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Analytical Simulations for Shaking Table Tests of a Full Scale Buckling Restrained Braced Frame

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Abstract

The seismic response tests of a full-scale five-story passively-controlled steel building have been conducted on the E-Defense shaking table in Japan in March 2009. Before the tests, a blind prediction contest was held to allow researchers and practitioners from all over the world to construct numerical model and predict the dynamic responses of the building frame equipped with buckling restrained braces or viscous dampers. This paper presents the response predictions made before the tests of the buckling restrained braced frame (BRBF). A three-dimensional shell finite element subassembly model was constructed to investigate the stiffness of the buckling-restrained brace end joint. The dynamic test results of the sample steel dampers provided by the organizer were used to calibrate the BRB strength in the numerical model. The details of the numerical model for the composite beam, hollow structural section column, and the beam-to-column panel zone are described. Base on the numerical and test analyses, effective numerical models are presented and the recommended nonlinear modeling techniques for BRBF are provided.

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Keywords: Nonlinear analysis, BRBF

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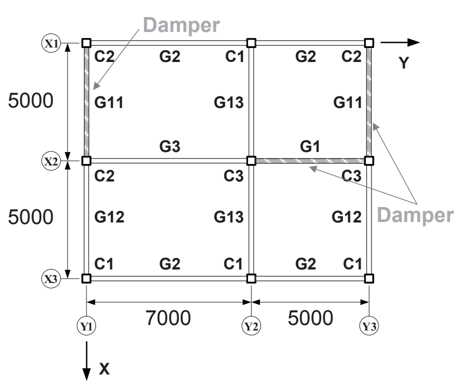
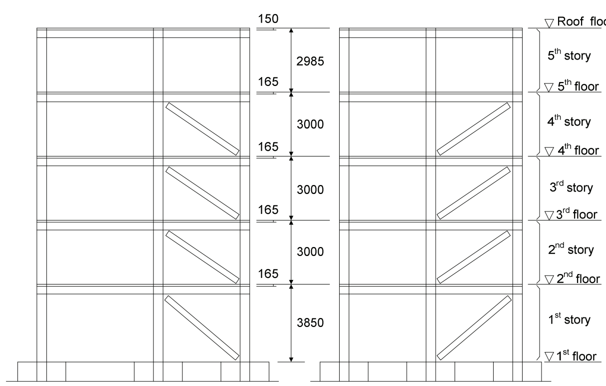
1. Introduction

In March 2009, the world's largest three-directional earthquake simulation table, E-Defense shaking table, was utilized for experimental tests of a full-scale five-story passively-controlled steel building. Before the tests were executed, a blind prediction contest (Hikino et al. 2010) was held. The group of researchers from the Taiwan National Center for Research on Earthquake Engineering (NCREE) participated in the blind contest. The NCREE team used the nonlinear structural analysis programs OpenSees (McKenna 1997) (http://opensees.berkeley.edu) and PISA3D (Lin et al. 2009) to conduct the dynamic analyses of the building frame equipped with steel dampers (buckling restrained braces) and viscous dampers, respectively. This NCREE team was awarded the second place in the three-dimensional (3D) steel damper category, the third place in the two-dimensional (2D) steel damper category, and the third place in the 3D viscous damper category. This paper describes the detailed OpenSees modeling techniques for buckling restrained brace frame (BRBF).

Figure 1: Five-story buckling restrained braced frame.



(a) Floor framing plan



(b) Frame elevations

Figure 2: Floor framing plan and elevations (mm).

2. Test Of A Five-Story BRBF

2.1 Introduction Of The Test Frame

The photo of the full-scale five-story BRBF is shown in Figure 1. The contestants were required to predict the key maximum experimental responses of the specimen under the three-directional shaking. Before the five-story frame tests, cyclic loading test results of steel damper specimens under various loading frequencies were also provided by the organizer to allow the contestants to gain a better insight of individual damper member.

The floor framing plan and the elevations are shown in Figures 2(a) and 2(b), respectively. The longitudinal and transverse directions of the building are defined as the Y and X directions, respectively. The structural configuration consists of two bays of 7 m and 5 m in the Y direction and two bays of 5 m each in the X direction. The story height is 2.985 m for the 5th story, 3.85 m for the first story, and 3 m for the second, third, and fourth stories. Each thickness of the concrete slab is 165 mm except the roof floor which is 150 mm. The 75 mm deep metal deck was spanning in the Y direction. Wide flange sections are used for beams, and square box sections for columns. The steel material is SN490B for the frame beams and BCR295 for the columns. As shown in Figure 2, the BRBs were equipped in the first, second, third, and fourth stories in the X2, Y1, and Y3 frames.

