Numerical Modeling of BRB Frame Systems With and Without Concrete

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***Abstract*—Bracing as an inactive control system can play important role in structure resistance to side forces such as earthquake one of the best and economic methods of utilizing bracing capability is the use of their inflexible capacity. Finite element modeling of buckling restrained braced is so difficult because of complicate interaction between steel and concrete. In this study two finite element models of BRBs include model with and without concrete have created and verified with experimental results. The result of this study shows that the model without concrete can be used as an alternative of model with concrete.**

Keywords*— *Buckling restrained brace; Springs; Numerical modeling

I. Introduction

Steel braces are often used to provide lateral stiffness of steel structures as an economic means. However, buckling of braces is with unsymmetrical mechanical behavior in tension and compression and in results energy dissipation capacity of a steel braced structure and ductility subjected to earthquakes are limited. The braced frame typically exhibits substantial deterioration of strength when loaded in compression monotonically or cyclically. Xie [1] studied on state of art on buckling restrained braces. If the buckling of a steel brace is restrained and the same strength is ensured in both tension and compression, stable performance of braces will be assured and the ductility and hysteretic behavior will be improved [1-3]. The buckling-restrained brace consists of a steel core encased in a steel tube filled with concrete. The steel core carries the axial load while the outer tube, via the concrete (buckling–restraining mechanism), provides lateral support to the core and prevents global and local buckling. A thin layer of unbounded material along the steel core at the concrete interface eliminates shear transfer during elongation and contraction of the steel core and also accommodates its lateral expansion in compression. It is the ability of the steel core to contract and elongate freely within the confining steel concrete-tube assembly that leads to the name unbounded brace (UB). Results from past studies [2–6] showed that BRBs can undergo fully-reversed axial yield cycles without loss of stiffness or strength, which exhibits similar yielding and ultimate strength and good seismic energy dissipation, and the ultimate ductility and cumulative plastic ductility of that are quite beyond demand.

A 0.7-scale one-bay one-story Buckling-Restrained Braced Frame (BRBF) was tested under cyclic displacement histories by Aiken et al. [7] at the University of California, Berkeley. Cracks occur in the beam, column, beam–column–brace connections and gusset plates due to torsional buckling of the beam and out-of-plane displacement of the BRBs. Tsai et al. [8, 9] conducted two tests on big-scale BRBFs at the National Center for Research on Earthquake Engineering (NCREE).

Long brace-gusset plate connection of BRBs leaded to buckling of gussets at story drift of 0.01 rad. The cyclic behaviors of five full-scale one-bay one-story BRBFs were tested by Christopulos [10]. BRBs were connected to the frame with gusset plates and bolts; and beams were connected to the columns with single-plate shear tabs. The beams and columns close to BRB connections yielded and buckled, and then BRBs failed. Roeder et al. [11] conducted the tests of five full-scale one-bay one-story BRBFs at the University of Washington. The performances of BRBFs were influenced by gusset plate geometry, type of bolted brace–gusset plate connection, and orientation of the BRB core plate.

Failures of BRBFs were attributed to out-of-plane distortion of the BRB at story drift ratio between 0.022 and 0.024. Fahnestock and Victoria [12] did the experimental research of a 0.6-scale four-story BRBF by using hybrid pseudo-dynamic earthquake simulations and quasi-static cyclic loading. The beams were connected to beam stubs using bolted web splices and BRB were pinned to gusset at beam–column joints.

During the earthquake simulations, the frame did not exhibit substantial deterioration of strength and stiffness at a story drift ratio of 0.48. The test was finished when yielding segments of inner core of BRBs fractured. It is concluded that the frame with proper design had the ability to withstand severe earthquake and maintain its loadbearing and deformation capacity. It is found that one main failure mode of BRBF is the fracture of beam–column–brace gusset welds due to frame action. A four-story BRBF tested by Victoria and Fahnestock [13] was analyzed based on a three-dimensional FE mode in ABAQUS, which was calibrated with test results. The influences on global structural response and local connection demand for different types of connection configurations are studied. BRBFs may not allow the braces to realize their full ductility capacity due to connection failure modes.

