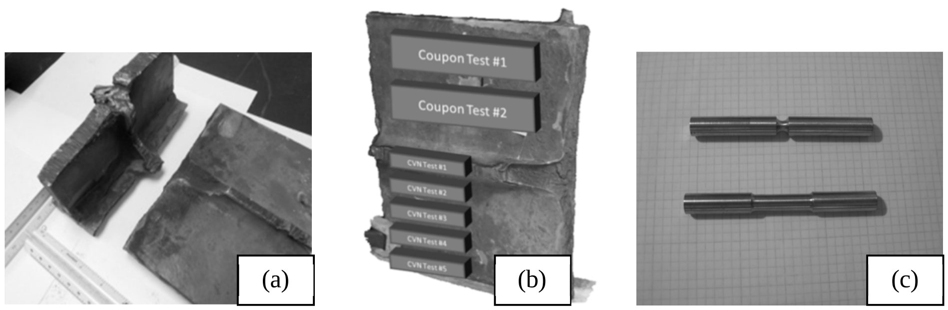

Figure 3. Undamaged samples cut from failed frame (a), locations of coupon extraction (b), and fabricated coupon samples (c)

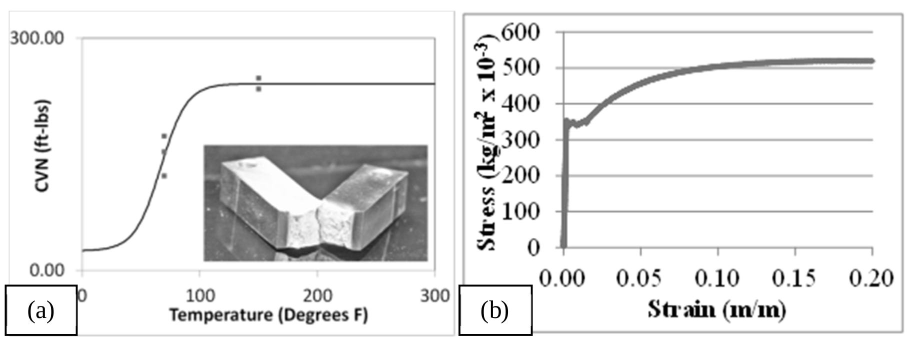

Figure 4. Charpy V Notch energy results (a), and sample stress-stain curve from extracted specimen (b)

3. STRUCTURAL MODELING AND NONLINEAR TIME HISTORY (NTH) FRAME SIMULATIONS

A three-dimensional model of St. Asaph Street Parking Garage was built using the open-source software OpenSEES in order to capture the highly nonlinear structural response of the garage to the known time histories. An important aspect of this study was to determine whether unexpectedly large building deformation demands (e.g. interstory drifts or link rotations) played a role in the frame failures. As-built drawings were obtained to accurately model the framing system for the garage. In addition to defining building geometry, drawings were utilized to assess connection flexibility, diaphragm and foundation constraining effects, and global torsional properties.

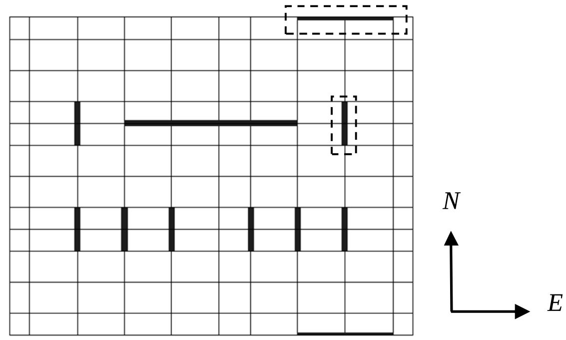

Figure 5. Plan rendering of the EBF configuration for 1st and 2nd stories. Dashed boxed frames comprise the EBF configuration for 3rd story.

