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ANALYTICAL STUDIES OF A FULL-SCALE STEEL BUILDING SHAKEN TO COLLAPSE

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***Abstract**. This paper describes the numerical simulations for earthquake engineering research. The applications of a general-purpose structural analysis computer program PISA3D, developed in Taiwan National Center for Research on Earthquake Engineering (NCREE), on the seismic frames are introduced. In this paper, a four-story building is used as a case study to investigate modeling techniques for nonlinear structural response and collapse analyses. This paper presents several numerical models in detail. The models for the simulation of steel hollow structural column buckling are discussed. A column model incorporating the fibered beam-column element using the cyclic buckling fibers proposed in this study can satisfactorily simulate the degrading force versus deformation relationships found in the ABAQUS column model subjected to the local buckling. A frame model incorporating the same type of fibered column element could predict the collapse time of the test building.*

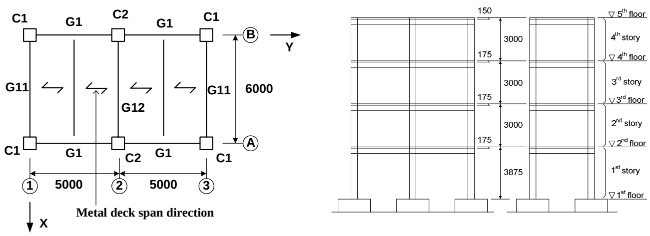
1 INTRODUCTION

In September 2007, the world's largest 3-directional earthquake simulation table, E-Defense shaking table was utilized for tests of a full-scale four-story steel building shaken to collapse. Before the tests were executed, a blind prediction contest was held [1]. Three groups of researchers from the Taiwan National Center for Research on Earthquake Engineering (NCREE) participated in the blind contest [2]. One team used a general purpose nonlinear structural analysis program PISA3D, and submitted the 3-D and 2-D predictions. This PISA3D/NCREE team was awarded second place in the 2-D analysis research category. In this paper, this four-story building is used as a case study to investigate modeling techniques for nonlinear structural responses and collapse analysis. This paper presents several numerical models in detail. The models for the simulation of steel hollow structural column buckling are discussed. A basic model incorporating the bilinear plastic hinge column elements was constructed first. Then the refinements were carried out by replacing the 1st-story columns in the basic model with the nonlinear degrading column elements. Two types of degrading column are considered including the fiber and hinge models, with and without the effects of axial-flexural interaction, respectively. Based on these nonlinear response analyses, it is illustrated that the collapse responses of the building can be estimated satisfactorily by incorporating degrading fibered columns.

The photo of the full-scale four-story steel building is shown in Fig. 1. The contestants were required to predict the key maximum experimental responses of the specimen under the three-directional incipient-collapse level shaking as well as the specimen's collapse time under the collapse level earthquake [2]. The floor framing plan and the elevations are shown in Figures 2a and 2b, respectively. The longitudinal and transverse directions of the building are defined as Y and X directions, respectively.

The structural configuration consists of two-bays of 5m each in the Y direction and one-bay of 6m in the X direction. Each story height is 3.5m except the first story is 3.875m high. The thickness of the concrete slab is 175mm for the 1st, 2nd and 3rd floor and 150mm for the 4th floor. The 75mm deep metal deck was spanning in the Y direction. Wide flange sections are used for beams, and rectangular hollow structural sections (HSS) for columns. The steel material is SN400B for the frame beams and BCR295 for the columns. The building was designed following the current Japanese specifications and practices [3].

Figure 1: Collapse of the four-story steel building.



(a) Floor framing plan (b) Frame elevations

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

The building specimen was subjected to 3-directional ground motions recorded during the 1995 Kobe earthquake at the Takatori train station. The test consisted of repeated applications of the records with progressively increasing scale factors. The Takatori motion scaled by 0.6 was designated as the incipient-collapse level motion, and the full scale Takatori motion was designated as the collapse level motion. For the purposes of discussion, the incipient-collapse level and collapse level excitations are defined as EQ-I and EQ-C, respectively.

