NUMERICAL MODELING OF POST-BUCKLING BEHAVIOR AND FRACTURE OF STEEL CONCENTRICALLY BRACED FRAMES

Dimitrios G. Lignos, Ph.D.
Assistant Professor
McGill University
Montreal, Quebec, Canada, H3A 2K6
E-mail: dimitrios.lignos@mcgill.ca

Taichiro Okazaki, Ph.D.
Associate Professor
Hokkaido University
Sapporo, Hokkaido, Japan, 060-8628
E-mail: tokazaki@eng.hokudai.ac.jp

Tsuyoshi Hikino, Ph.D.
Manager
Steel Structures Engineering Division, Nippon Steel Engineering Corporation, Ltd.
Tokyo, Japan
E-mail: hikino.tsuyoshi@nsc-eng.co.jp

Kouichi Kajiwara, Ph.D.
Director
Hyogo Earthquake Engineering Center, National Research Institute for Earth Science and Disaster Prevention (E-Defense)
Miki, Japan
E-mail: kaji@bosai.go.jp

Masayoshi Nakashima, Ph.D.
Professor
Kyoto University
Kyoto, Japan
E-mail: nakashima@archi.kyoto-u.ac.jp

This paper discusses a nonlinear inelastic cyclic model to predict the effect of low cycle fatigue on the behavior of steel brace elements as parts of concentrically braced frames subjected to earthquakes. Steel braces are modeled with fiber elements that the integration points along the brace element are based on the Gauss-Lobatto quadrature rule. Fracture is incorporated in the analytical model of a steel brace and can initiate according to a rain-flow-counting rule. The fracture model is calibrated against experimental data on tubular steel braces to quantify the parameters that are critical for accurate simulation of brace fracture due to low cycle fatigue. Through a validation with a series of shake table tests that were conducted at the world's largest shaking table facility, it is concluded that the numerical model is able to predict the occurrence of brace fracture, post-buckling and post-fracture behavior of the test frame.

1. INTRODUCTION

Typical steel construction in United States and Canada has shifted during the past few years from steel moment resisting frames to concentrically braced frames (Uriz and Mahin [1], Tremblay et al. [2]). Experimental tests that were conducted in the late 1970s and 1990s [3,4,5] indicated that axially loaded members degrade with the width-to-thickness and slenderness ratios of the brace. More recently, Lehman et al. [6] indicated that secondary bending in the framing members of a steel braced frame caused by frame deformation may affect brace connections. However, modeling guidelines of steel braced frames are not well defined. Therefore, experimental data that have been conducted in the past should be utilized in order to reliably assess the seismic performance of steel braced frames subjected to severe ground motions.

This paper discusses important aspects for numerical modelling of post-buckling behaviour and fracture of steel concentrically braced frames by using a recently developed analytical model [7] for axially loaded members. Modeling recommendations are based on calibrations from past tests on individual cyclic static steel brace component tests. The numerical model for such components is validated with a recently conducted shaking table test of a concentrically braced frame that took place at the Hyogo Earthquake Engineering Research Center (E-Defense) in Japan.

2. NUMERICAL MODEL FOR STEEL BRACES

The numerical model that was originally developed by Uriz et al. [7] is utilized for modeling inelastic buckling and fracture of steel concentrically braced frames. This model is based on the force formulation proposed by Spacone et al. [8]. The advantage of such formulation is that force-interpolation functions are exact in the absence of 2nd order effects and with the use of just a single element the curvature distribution can be sufficiently represented along the entire member length. Assuming that the end forces of an axially loaded element in the local coordinate system are $\bar{p}$, then we can transform $\bar{p}$ to the global coordinate system by using a rotation transformation matrix a_r. Therefore,

$$p = a_r^T \bar{p} \tag{1}$$

After differentiating the end forces in the global coordinate system with respect to the end displacements, we can obtain the tangent stiffness matrix of the element,

$$k_e = \frac{\partial p}{\partial u} = a_r^T \frac{\partial \bar{p}}{\partial u} a_r \tag{2}$$