Figure 6. N-E view from ground level

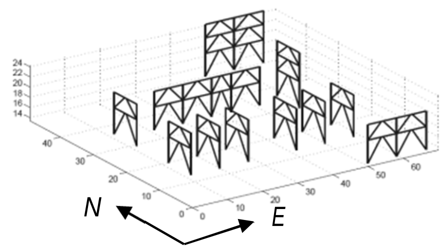

Figure 7. N-E, 3-D rendering of EBF system

3.1. Building and Ground Motion Details

The parking structure is a 3-story building consisting of reinforced concrete gravity and floor slab systems. Ascending and descending roadways run in parallel on ramps located on the North-end of the structure. The lateral resisting system is an Eccentrically Braced Frame (EBF) system configured unsymmetrically in plan along the E-W direction (see Figure 5), and symmetrically along the N-S direction in the first two stories. The third story lateral system consists of the frames indicated by dashed boxes in Figure 5. EBF columns frame into the concrete slab above, contributing to the vertical support of the slab. Collector beams and links are positioned below slabs and do not support gravity loads. Beams and braces are comprised of Universal Beam (UB-) and Universal Column (UC-) steel I-sections respectively, while columns are mainly UC- steel sections encased by square reinforced concrete sections. EBFs are connected to steel base plates embedded in concrete footings.

Local ground motion histories were obtained from strong motion data recorded at the site of the Christchurch Hospital located adjacent to the parking garage. Raw and processed data, including acceleration, velocity, and displacement time histories, were obtained through the New Zealand hazard monitoring system GeoNet.

3.2. Structural Modeling and Simulations

EBFs are designed to dissipate energy though ductile yielding of the link. In light of there being no evidence of yielding in frame components other than the link, all elements other than the link were modeled elastic to reduce simulation cost. Since link yielding was anticipated, it was essential to model the links with sufficient detail in order to accurately capture the nonlinear response of the entire structure. Based on AISC Seismic Provisions (AISC 2010), as well as New Zealand Standards (NZS3404 1997), EBF links are characterized as either ‘short’, ‘intermediate’, or ‘long’ based on length and section geometry. Links in the St. Asaph Street garage are considered “short” by both specifications. With such a characterization, it was expected that links would yield primarily due to shearing resulting from the opposing vertical forces exerted by the connecting braces (Richards 2004), though moment-shear interaction would undoubtedly be present. Research has shown that inelastic behavior of links due to shearing in a symmetric k-brace configuration can be accurately modeled using inelastic beam elements with concentrated yielding at each end (Whittaker *et al.* 1987). The yield strength assigned to the link hinges can be calibrated so that the moment-curvature curve of the link section will induce the physical shear-displacement curve, based on a proportional relationship between the applied shear and moments at the ends of link. This method, though proven reliable, introduces a non-physical parameter in the distance from the ends of the link in which the "hinges" are prescribed. OpenSEES provides the option to model elements as nonlinear elements with fiber sections, in which analysis using the Principle of Virtual Work can be employed to model the spread of plasticity throughout the element (Neuenhofer *et al.* 1998) , thus eliminating the need to assign artificial hinge locations. All links in the structure were modeled using nonlinear elements with fiber sections corresponding to specified link section geometries.

Important three-dimensional effects were considered in structural modeling and in the development of the global parameter study. Soil stiffness was selected as the global model parameter based on expected impact to overall response. Global model parameters were limited to a single variable since the focus of the study is to investigate the initiation of failure at the local level. Hence, most model parameters are assigned at the local level and are discussed in detail in the following section. EBF columns connected to shared footings were constrained to displacements in the same plane as other columns connected to that footing, while the soil flexibility was modeled using fixed calibrated axial springs connecting to each column base.

Based on as-built drawings and slab dimensions, nodes at locations where columns framed into slabs were considered constrained to a rigid diaphragm at each floor. This assumption implies minimal effect on floor stiffness due to ramp effects, and was based on the location and relative size of the ramps as well as the dimensions and properties of the slab system. Damping ratios of 2% were assigned for modes 1 and 3 to impose Rayleigh Damping on the structure. To avoid unrealistic damping forces due to nonlinear structural response, damping coefficients were determined at each time step from the mass and tangent stiffness matrices (Charney 2008).

