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FINITE ELEMENT SIMULATION OF A STEEL BRACED FRAME STRUCTURE DAMAGED BY THE 2011 TOHOKU EARTHQUAKE

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Abstract. *During post-earthquake reconnaissance after the March 11, 2011 Tohoku Earthquake, the writers examined a damaged parking ramp structure. The structure was a two-story steel structure with a 52.5 by 48.5-m floor plan which relied on chevron braces for lateral load resistance. The earthquake damaged nearly all first-story braces in their connection to the beam. The majority of bracing connections had fractured in the east-west frame while many bracing connections were distorted, but none of them fractured, in the north-south frame. The cause of the observed damage was examined by detailed finite element simulations. Planar models of the structure, one extracted from the east-west frame and another extracted from the north-south frame, were subjected to static cyclic loading and to a ground motion record obtained within 800 meters from the structure. The simulation captured the main features of the observed damage. The results suggest that the bracing connections in the east-west frame failed under a compressive force much smaller than the buckling load of the brace. Although they failed similarly, the bracing connections in the north-south frame were almost strong enough to develop the compressive strength of the brace. This difference may explain the reason why the damage differed in severity between the two loading directions. While fracture was not explicitly modeled in the simulation, the computed plastic strain indicated that the observed fracture in the bracing connections was due to severe cyclic deformation produced while the brace was in compression.*

1 INTRODUCTION

Steel concentrically braced frames have been used widely in high-seismic regions due to their efficiency in meeting lateral-load resisting requirements. Based on extensive research since the 1970’s, it is well known that the cyclic loading performance of steel braces depend on their slenderness ratio and on the width-to-thickness ratio of their cross sectional elements, and that adequate detailing of the bracing connection is critical to avoid premature fracture at the end of the brace. These findings have been reflected in design codes around the world [1, 2]. In terms of analysis capabilities, researchers have proposed methodologies to predict the occurrence of fracture from cumulative damage and capture the structural response following brace fracture [3, 4]. Despite the substantial advances in research, many steel braced frames performed poorly in recent earthquakes [5, 6]. This paper examines a parking ramp structure damaged by the March 11, 2011 Tohoku earthquake. The member proportion and connection details of this structure represented common practice in Japan. Results from detailed finite element simulation are used to discuss the possible causes of the damage and means to improve the seismic performance of steel braced frames.

2 THE DAMAGED STRUCTURE

During post-earthquake reconnaissance after the 2011 Tohoku Earthquake, the writers examined a damaged parking ramp structure. The structure was a two-story steel structure with a 52.5 by 48.5-m, 9 by 7-span floor plan, comprising square-hollow structural section (HSS) columns, I-section beams, and round-HSS braces, all of galvanized steel. The beams were simply connected to the columns, and single-lap bolted connections were used for the bracing connections. Although not confirmed, it is believed that the base of the column base (which was covered in asphalt and hence not visible) was connected to a foundation through a base plate and anchor rods. Fig. 1 shows the primary dimensions and sections in the east-west and north-south braced bays. The slenderness ratio of the braces based on the diagonal, face-to-face distance between the beam and column, was 54 in the east-west bay and 69 in the north-south bay. The residual story drift ratio was 0.01 rad. in the east-west direction but minimal in the north-south direction. As indicated in Fig. 2, nearly all first-story braces were damaged in their connection to the beam. The majority of bracing connections had fractured in the east-west frames while many bracing connections were distorted, but none of them fractured, in the north-south frames. The damage is extremely concerning because this structure relied solely on the braces for lateral-load resistance. The structure was sealed from entry when the writers visited, and demolished shortly afterwards.