Souza demonstrated that it is possible to decouple the element formulation from the geometric transformation of the element response quantities as discussed in Uriz et al. [7]. Therefore the inelastic beam-column element with small deformation theory can represent the moderate to large deformation effects of inelastic buckling of a brace. For this purpose, a brace similar to the one that is shown in *Fig. 1a*, needs to be subdivided into at least 2 elements (see *Fig. 1b*). Fiber elements are used for the element refinement as shown in the same figure. The inelastic beam-column element that is shown in *Fig. 1b* accounts for interaction of axial force and bending moment along the brace. For this purpose a fiber model is used to model the cross section. *Figure 1c* shows an example of fiber discretization for a Hollow Square Section (HSS). The uniaxial hysteretic steel material is assigned to the fiber elements. This is the Menegotto-Pinto [9] material model with kinematic and isotropic hardening as shown in *Fig. 1d.* the material properties for this model should be based on the measured material properties of the actual steel brace to be modeled. By integrating this hysteretic material over the cross section of the brace with the use of fiber models we can take into consideration the interaction of axial load and bending. The integration points along the brace element are based on the Gauss-Lobatto quadratic rule with two integration points at the element ends. Note that the main assumption for this modeling approach is that plane sections remain plane after deformation and that the section shape is retained.

Uriz et al. [7] illustrated that two element subdivisions of a steel brace are sufficient to capture the section curvature along a steel brace. However, to model global buckling of a steel brace an initial camber is needed. The value of this camber ranges from 0.05% to 0.10% of the brace length. Okazaki et al. [10] concluded the same after utilizing the same modeling approach to simulate the seismic response of an eccentrically braced frame that was recently tested at the E-Defense laboratory in Japan. This test is briefly discussed in Section 3 of this paper.

Fracture of a steel brace is also incorporated in the numerical model. Fracture can initiate according to a rain-flow-counting rule. Linear strain accumulation is assumed for this purpose and a Coffin-Manson relationship in the logarithmic domain. For this purpose, a fatigue uniaxial material is used that wraps around the uniaxial hysteretic material shown in *Fig. 1d* and does not influence the stress-strain relationship of the parent material. In order to model inelastic buckling and fracture initiation of a steel brace a number of calibration studies have been conducted with past experimental data. Details about the calibration procedure are discussed in Okazaki et al. [10].

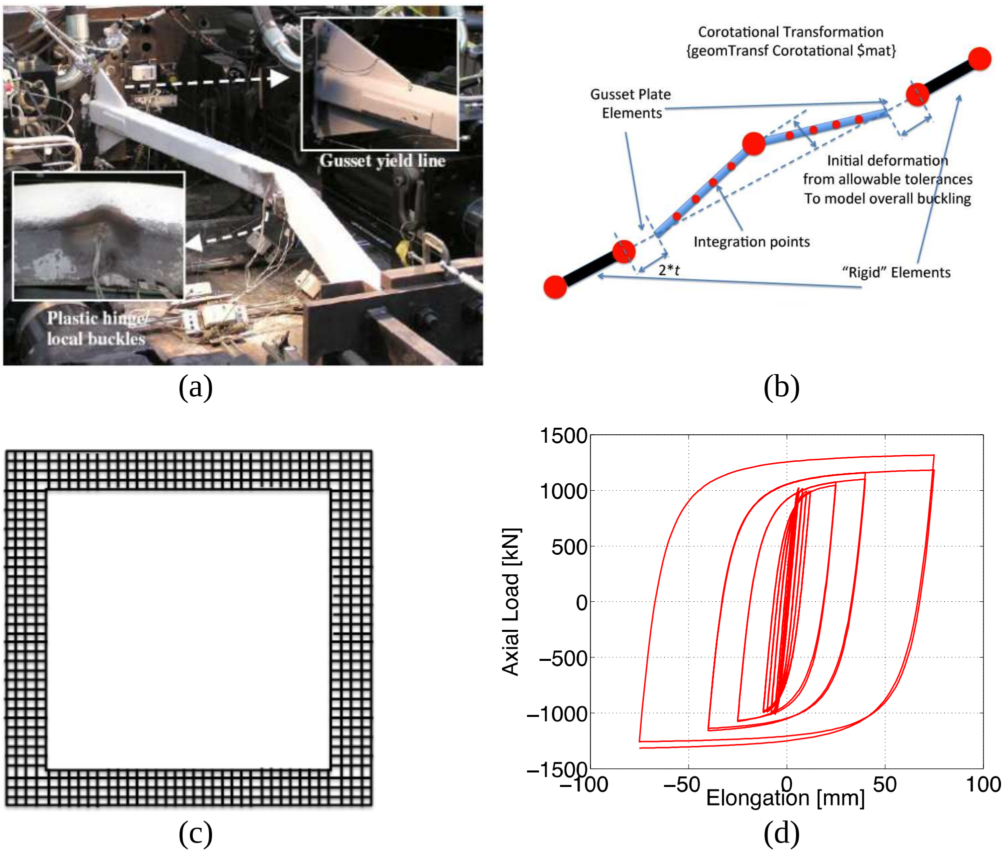
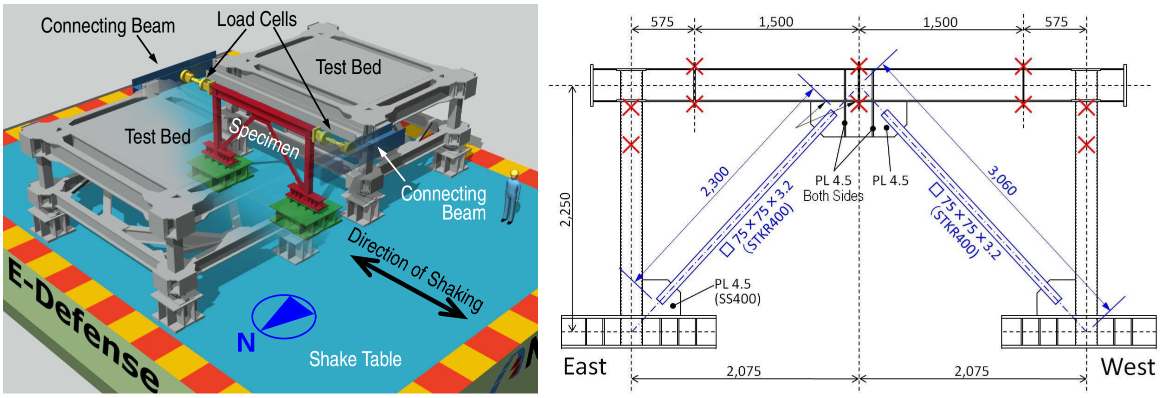