2 PISA3D

The Platform for Inelastic Structural Analysis of 3-D Systems (PISA3D) [4] is developed in the Dept. of Civil Engineering of National Taiwan University and maintained in NCREE. It is an object-oriented general-purpose computational platform for nonlinear structural analyses. The PISA3D incorporates the object-oriented concept and the Design Patterns to construct the software framework, making it easy to maintain and extend. PISA3D provides a rather large variety of nonlinear materials and elements. PISA3D users could build 3-D analytical models and perform nonlinear analyses to investigate the responses of structures under the combined load effects. Users can download the program for free from http://pisa.ncree.org.tw. In this paper, PISA3D is adopted to present the response simulations of the frame specimen.

3 BASIC MODEL

In the basic model, the frame mass is considered as a lump mass located at the top elevation of the concrete slab of each floor. However, the location of the mass is eccentric due to the asymmetric configuration of the exterior walls. The Newmark method of constant average acceleration scheme (β=1/4) is used for the time integration. The Rayleigh damping ratio of 2% for the first and second modes is adopted for the 3D model. The modeling techniques of the basic model are described below.

3.1 Fibered beam model

All the frame beam members are considered as the steel and concrete full-composite beam in the basic model. The beam model adopted fibered beam-column element to represent the composite floor beam. The fibered beam-column element in the PISA3D is flexibility-based. The element formulation relies on force interpolation functions that strictly satisfy the equilibrium of the bending moments and the axial force along the element [5]. The steel beam consists of 13 fibers using the bilinear material model and the yield strength provided by the contest organizers. The stress versus strain relationship of the concrete provided by the organizer was used for concrete fibers. Five integration points along the fibered beam-column element were chosen to integrate the element responses.

3.2 Panel zone joint

From the beam-to-column subassembly tests [6] conducted by the organizer, shear deformations were evident in the panel zone. Thus, in the basic model, the panel zone deformations were specifically included by introducing zero-length joint elements while the member rigid end offset feature for the joining beam and column ends were incorporated into the analytical model. In the present study, rotational stiffness, *K* and yield capacity, *My* for all panel zone joint elements were calculated using the following two equations [7]:

s

$$K = \frac{G d_b d_c t_j}{\left(1 - \frac{d_b}{H}\right)} \tag{1}$$

$$M_y = \frac{0.55 F_y t_j d_b d_c}{\left(1 - \frac{d_b}{H}\right)} \tag{2}$$

where d_b, d_c are the beam and column depth, respectively. The t_j and *H* are the thickness of the panel zone and the story height, respectively.

3.3 Hinge model for the column

In the basic model, hinge-model beam-column element was adopted for all the column members. This beam-column element is a lumped-plasticity model. It has an elastic component that is series-connected by one shear and one flexural plastic hinges at each end. All the nonlinear deformations can only take place in these hinges. The bilinear material property was adopted. The plastic section modulus and the steel yield strength provided by the organizers were used. No axial-flexural interaction is considered in this model. The material strain hardening ratio was assumed to be 0.02.

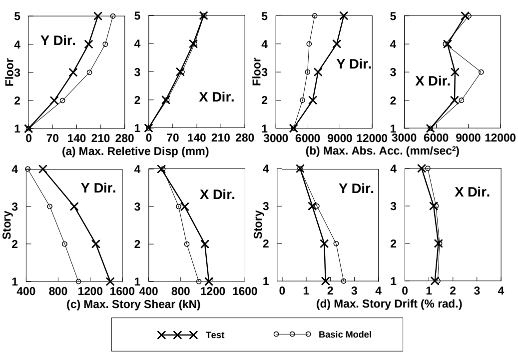


Figure 3: Absolute maximum responses of basic model subjected to the EQ-I.

3.4 Comparisons between the basic model simulation and test results

Figures 3a to 3d show the absolute peak responses of the basic model under the EQ-I. It is evident that the displacement-related responses (maximum floor displacements and story drift) in the X direction are very close to the test results. Figures 3a shows that the maximum floor displacements computed from the basic model were larger than the test results. The basic model underestimated the story shear responses (Figure 3c). For some unknown reasons, this under-prediction on story shear has been found common in most teams participated in the blind contest [8].