The use of single-lap bolted connections in the bracing connections seemed to be a primary cause of the extensive damage. The difference in out-of-plane flexibility was believed to have caused the fracture to occur only in the upper-end gusset plate and not in the lower-end gusset plate. Fig. 3 shows bracing connections at different stages of failure, from (a) out-of-plane

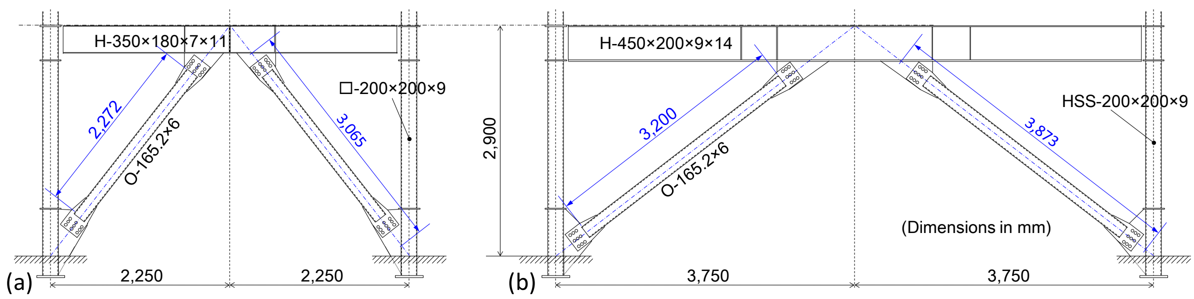

Figure 1. Primary dimensions of braced bay in: (a) east-west frame; and (b) north-south frame.

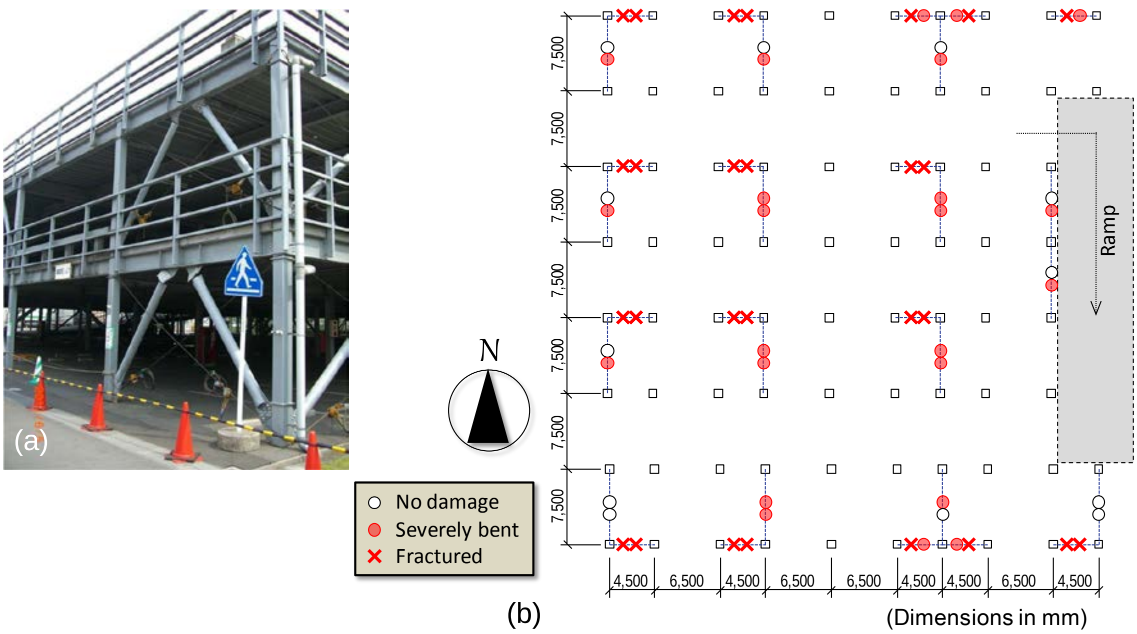

Figure 2. Damaged parking ramp: (a) photo; and (b) floor plan indicating damage location in the first story.