Chou and Chen [14] proposed an inelastic plate buckling equation together with coefficient charts to predict ultimate load of gusset plate connections of BRBF. Free-edge stiffeners welded to central gusset plates were demonstrated to be an effective way to increase yielding load or post-yield strength of gusset plate connections. The dual gusset plates sandwiching a BRB core reduce gusset plate size, eliminate the need for splice plates, and enhance connection stability under compression. Chou and Liou [15] conducted the experimental and nonlinear finite element analysis program to investigate ultimate compression load and bending rigidity by testing ten large dual gusset- plate connections used for BRBFs. The ultimate compression load of the dual-gusset-plate connection was reasonably predicted by suggested computation model. A design procedure which considers both frame and brace action forces on the corner gusset connections was proposed by Chou and Liu [16]. The research of Chou and Liu [17] found that without free edge stiffeners, the single corner gusset plate buckled at a significantly lower strength and the buckling could be eliminated by using dual corner gusset plates similar in size to the single gusset plate. At low drifts, the frame action force on the corner gusset was of the same magnitude as the brace force. At high drifts, however, the frame action force significantly increased and caused weld fractures at column-to-gusset edges.

Jeffrey [18] proposed a novel connection where the gusset is only connected to the beam and is offset from the column face. A three story frame with the novel connection was tested under quasi-static cyclic loading. The connection can withstand 3% frame drift and the performance of the frame is very good. Large-scale shake table tests were performed to examine the out-of-plane stability of BRBs placed in a chevron arrangement in a single-bay, single-story steel frame [19]. A simple stability model predicted the BRBs with a flexible segment at each end of the steel to fail due to out-of-plane buckling at a force smaller than the yielding strength of the steel core. It is found that BRBF provide more stable hysteretic behavior than conventional special concentrically braced frames (SCBF). A high confidence of BRBF of achieving the collapse prevention limit state was provided [20]. A three-story single-bay full-scale BRBF was tested under a series of hybrid and cyclic loading tests [21]. (BRBs) were installed in the frame specimen. BRBs include two thin BRBs and four end-slotted BRBs which all using welded end connection details. The recommendations on the seismic design of thin BRB steel casings against local bulging failure were put forward.

In the past, many experimental and analytical studies were done on the behavior of BRBs, but there is still limited experimental and numerical data on system-level performance of BRBFs. The studies that have been conducted on the BRBFs have also identified undesirable failure modes, such as the damage to the beam and beam-column-BRB connections region due to frame and brace action forces, including fractures of the gusset and beam welds, local buckling on the flanges and webs of the beams and enforced loops. Also numerical modeling of BRBs is difficult because of steel-concrete interaction problems. In this study new method is provide to resolve this issue. This study is to compare two types of numerical modeling of buckling restrained braces such as models with and without concrete. In the model without concrete, springs used to provide braces against buckling.

II. NUMERICAL MODELING

A. *Experimental Model Details*

The analytical study involves developing finite element model of buckling restrained brace frame system (BRBs) for the purpose to show a new way on finite element method. Thus two story frames with the general configuration shown in Figure 1 were employed. The section of circular steel tubes is 219 × 4 (mm, diameter x thickness, D × T); the H section of middle beam is 194 × 150 × 6 × 9 (mm, flange width × web height × flange thickness × web thickness); and the H section of top and bottom beams are 300 × 150 × 6.5 × 9 (mm, flange width × web height × flange thickness × web thickness). BRBs with a rectangular inner core have a section of 100 × 8 (mm, width × thickness). The detail of BRB member has shown in Fig. 1.There is a half story at the bottom of the frame as shown in Fig. 2, the half story height is 600 mm.