From Fig. 3a, it can be found that the analytical Y-direction displacement responses were somewhat greater than those in the X direction. This finding agrees with the observed test responses. Under the EQ-C, the test frame did collapse primarily along the Y direction [3]. Figure 4 shows the first story drift time history responses in the Y direction of the basic model under the EQ-C. It can be found that the collapse drift ratio of 0.13-radian defined by the contest organizers was never reached in the response history analysis using this basic model. Further studies are necessary.

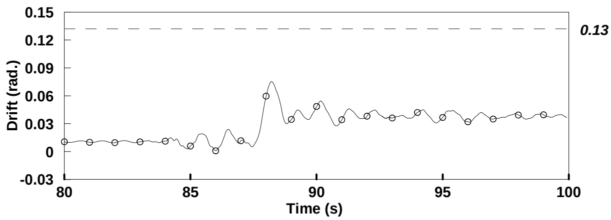


Figure 4: First story drift time histories in the Y direction under the EQ-C for the basic model.

Figure 5: Local buckling of the first-story column at the end of the tests.

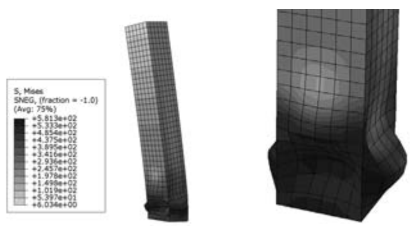


Figure 6: Local buckling of the ABAQUS column model.

4 REFINE THE COLUMNS USING PARALLEL MATERIAL HINGE MODEL

When the specimen was subjected to the EQ-C, local buckling occurred at both the bottom and top ends of several first story columns [3]. Figure 5 shows the bottom column end local buckling of the 1st-story column specimen at the intersection of Frame Lines A and 2 (Figure 2a). This local buckling led the building into a side-sway collapse mechanism in the first story [3]. Recall in the basic model that the column strength degradation due to the local buckling was not specifically considered as all column elements were represented using bilinear material. This should be helpful in explaining why the basic model failed to predict the collapse of the specimen.

In order to refine the column properties in the frame model, an independent ABAQUS [9] finite element (FE) column model was constructed to study the local buckling response. This column analytical model is 1938 mm high (one half of the 1st-story column of the specimen) and the cross section is HSS300×300×9 mm, identical to the column members in the test frame. The column bottom end is fixed but the top end is free. The FE model was constructed using the 4-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 936 shell elements. This FE column model was subjected to a constant axial load of 257 kN (observed from the gravity load in the bottom corner column in the basic model) and cyclic increasing lateral displacements in one direction only.

Figure 6 shows the local buckling response of the ABAQUS column model when the column top end reached 0.04 radians lateral drift. Figure 7 shows the ABAQUS column base moment versus top end lateral drift relationships. It is evident that the HSS300×300×9 mm column strength degradation is rather pronounced when the lateral drifts are greater than 0.02 radians. In order to refine the column element properties in the frame model, the "Parallel" material model in PISA3D was adopted in this study. The construction of the hinge-buckling-column "HBC" model utilized two material properties and the feature of parallel material to simulate the cyclic local buckling response of the column. In the present study, the parallel material property as shown in Fig. 8c is a result of combining the degrade material given in Fig. 8a with the hardening material illustrated in Fig. 8b. The cyclic responses of a PISA3D "HBC" model for the 1938 mm high HSS300×300×9 mm column using the Parallel material are also plotted in Fig. 7. It can be found from comparing the ABAQUS and PISA3D results that the column's cyclic strength degradation due to the local buckling can be better simulated from using the HBC model than the bilinear hinge column model. The effects of axial loads on the accuracy of the HBC models will be discussed later.

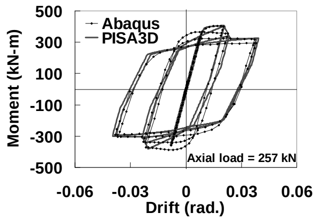


Figure 7: Comparing the HBC cyclic column lateral force versus deformation responses with the ABAQUS FE results.

(a) Degrade Material (b) Hardening Material (c) Parallel Material

Figure 8: Parallel material model in PISA3D (a) + (b) = (c)

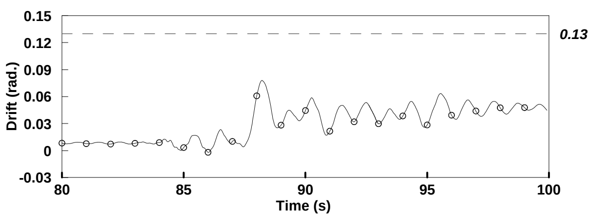


Figure 9: First story drift time histories in the Y direction under the EQ-C for the HBC model.