Figure 3. Connection damage: (a)bent out of plane; (b) fracture initiation; and (c) complete fracture.

distortion, (b) fracture initiation near the termination of the welds connecting the brace to the gusset plate, to (c) complete fracture. Although the cause of damage seemed to be straight-forward, a number of questions remained after the field study. Most notably, why was the damage clearly more severe in the east-west frames than in the north-south frames, and what was the actual lateral strength of the structure?

3 FINITE EEMENT MODELS

In order to seek answers to the questions, detailed numerical simulations were conducted using the finite element analysis software ADINA [7]. Separate planar, single-bay chevron braced frame models were constructed to represent the east-west frame and north-south frame. The columns were pinned at the base and the beams were simply connected to the columns. As shown in Fig. 4, the planar models were laterally braced at locations where orthogonal beams framed into the column or beam. The finite element models used isoparametric solid elements at regions where plastic deformation was expected. The regions expected to remain elastic were modeled by a beam element with 7 degrees of freedom at each end node. Two solid elements were used across the material thickness to capture local deformation and local buckling. At the bracing connection, the spliced material was merged at the faying surface and the bolts were not explicitly modeled. Material nonlinearilty was modeled based on the von Mises yield criteria and combined hardening rule assigning a 1:19 ratio between kinemat-

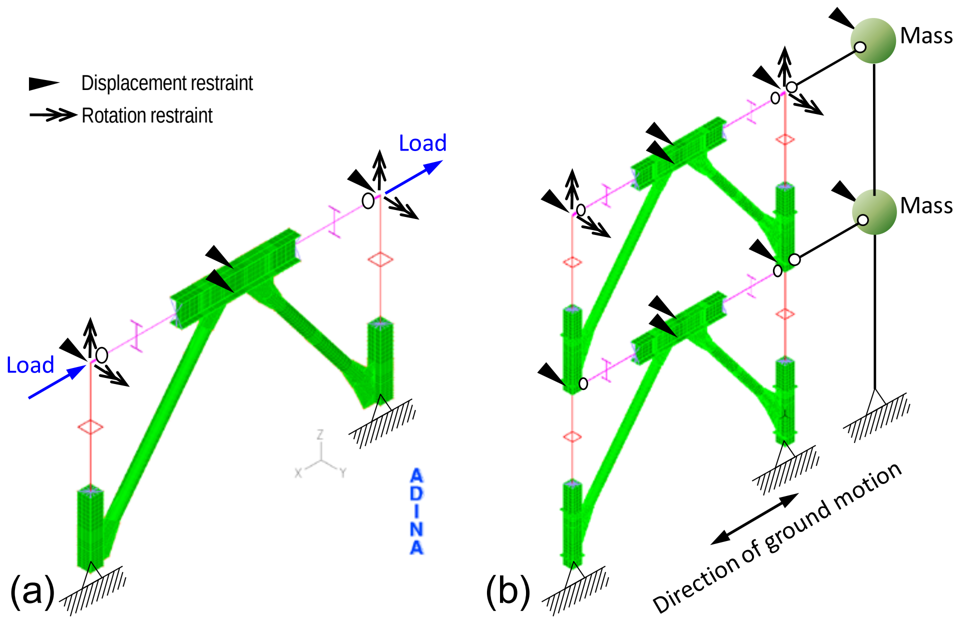

Figure 4. Finite element models: (a) statically loaded, single-story; and (b) dynamically loaded, two-story model.

ic and isotropic hardening. Geometric nonlinearity was modeled using a large displacement-small strain formulation. Unless stated otherwise, the brace was assigned a yield strength of 295 N/mm^2 and all other material was assigned a yield strength of 235 N/mm^2. The material strength values represented the specified minimum strength of structural steel in Japan.

The single-story model shown in Fig. 4a was subjected to static cyclic loading. Load was applied in displacement control, equal at both ends of the beam, increasing the story drift ratio from elastic cycles up to ±0.018 rad., and repeating each cycle twice.