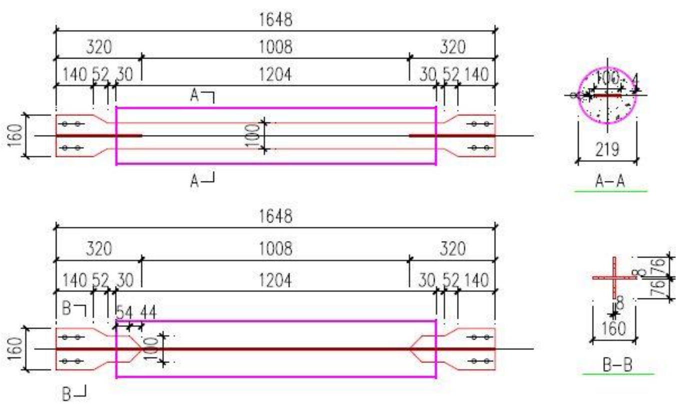

Figure 1: Detail of BRB specimen

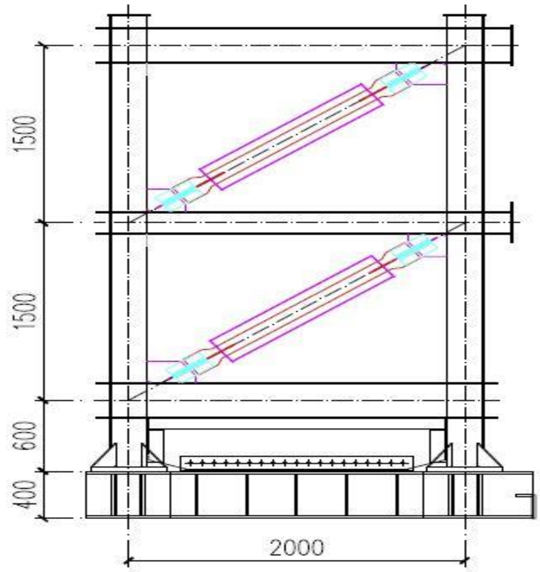

Figure 2: Dimensions of frame specimen

B. Mterial Property

The model also contains non-linear material property, non-linear geometric behavior and non-linear analysis. The material properties of all the structural steel components were modeled using an elastic-plastic material model from Abaqus. Non-linear material property in an Abaqus model requires the use of the true stress (σ) versus the plastic strain ($\mathcal{E}^{pl}$) relationship, this must be determined from the engineering stress-strain relationship. The stress-strain relationship in compression and tension are assumed to be the same in Abaqus. Abaqus approximates the smooth stress-strain behavior of the material with a series of straight lines joining the given data points to simulate the actual material behavior. Any number of points can be used. Therefore, it is possible to obtain a close approximation of the actual material behavior. The material will behave as a linear elastic material up to the yield stress of the material. After this phase, it goes into the strain hardening phase until reaching the ultimate stress. The density should be defined for members when dynamics analysis used for models. Combined (isotropic -linear kinematic) hardening rule with a Von Mises yielding criterion is applied to simulate the plastic deformations of the models shell components. Steel mechanical properties were included: Young's modulus=2.1E6 kg/cm^2, Poisson's ratio=0.3, yield stress=3610 kg/cm^2, ultimate strength=5080 kg/cm^2.

The constitutive behavior of concrete is modeled using a three-dimensional continuum, plasticity based damage model [23]. The concrete damaged plasticity model is efficiently capable of modeling concrete in all types of elements like beams, trusses, shells and in present case, especially solids. Inelastic behavior of concrete is depicted using the concept of isotropic damaged elasticity along with the isotropic tensile and compressive plasticity. The value of concrete mass density is 2400 kg/m^3. The concrete compressive strength was assumed to be a nominal 28 N/mm^2. The compressive yielding curve was taken as that of a typical concrete from [24]. The tensile cracking stress was assumed to be, conservatively, approximately 5.6% of the peak compressive stress as recommended in [24]. After tensile cracking, the stress-strain relationship in tension softens as load is assumed to be transferred to the reinforcement. The tensile strength of the concrete is ignored after concrete cracking.

C. Boundary Condition and Loading

Three–dimensional nonlinear finite element of BRBs was created using Abaqus computer program [18]. A displacement-control loading was applied on the tip of the floor by imposing cyclic displacement based on SAC loading protocol (Fig. 3) [25]. The floor tip displacement corresponding to the inter story drift angle of 0.01 rad. was 3 cm.