From the findings stated above, the refined HBC model was constructed and studied by replacing the 1st-story columns in the basic model with HBC model columns. The responses of the refined HBC frame model were very similar to the basic model. Figure 9 shows the first story drift time history response of the refined HBC model under the EQ-C. It can be found that the 0.13-radian drift was still not reached in the refined HBC frame model. The basic and the refined models introduced so far never considered the effects of interactions among the varying axial loads and bi-axial bending moments in columns as hinge models were adopted. Using the fiber model for column, the axial-flexural interactions can be conveniently incorporated into the frame model. In this approach, the compressive stress degradation characteristic has been implemented into the buckle-material fiber model in order to capture the local buckling responses of the column.

5 REFINE THE COLUMNS USING BUCKLE MATERIAL FIBER MODEL

The Buckle material in PISA3D adopted the cyclic response rules proposed by Maison and Popov [10] as shown in Fig. 10. Users can specify the stress and strain values at the control points to adjust the hysteresis responses. The "FBC" column model adopted fibered beam-column element with a cross section consisting of 44 fibers using the Buckle material model given in Fig. 10. Five integration points were used. In this study, the degradation characteristics in the cyclic responses obtained from the aforementioned ABAQUS FE column analysis was used to calibrate the parameters of the FBC column model. Two levels of column axial loads (257 and 515 kN for corner column and center column, respectively) were chosen. Figure 11 shows that at both two different axial force levels, the FBC column model can satisfactorily simulate the cyclic degrading responses of the ABAQUS FE model. It can be found that under the 515 kN constant axial load, the cyclic degradation of the ABAQUS FE model is severer than in the case of 257 kN axial load. This phenomenon has been well captured by using the same set of degrading parameters in the two FBC models. Based on this finding, a more refined frame model was constructed.

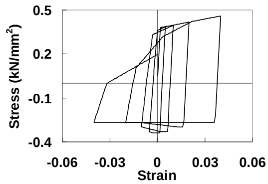


Figure 10: The stress versus strain relationship of each fiber in the FBC model.

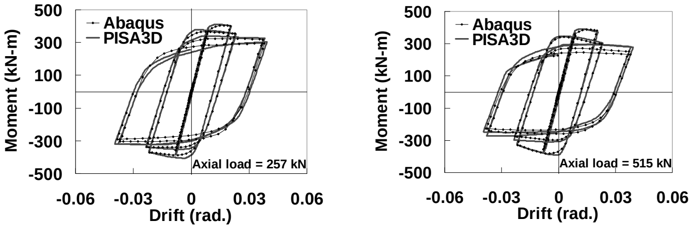


Figure 11: Comparing the FBC cyclic column lateral force versus deformation responses with the ABAQUS FE results for two levels of column axial loads.

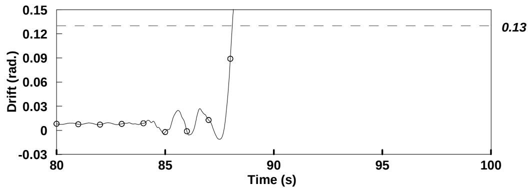
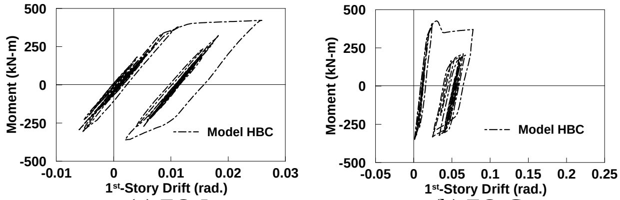


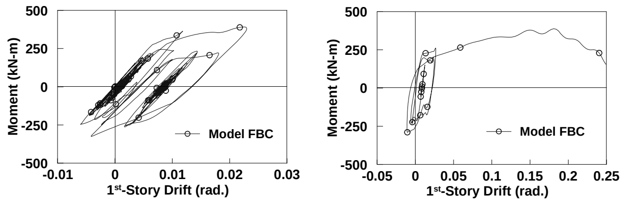
Figure 12: First story drift time histories in the Y direction under the EQ-C for the FBC model.