Fig. 1 Modeling of inelastic buckling of a steel brace

3. SHAKING TABLE TEST VALIDATION

In order to validate the capability of the numerical model discussed in Section 2 for predicting inelastic buckling, fracture and post-fracture behavior of steel concentrically braced frames a single story single span chevron-braced frame specimen was tested at the world's largest shaking table facility at E-Defense. *Figure 2a* shows the test setup on the shaking table. The test bed developed by Takeuchi et al. [11] was used to simulate the seismic masses acting on the test specimen with a pair of test beds, one at each side of the loading plane of the test specimen. The test beds were connected at the top of the specimens with a load cell. The test specimen shown in *Fig. 2b* was fabricated with tubular HSS75x3.2 steel braces. The elliptical clearance rule proposed by Lehman et al. [6] was utilized for sizing the gusset plates. More details about the test specimen are discussed in Okazaki et al. [10]. The shake table tests were conducted by introducing the East-West component of the JR Takatori motion seven times, with target amplification level increasing from 10, 12, 14, 28, 28 (second time), to 42 and finally 70%.



(a) Test setup on the shake table (b) Test specimen

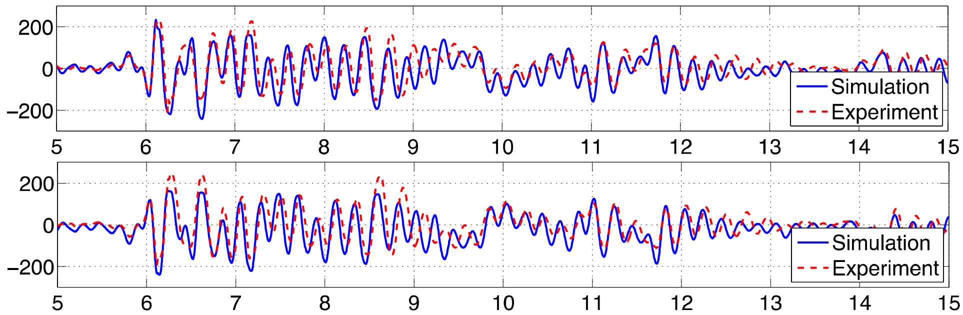
Fig. 2 Test specimen for shake table testing at E-Defense

The structure remained elastic during the 28% motion but some inelasticity occurred during the 42% motion. During the 70% motion the lateral stiffness of the test specimen suddenly reduced after experiencing two and one-half drift cycles of -0.015 to 0.01rad. During these drift cycles, the strength of the system dipped notably due to fracture of both steel braces. *Figure 3* illustrates the performance of the steel gusset plates and the tubular brace after the completion of the shaking table test. Both steel braces fractured near mid-length of the member. The gusset plates near the column base and middle of the beam folded along the elliptical clearance. This is in agreement with findings discussed in [6].