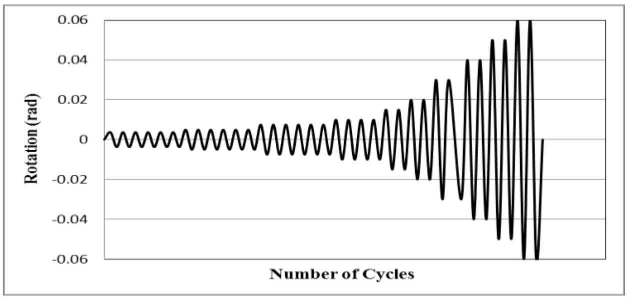

Figure 3: SAC protocol loading

D. Interactions

In the experimental modeling beams to columns and braces plates to beam-column connected by weld operations. To simulate weld connection, tie constraint used to define the interactions between steel components. Furthermore tie constraint was selected to define interaction between steel and concrete. Furthermore, concrete was deleted on model of BRBs without concrete. On the other hand some springs is used an alternative method for concrete roles.

E. Springs Stiffness

If the steel core elements size is too small that steel core buckling load is greater than the core yield load, it can be avoided in some nodes to define its springe. Buckling force of a core with a rectangular profile on weak directions obtain as follows:

$$P_{cr} = \frac{\pi^2 Ebt}{12L^2} \quad (1)$$

Where E, b, t and L is Yang module of steel core, steel core height, steel core wide and an element length respectively. The magnitude of steel core yield obtain be Eq 2:

$$F_y = bt\sigma_y \quad (2)$$

To calculate the minimum length of mesh that all nodes need springs, it should make Eq 1 greater than Eq 2 which resulted:

$$L < \frac{\pi t}{\sqrt{\frac{12\sigma_y}{E}}} \quad (3)$$

Figure 4 shows the model with spring (without concrete) which the spring stiffness is 850kN/mm.

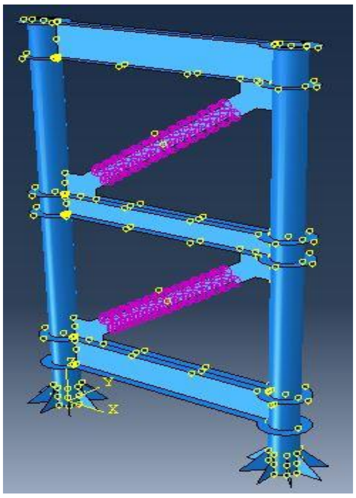

Figure 4: The BRB model with springs.

F. Meshing

All steel components such as columns, beams and steel brace core were modeled using 4-node shell element (S4R in Abaqus elements library) with plasticity, large deflection, and large strain capability. This element has six degrees of freedom per node. Figure 5 and 6 shows a typical finite element meshing used in this study. Also C3D4R used for concrete. This element has 4 nodes and three degree of freedom per node. In addition this element usually uses for complicate geometry part of model which other types of solid element can NOT support it.

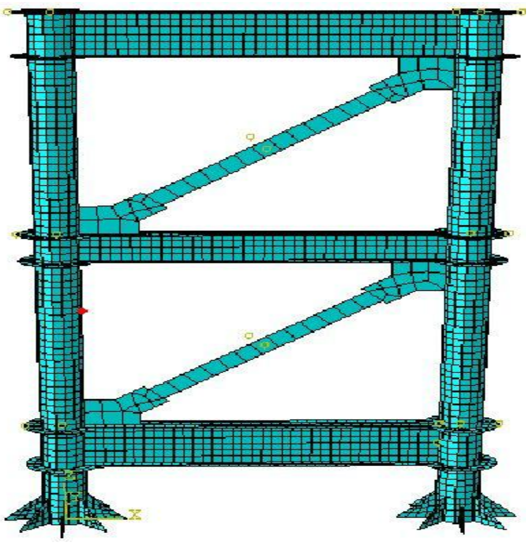

Figure 5: The BRB model without concrete.