2.2 Blind Contest Rules

The BRBF specimen was subjected to three-directional ground motions recorded during the 1995 Kobe earthquake (Nakashima et al. 1998) at the Takatori train station. The test consisted of repeated applications of the records with progressively increasing scale factors of 0.15, 0.4, 0.5, 0.7, and 1.0. Participants were advised to carry out time history analysis for these consecutive motions in order to evaluate the specimen responses. The contest targets are 0.15 and 1.0. The required submission of the predicted responses included the following (Hikino et al. 2010).

Absolute maximum values of relative displacement from base, absolute maximum acceleration at each floor; absolute maximum values of story shear, story drift angle; maximum and minimum values of axial deformations and axial forces of the dampers in the first and fourth stories; absolute maximum strain at a specified point in an elastic region.

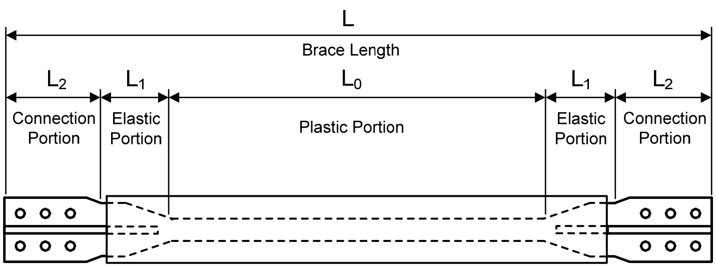


Figure 3: Buckling restrained brace.

3. Numerical Model for Prediction

In the numerical model for prediction, the frame mass is considered as a lump mass located at the top of the slab of each floor. However, the location of the mass is eccentric according to the drawings provided from the organizer. Rigid floor diaphragm is assumed. The geometric nonlinearity is not considered because the story drift is estimated to be small. The Newmark method of constant average acceleration scheme (β=1/4) is used for the time integration. Rayleigh damping is adopted, and the damping ratios are assumed to be 2% for the first and second modes. The modeling techniques of the prediction model are described below.

3.1 Numerical model for BRB

In the OpenSees numerical model, each BRB member was simulated using a truss element with the Giuffre-Menegotto-Pinto steel material.

3.1.1 BRB stiffness

The BRB is primarily composed of a ductile steel core, a buckling-restraint mechanism, and an unbonding mechanism. The steel core consists of three portions (see Figure 3): a plastic portion, an elastic portion, and a connection portion. Only the steel core is supposed to provide stiffness to the BRB. The effective stiffness of a BRB was computed using the following equation (Tsai and Hsiao 2008):

$$K_{eff} = \frac{1}{\frac{1}{K_p} + \frac{1}{K_e} + \frac{1}{K_c}} \tag{1}$$

Where K_{eff} is the BRB effective stiffness. K_p, K_e, and K_c are the stiffness of the plastic, elastic, and connection portions, respectively. However, K_c is unclear because of the existence of the gusset plates in the BRB end joints. Thus, a three-dimensional shell finite-element (FE) subassembly model as shown in Figure 4(a) was constructed using ABAQUS (2006) to calibrate K_c of the BRB. Only steel part of the BRB frame was built. This numerical subassembly model is a two-story frame for computing BRB stiffness because the BRB length of first story is different from that of other stories (see Figure 2). This FE model was constructed using the four-node, quadrilateral, stress/displacement shell elements with the reduced integration and a large-strain formulation (S4R). The bilinear steel material model was adopted for a total of 37601 shell elements. As shown in Figure 4a, the column bases are fixed and out-of-plane constraints were applied to the beam-to-column connections. The lateral displacement was applied to the center of the third-floor beam. K_c can be computed from the connection-portion deformation (Figure 4(b)) and the BRB axial force. Thus, K_{eff} computed using Equation (1) was incorporated into the OpenSees BRB model.