4. LOCAL FINITE ELEMENT (FE) MODEL AND SIMULATIONS

Local continuum models of the failed Eccentrically Braced Frames (EBFs) were constructed using ABAQUS, incorporating 3-D scans of the actual failed components to model each link and, in the case of the fractured frame, the fractured portion of the collector beam. As previously mentioned, visual examination of the fractured frame indicated that the location of the brace flange was not in line with the web stiffener in the beam above. This may have induced strain concentrations near the weld during loading. Preliminary FE simulations illustrate the strain concentration at this location (see Figure 9). The parameter study pertaining to the local model of the fractured frame was conditioned to place focus on the discrepancy between the as-built and as-designed conditions of the fractured link. In addition, measured fracture toughness parameters and minimum required toughness parameters were interchanged for each geometry scenario and each link to investigate the effects on fracture initiation. All scenarios were subjected to three time histories: (1) and (2) were obtained from global response to ground motion time histories as mentioned in the previous section, and (3) was obtained using a verified test protocol to determine the theoretical capacity of the link (Richards and Uang 2006).

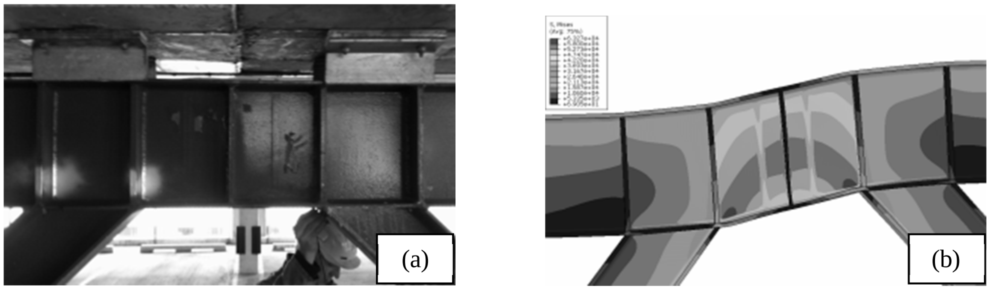

Figure 8. Photograph of failed link (a), and loaded FE model link (b)

4.1. Description of FE Model

Each local model consists of approximately 500,000 hexahedral linear elements, designated to model the elastic-plastic response exhibited by the extracted coupons during the material tests. Displacement boundary conditions were prescribed at the ends of the collector beams and braces. The three aforementioned deformation histories were imposed at these locations. Analysis to determine stress concentrations was conducted on the continuum model to determine the likelihood of fracture initiation.

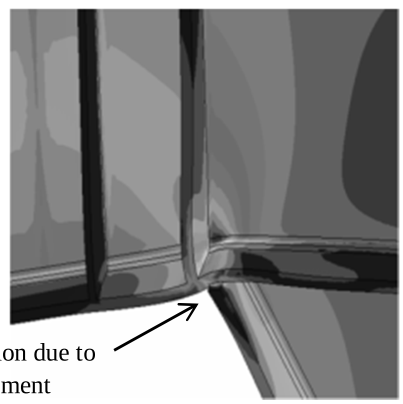

Figure 9. FE model – Stress concentration due to connecting brace and web stiffener location

5. OUTCOMES AND LIMITATIONS OF WORK

The simulation framework presented herein demonstrates not only a clear forensics examination of field failures, but also the state of the art in modeling methodology. The recorded data from the M 6.3 Christchurch, NZ earthquake from February, 2011, along with the detailed examination of the undamaged specimens from failure locations, provided a rich foundation on which a rigorous and multi-scale simulation framework could be formulated. The key findings are that the materials used in the link construction were highly ductile, and the demands were not entirely unexpected. However, as shown in Figure 9 above the misalignment of the stiffener with respect to the brace flange produced a severe stress and strain concentration leading to premature failure. Design considerations that may mitigate this type of failure are part of ongoing work.

5.1. Limitations of Work

A consequence of using a robust, multi-scale framework is the possibility of introducing an unmanageable number of model parameters. For this case study, various model assumptions were made at the global scale (e.g. rigid diaphragm, base fixity, damping properties) to allow for a sufficient number model parameters to be investigated at the local scale. These may introduce error.

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