The two-story model shown in Fig. 4b was connected to an elastic gravity column and concentrated mass that represented the system mass divided by the number of braced bays in the loading direction (16 in the east-west direction, 15 in the north-south direction). This model was subjected to a ground motion record obtained at a station within 800 meters from the structure. Raleigh damping was employed by assigning a damping ratio of 0.02 to the first two vibration modes.

4 ANSLYSIS RESULTS

4.1 Static Analysis

Fig. 5 plots the relationship between story shear and story drift ratio obtained from the static, cyclic loading analysis. Fig. 6 plots the relationship between axial force and elongation of one of the two braces (the brace that is first compressed), obtained from the same analysis. Each figure shows results obtained from the single-story, east-west and north-south model. Fig. 6 indicates the tensile yield strength 885 kN, and maximum compressive strength 689 and 621, respectively, for the east-west and north-south frame, computed according to the Japanese provisions [1] and based on the yield strength 295 N/mm^2.

Fig. 5 shows that both models exceeded the elastic limit at a story drift ratio of 0.003 rad., and subsequently experienced a drop in strength. The response gradually stabilized as the deformation amplitude increased, although, as described later, the apparently stable response relied on unrealistic deformation in the bracing connections. It is also noted that the stiffness of the system degraded to 30% of the initial value as the deformation increased. Fig. 6 shows

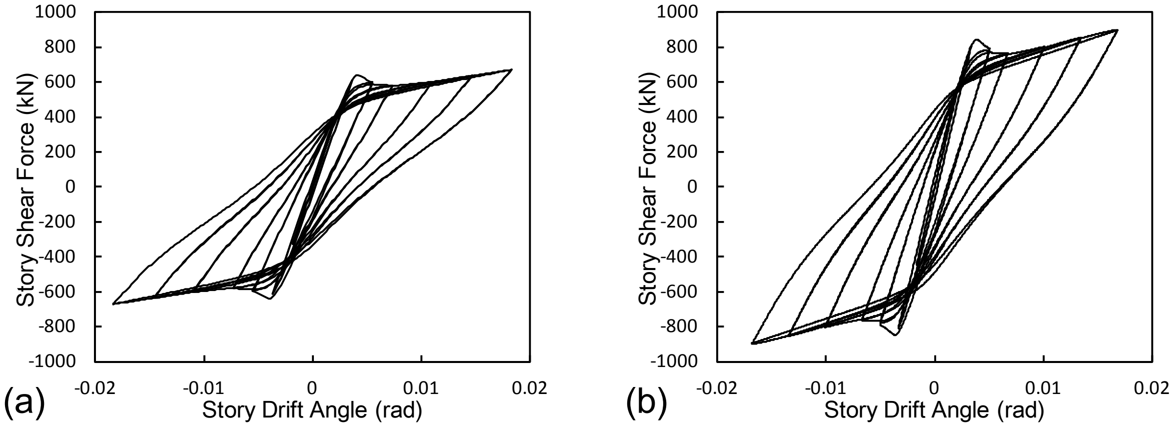

Figure 5. Static, cyclic loading response: (a) east-west model; and (b) north-south model.