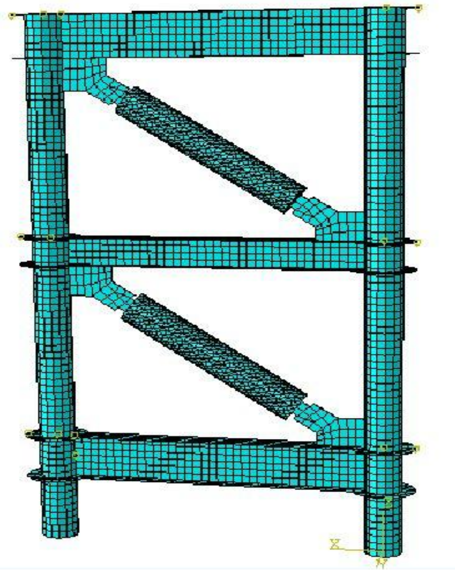

Figure 5: The BRB model with concrete.

III. VERIFICATION

To verify the analytical models, an experimental specimen tested by M. Jia et al. [26] was modeled. The specimen includes composite moment frame and BRBs. The frame consists of concrete-filled circular hollow section steel tube columns and steel beams. There is a close agreement between the experimental results obtained by M. Jia et al. [26] and the numerical results. It can be seen from the figure 7 and 8 that the maximum shear force for experimental specimens and finite element model with concrete is 450 kN and 415 kN, respectively; which shows a 7% difference in maximum values. It can be seen from the figure 7 and 9 that the maximum shear force for experimental specimens and finite element model without concrete (model with spring) is 450 kN and 445 kN, respectively; which shows a 1.5% difference in maximum values The comparison between the test and finite element analysis indicates that the finite element modeling procedures produce an accurate model, which should lead to accurate response prediction in the parametric study. In addition the model with springs (without concrete) provided convergence error that occurred in the model with concrete.

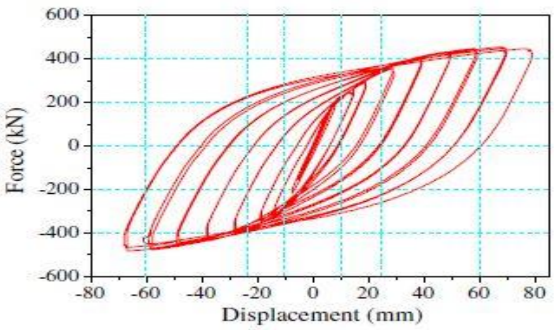

Figure 7: Hysteretic curve for experimental result

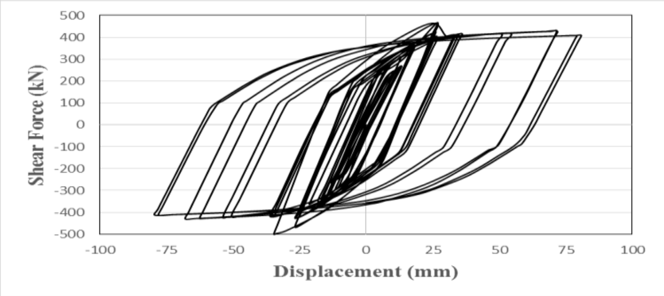

Figure 8: Hysteretic curve for BRB model with concrete

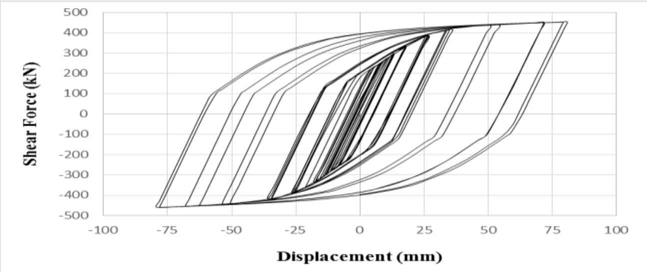

Figure 9: Hysteretic curve for BRB model without concrete

CONCLUSIONS

In this study two finite element models of BRBs include model with and without concrete have created and verified with experimental results. All steps of BRBs modeling have been mentioned.

The result of this study shows that the model without concrete can be used as an alternative of model with concrete. As indicate it can trust to finite element models as reliable method to consider buckling restrained brace frame system.

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