Full-scale damper tests were conducted at Tokyo Institute of Technology (Ooki et al. 2009), and the test results (static and dynamic) were distributed to the participants of the blind test. The tests combined the static loading and four kinds of dynamic loading frequency (0.2, 0.5, 1, 2 Hz) with three kinds of BRB stroke (12, 24, 36 mm). As shown in Figure 5, the static and dynamic loading test results are remarkably different. For considering the velocity dependency of the BRB, the parameters of the BRB numerical model were specified based on the dynamic loading test results. The test results of 2 Hz loading frequency was used because the vibration period of first mode is 0.5 s in the numerical frame model. Figure 5 shows that the numerical BRB model could well simulate the dynamic loading test results under different BRB strokes by using the same set of numerical parameters.

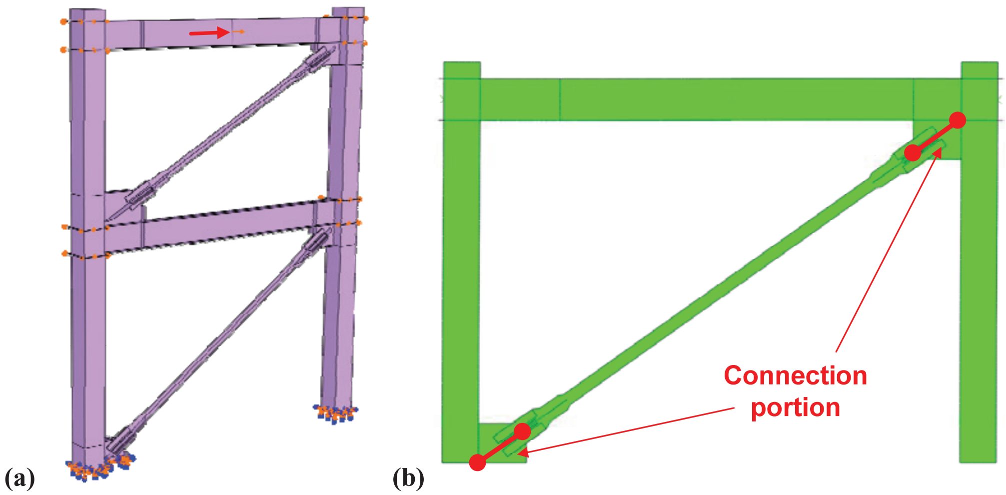
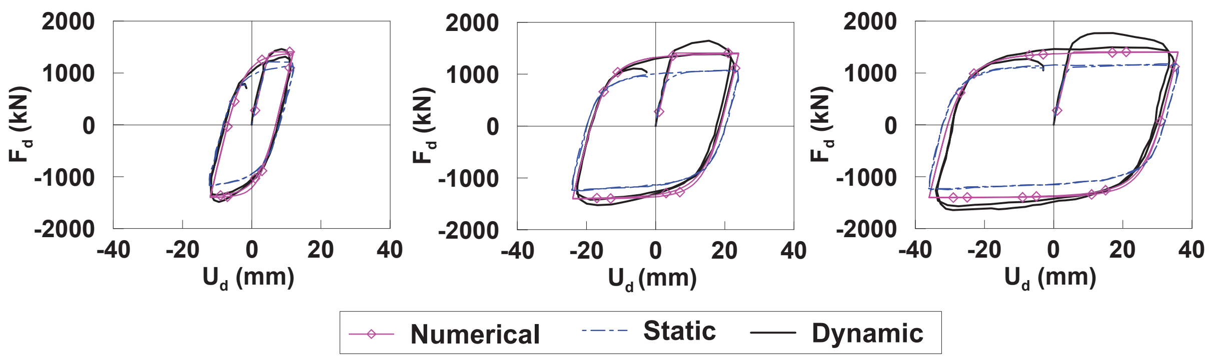


Figure 4: 3D shell model for determining BRB stiffness.

3.1.2 BRB strength



Static loading = 0.05%/s, Dynamic loading = 2 Hz
Figure 5: Cyclic numerical and test results of steel dampers.

3.2 Numerical model for composite beam

The frame beam member is considered as the steel and concrete full-composite beam in the numerical model. The nonlinear beam-column element in OpenSees is adopted to represent the composite floor beam. This element is a force-based distributed plasticity element. (Neuenhofer and Filippou 1997). The fiber section is adopted to define the force-deformation response at a cross section of the beam-column element.