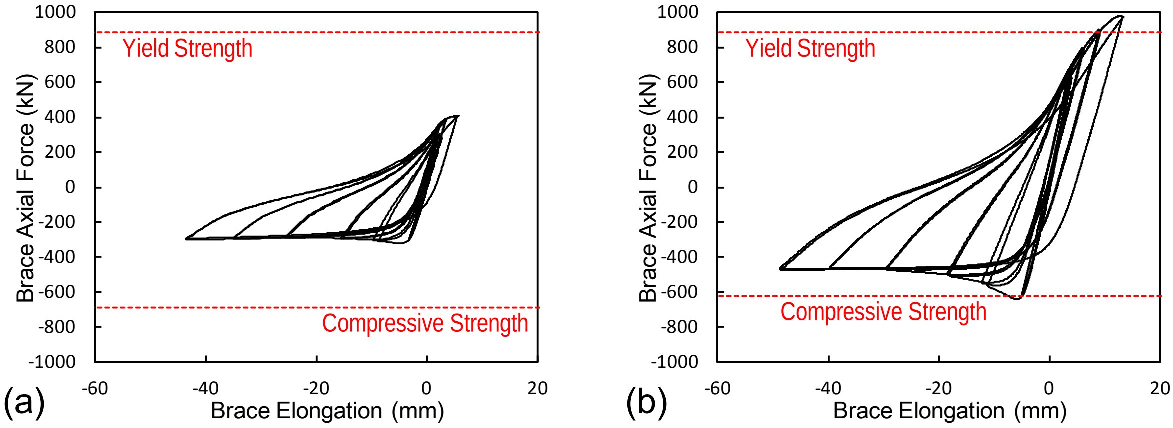

Figure 6. Hysteresis of brace in: (a) east-west model; and (b) north-south model.

that deformation of the brace progressed primarily in the shortening direction. This is a common feature of braces placed in a chevron arrangement: Due to the force unbalance following buckling of the compressed brace, the beam deflects downwards, and therefore, further elongation of the brace is prevented while contraction is promoted. A striking difference is noted between the strength of the two braces. While the north-south brace developed the yield strength in tension and the buckling strength in compression, the east-west brace developed less than half the design strength in tension or compression.

Fig. 7 shows the deformation of the model at the end of the analysis where the model was deformed to a story drift of -0.018 rad. In both models, deformation concentrated in the upper-end bracing connection. The beam is deflected downwards, as described above, due to the force unbalance between the two braces. In fact, throughout the analysis, all other members remained elastic while the upper-end bracing connection folded and straightened, and the beam deflected downwards twice within each loading cycle. The deformed shape suggests that, due to the intrinsic eccentricity in the single-lap connection and lack of edge support, the gusset plate deformed out of plane when compressed. The computed deformation coincides with the damage of the original structure described previously.

Out-of-plane deformation of the gusset plate started at a compressive force merely half of the buckling load of the brace in the east-west model, and near the buckling load of the brace in the north-south model. Analysis of the north-south model was repeated by reducing the yield strength of the brace from 295 to 235 N/mm^2. In this analysis, the brace buckled prior to failure of the gusset plate, and subsequently, inelastic action concentrated in the braces rather

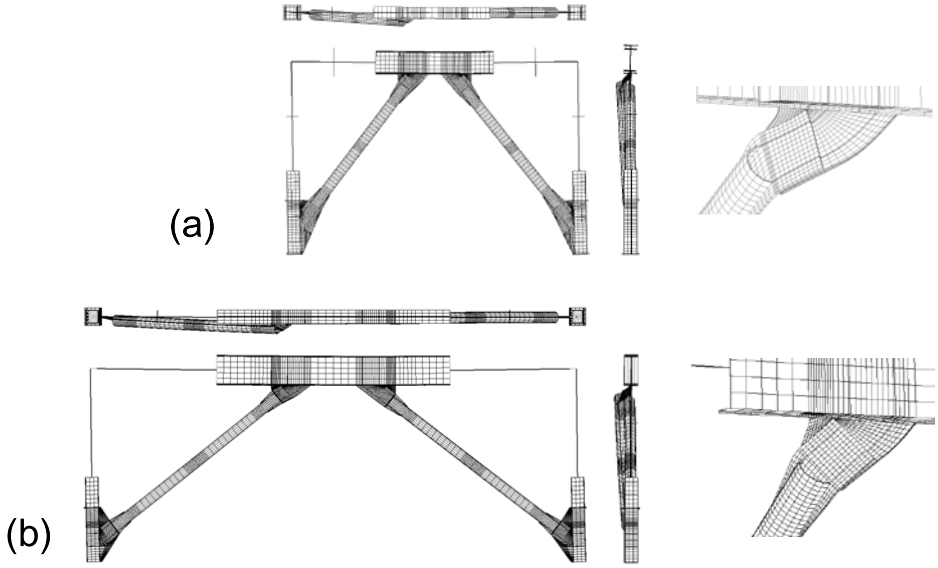

Figure 7. Deformed model at end of analysis: (a) east-west model; and (b) north-south model.

than in the gusset plates. Consequently, in compression, the gusset plate was substantially weaker than the brace in the east-west model, but the gusset plate and brace were similar in strength in the north-south model. While fracture was not explicitly modeled in the simulation, the computed plastic strain indicated that the observed fracture in the bracing connection (see Fig. 3) was due to severe cyclic deformation produced while the brace was in compression.