The refined "FBC" frame model was constructed by replacing the 1st-story columns of the basic model with the FBC-model columns. Figure 12 shows the 1st-story drift time histories under the EQ-C for the FBC frame model. It can be found in the figure that this model reached the 0.13-radian collapse criterion and the time of the occurrence was very close to the test results [11]. Figures 13 and 14 show the "HBC" and "FBC" center column base moment (toward the longitudinal direction of the frame collapse) versus 1st story drift relationships for two levels of excitations, respectively. Under the EQ-I, the strength degradation was not yet developed in Model HBC as shown in Fig. 13a. However, it is evident that the strength deterioration in Fig. 14a (representing the column buckling) has occurred in Model FBC during the EQ-I. As a result, the column base responses of the two models are very different during the EQ-C as evidenced in Figs. 13b and 14b. The reduced strength as observed in the beginning of the FBC model in Fig. 14b was very similar to the response reported in the reference [3]. The strength deterioration was not reached in Model HBC during the EQ-I (Figure 13a) and its initial strength in the beginning of EQ-C (Figure 13b) was higher than that in FBC frame model. These results seem to fail HBC frame model to predict the collapse responses. The FBC frame model incorporating the effects of interactions among the varying axial loads and bi-axial bending moments in columns have captured the collapse of the building specimen. This should suggest that the collapse of the test frame, or the accuracy of analytical collapse simulation, is strongly governed by the severe local buckling of the columns in the first story. The cyclic column axial and bi-axial flexural interactions are critical in developing the local buckling of the columns. The proper column analytical model using reasonable degrading rules is important for the collapse simulation.



(a) EQ-I (b) EQ-C

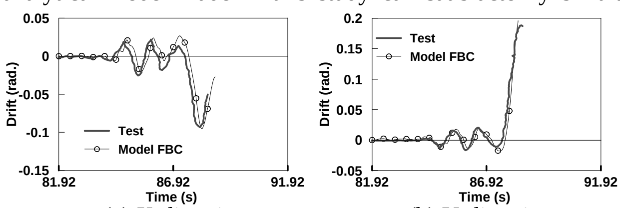
Figure 13: Column bottom end moment versus 1st-story drift relationships in Model HBC.



(a) EQ-I (b) EQ-C

Figure 14: Column bottom end moment versus 1st-story drift relationships in Model FBC.

Figure 15 compares the X and Y directions' 1st-story drift time histories of Model FBC and test results [11] under the EQ-C. In the both directions, the analytical results show good correspondence with the test responses. In addition, the time instant at which the inter-story drift of the analytical model reached 0.13 radians appears very close to that measured from the test (Figure 15b). It is illustrated that the refined FBC analytical model made in this study can satisfactorily simulate the collapse of the specimen.



(a) X-direction (b) Y-direction

Figure 15: First-story drift time histories under the EQ-C.

6 SUMMARY AND CONCLUSIONS

Based on the analytical and experimental studies, summaries and conclusions can be made as follows:

1. The basic frame model using bilinear hinge-model column elements failed to predict the frame collapse time. The key reason is that the column strength degradation due to the local buckling has not been considered.

2. Under the EQ-I, the strength degradation was not yet developed in the HBC frame model but was evident in the FBC frame model. The reduced strength as observed in the beginning of the FBC model was very similar to the response of the test specimen. This strength deterioration was not reached in the

HBC frame model during EQ-I and its initial strength in the beginning of EQ-C was higher than that in the FBC frame model. These results seem to fail the HBC frame model to predict the collapse responses.
3. The ABAQUS column local buckling responses can be represented using the PISA3D fiber-model column with Buckle Material. The proposed FBC column model can conveniently incorporate the combined cyclic strength degrading effects among the varying axial loads and bi-axial bending moments. The collapse of the test frame is strongly governed by the severe local buckling of the columns in the first story. The collapse time can be predicted in the FBC frame model using this column model element. This suggests that the analytical force versus deformation relationships of the first story columns strongly affects the frame collapse prediction. The column analytical model with proper degrading rules is important for the collapse simulation.

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