As shown in Figure 6, the influence of the metal deck orientation on the composite beam properties has been taken into account. Figure 6(a) and (b) illustrate the fiber sections of the composite beams along the Y and X directions, respectively. The effective concrete slab width (bE) was defined base on the previous research about the 2007 blind analysis contest (Yu et al. 2010), that is, 300 mm when the simulation result as shown in Figure 7 was obtained. The concrete slabs of the beams along the Y and X directions consist of 50 and 25 compression-only fibers, respectively. The steel beam consists of 40 fibers using the steel yield strength provided by the organizers for the kinematic hardening steel material. The stress-strain relationship of the concrete provided by the organizer was used for the concrete fibers. The concrete material follows the recommendations provided by Kent-Scott-Park and the unloading/reloading responses are according to the work of Karsan and Jirsa (1969). Five integration points along the fiber-beam-column element were chosen to integrate the element responses.

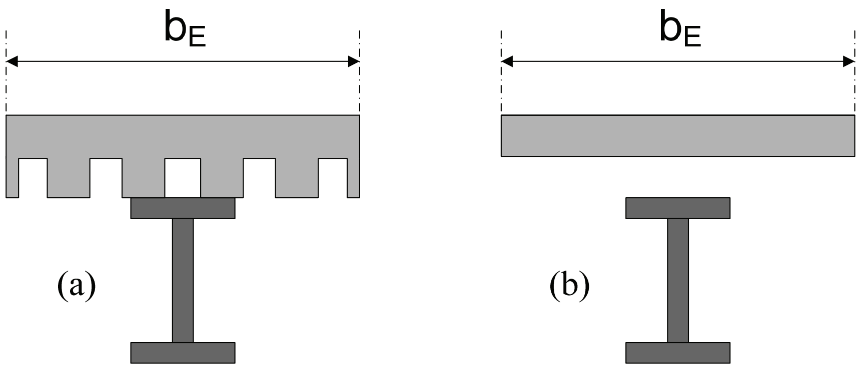


Figure 6: The cross section of composite beam along (a) the Y direction and (b) the X direction.

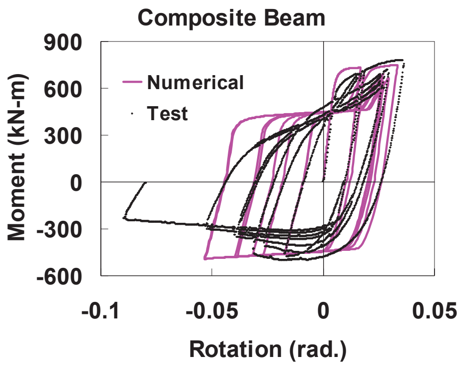


Figure 7: Composite beam response simulation.

3.3 Numerical models for column and beam-to-column connection

Each column member in the numerical model adopts nonlinear beam-column element with a fiber section consisting of 40 fibers. The beam-to-column connection numerical model does not take the shear deformation of the panel zone into account. That is, the beam and column properties were computed using center line to center line dimensions.

4 Key Prediction Results and Discussion

The numerical natural periods of the three lowest modes are 0.5, 0.481, and 0.343 seconds. The 3D nonlinear acceleration time history analysis was performed by lining up five ground motion records (0.15, 0.4, 0.5, 0.7, and 1.0 Takatori records). Figures 8a to 8d show the absolute maximum structural responses under the target excitations (0.15 and 1.0 Takatori). All the prediction results are very close to the responses observed in the test (Kasai et al. 2010). Figure 8d shows that the maximum story drift is less than 1% radian. The maximum and minimum values of axial deformations and axial forces of the dampers in the first and fourth stories under the target excitations are shown in Figure 9. Under the elastoplastic stage (1.0 Takatori), both prediction damper deformations and forces agree with the test responses. Nevertheless, damper responses under the elastic stage (0.15 Takatori) show some difference between prediction and test. This suggests that the damper strength is well predicted but the damper stiffness is not. Further studies are necessary.