4.2 Dynamic Analysis

The same modeling scheme was used to construct a two-story model representing the east-west frame (see Fig. 4b). Concentrated floor mass was assigned to a continuous gravity column whose elastic properties equaled that of three columns independent from the lateral-load resisting system in the east-west direction. Although not shown in this paper, the simulated damage was similar to that obtained in the static analysis, and resembled the actual damage.

Fig. 8 shows the relationship between axial force and elongation obtained for one brace each from the first and second stories. As in Fig. 6, the design strength in tension and compression are indicated in the figure. Despite the presence of an elastic gravity column, damage concentrated in the first-story braces while the second-story braces remained elastic. This is contrary to the actual damage where the same failure in the bracing connection was found in a

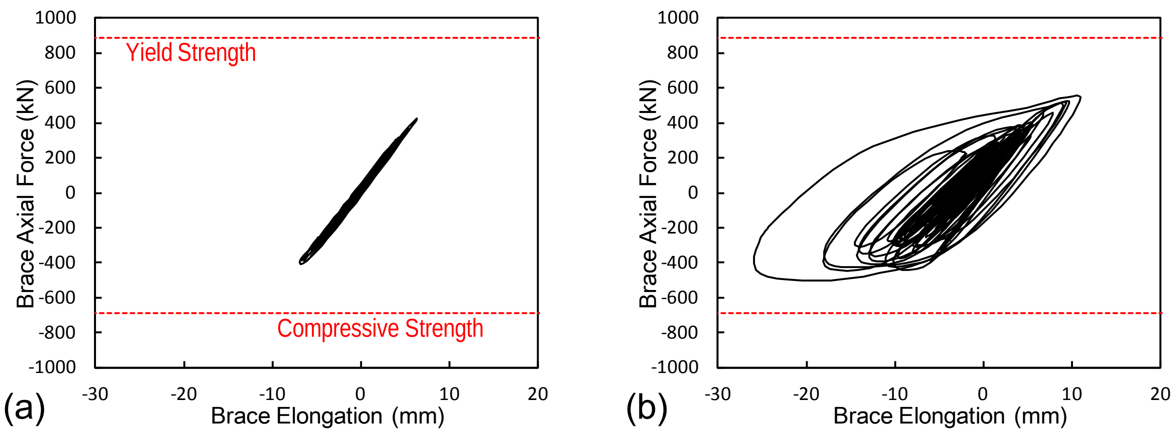

Figure 8. Hysteresis of braces in: (a) second story; and (b) first story.

large number of second-story braces as well as first-story braces. This discrepancy may be due to the fact that, while the first-story stiffness must have degraded gradually in the original structure, buckling of a brace caused a drastic loss of stiffness in the single-bay model. Fig. 8b indicates that the braces failed to develop their strength expected in design. The axial deformation was smaller than the imposed deformation in the static analysis. The residual deformation was minimal at the end of analysis. Work is ongoing to improve the model such that the results better match the observed damage.

5 CONCLUSIONS

Detailed finite element simulations were conducted to examine a parking ramp structure that was damaged by the 2011 Tohoku earthquake. The following are the primary findings obtained from the simulation results.

- The finite element analyses were able to reproduce the damage observed in the original structure. The results suggest that fracture of the gusset plate was due to the deformation concentration when the brace was in compression.
- The finite element analyses indicated that the gusset plate was significantly weaker in the east-west frame than in the north-south frame. This result may explain the substantial difference in damage between the two frames.

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