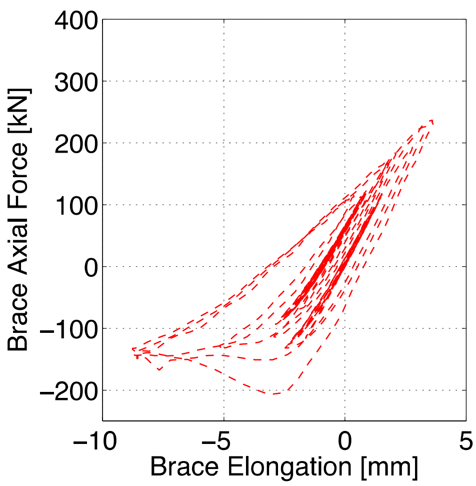
(a) (b) (c)

Fig. 3 Brace connections and fracture at midlength of the brace after the completion of the shaking table test

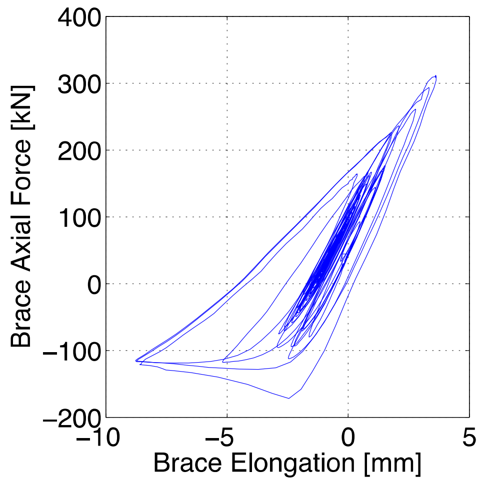
The test specimen was modeled in the OpenSees [12] simulation platform using a centerline fiber model. Inertia forces were modeled with a leaning column. The corotational transformation was utilized to model small P-Delta effects of the steel components. Steel braces were modeled based on the guidelines discussed in Section 2 of this paper. Three percent mass proportional damping was assumed at the first mode period of the test specimen. The predominant period of this specimen was 0.20sec. *Figure 4a* shows the axial force histories for the two braces based on numerical simulations for the 42% JR Takatori motion. In the same figure we have superimposed the same quantities as measured during the shake table test. The comparison indicates that the force distribution for both braces is predicted fairly accurately when global and inelastic buckling of the steel braces occurs. The same can be seen from *Fig. 4b* and *4c* that shows a comparison between experimental and simulated response of the brace axial force versus brace elongation for the 42% JR Takatori motion.



(a) Brace force history during 42% of JR Takatori motion



(b) Experimental data (42% JR Takatori)



(c) Simulated data (42% JR Takatori)

Fig. 4 Comparison of experimental versus analytical results during 42% of JR Takatori motion

During the 70% JR Takatori motion the two braces fractured. This resulted to a loss of lateral system of the test specimen. The numerical model discussed in Section 2 is able to capture the stiffness deterioration due to brace fracture. *Figure 5* shows the normalized base shear history of the test specimen during the 70% JR Takatori motion. In the same figure we have superimposed the simulated response of the test specimen, which was modeled in OpenSees [12]. This figure demonstrates that the numerical model of the test specimen is able to predict its post-fracture behavior after the occurrence of brace fracture. The phase difference between simulated and experimental results after the first 9 seconds of the ground motion is attributed to the fact that the predicted fracture time of the second brace occurred 2 seconds earlier than the time that occurred during the experiment.

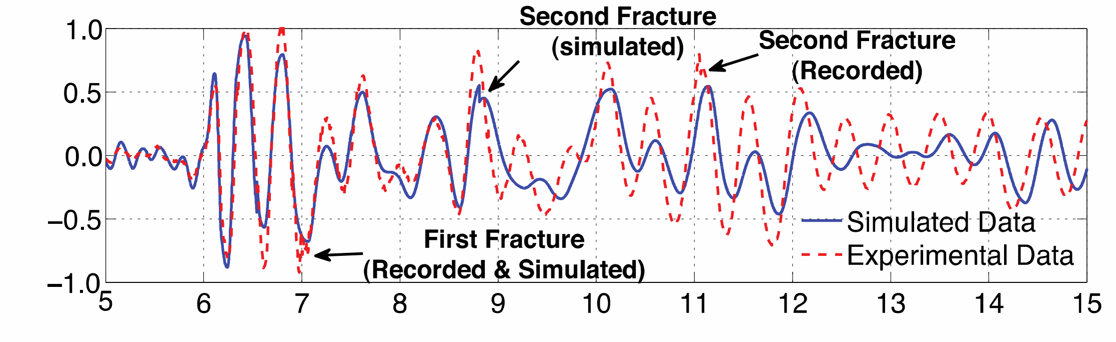


Fig. 5 Normalized base shear history of the test specimen during 70% JR Takatori motion