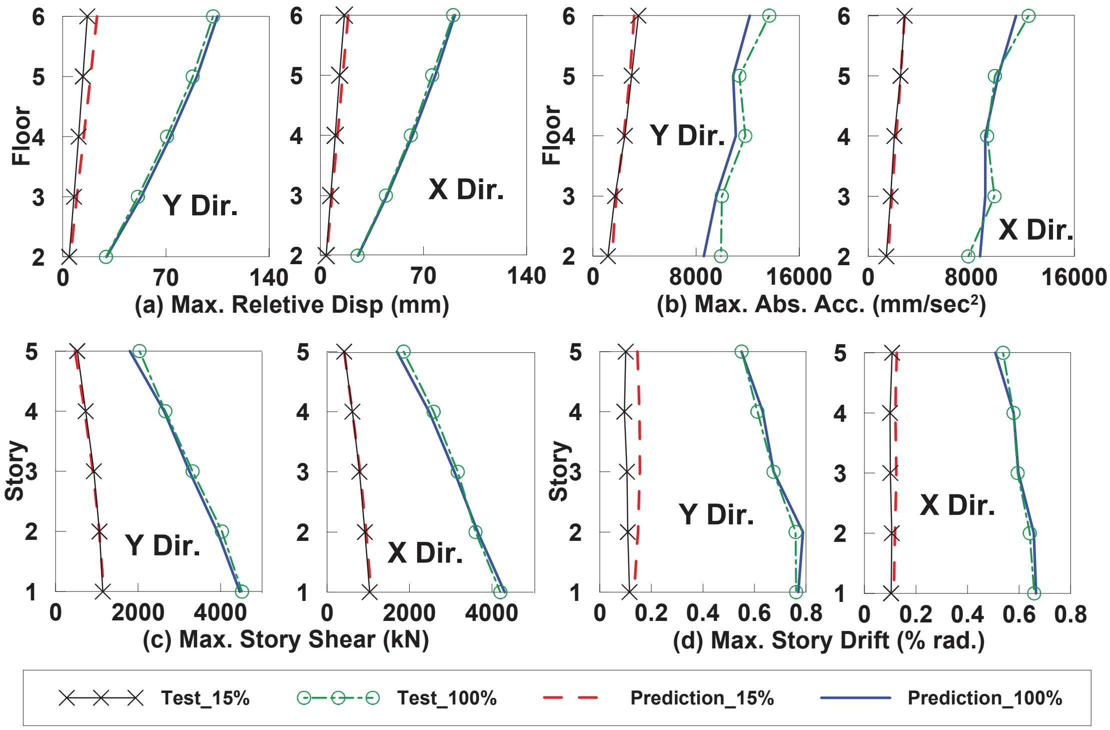


Figure 8: Absolute maximum responses of test and prediction.

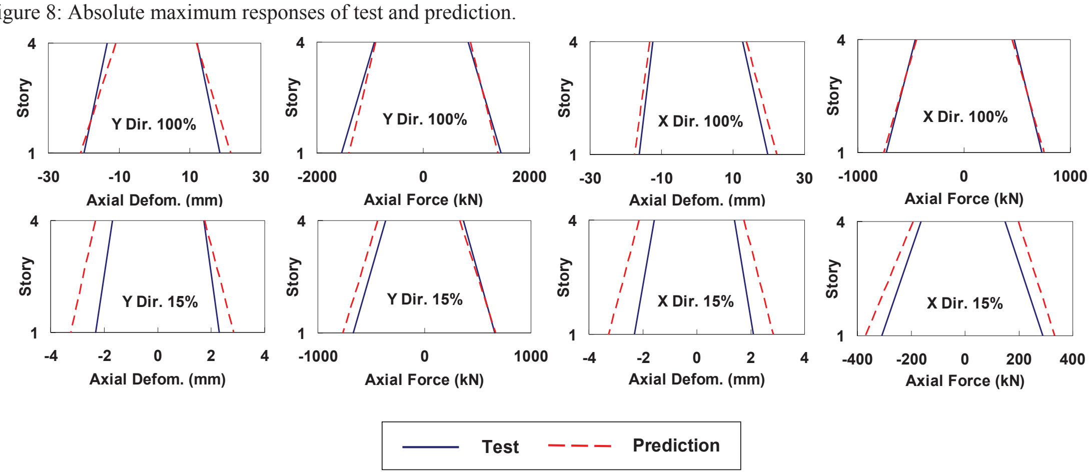


Figure 9: Maximum (tensile) and minimum (compressive) responses of steel dampers.

5. Summary and Conclusions

This paper describes the modeling techniques and the prediction results in the participation of the 2009 E-defense blind contest. The prediction results show well agreement with the test responses. Considering the velocity dependency of the BRB according to the dynamic loading test results can satisfactorily predict the BRB strength when the BRBF was subjected to the dynamic excitations. Analytical simulation will be conducted to further improve the accuracy of the numerical model in the near future.

Acknowledgements

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