4. CONCLUSIONS

This paper demonstrates the capabilities of a numerical model that is able to simulate inelastic buckling, fracture and post-fracture behavior of steel braces. The model consists of a force-based beam-column element with distributed inelasticity and a fiber discretization of the steel brace cross section to account for the interaction between axial force and bending moment. The same numerical model incorporates fracture that can initiate according to a rain flow counting rule. The parameters of the fracture model are based on calibrations of the simulated response of single braces modeled in the OpenSees analysis platform against previous experimental data. The modeling approach discussed in this paper was validated with a recent shake table test of a single story chevron brace configuration with hollow square section braces. The story drift, story shear, brace elongation, and force distribution recorded from the shake table tests was traced fairly accurately by the numerical model. The model was able to predict the occurrence of brace fracture and subsequent behavior of the frame.

5. ACKNOWLEDGMENT

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ΑΡΙΘΜΗΤΙΚΗ ΠΡΟΣΟΜΟΙΩΣΗ ΤΗΣ ΜΕΤΑΛΥΓΙΣΜΙΚΗΣ ΣΥΜΠΕΡΙΦΟΡΑΣ ΚΑΙ ΘΡΑΥΣΗΣ ΜΕΤΑΛΛΙΚΩΝ ΠΛΑΙΣΙΩΝ ΜΕ ΣΥΝΔΕΣΜΟΥΣ ΔΥΣΚΑΜΨΙΑΣ

Δρ. Δημήτριος Γ. Λιγνός
Επίκουρος Καθηγητής
Πανεπιστήμιο McGill
Μόντρεαλ, Καναδάς, H3A 2K6
Ηλεκτρονική διεύθυνση: dimitrios.lignos@mcgill.ca

Δρ. Taichiro Okazaki
Αναπληρωτής Καθηγητής
Πανεπιστήμιο Hokkaido, Ιαπωνία
Sapporo, Hokkaido, Ιαπωνία, 060-8628
Ηλεκτρονική διεύθυνση: tokazaki@eng.hokudai.ac.jp

Δρ. Tsuyoshi Hikino
Διευθυντής
Τμήμα Μεταλλικών Κατασκευών και Ερευνών, Nippon Steel, Ltd.
Τόκυο, Ιαπωνία
E-mail: hikino.tsuyoshi@nsc-eng.co.jp

Δρ. Kouichi Kajiwara
Διευθυντής
Αντισεισμικό κέντρο Ερευνών Hyogo, Εθνικό Ινστιτούτο Ερευνών (E-Defense)
Miki, Ιαπωνία
E-mail: kaji@bosai.go.jp

Δρ. Masayoshi Nakashima
Καθηγητής
Πανεπιστήμιο Κιότο
Κιότο, Ιαπωνία
E-mail: nakashima@archi.kyoto-u.ac.jp

Το άρθρο αυτό περιγράφει ένα μη γραμμικό προσομοίωμα για την πρόβλεψη της μεταλυγισμικής συμπεριφοράς και ολιγοκυκλικής θραύσης μεταλλικών συνδέσμων δυσκαμψίας υπό ανακυκλιζόμενη φόρτιση. Οι σύνδεσμοι δυσκαμψίας προσομοιώνονται με τη μέθοδο των πεπερασμένων στοιχείων στα οποία η αριθμητική ολοκλήρωση κατά μήκος των στοιχείων βασίζεται στον κανόνα Gauss-Lobatto. Η προσομοίωση της θραύσης στο υστερητικό προσομοίωμα λαμβάνεται υπόψη με βάση τον κανόνα μέτρησης rain-flow. Το παραπάνω προσομοίωμα συγκρίνεται με πειραματικά δεδομένα μεταλλικών συνδέσμων τετραγωνικής διατομής για την ποσοτικοποίηση κρίσιμων παραμέτρων που επηρρεάζουν την ολιγοκυκλική θραύση μεταλλικών συνδέσμων. Εξάλλου, το εν λόγω υστερητικό προσομοίωμα για μεταλλικά πλαίσια με συνδέσμους δυσκαμψίας αξιολογήθηκε με χρήση πειραμάτων σε σεισμική τράπεζα.