3-D ANALYSIS METHODS FOR 2007 BLIND ANALYSIS CONTEST

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ABSTRACT :

A blind analysis contest has been conducted in conjunction with the full-scale total collapse test of a four-story moment frame in September 2007 at the E-Defense shake-table facility. The purpose of the contest is to stimulate development of computational methods and efficient modeling techniques for collapse analysis. In this report, the methods of modeling and analysis of the three winners of the category of three-dimensional analysis are presented.

KEYWORDS: Blind analysis contest, Three-dimensional analysis, Seismic response, Steel frame

1. INTRODUCTION

A shake-table test of total collapse of a four-story moment frame was carried out in September 2007 at the E-Defense shake-table facility, Japan. A blind analysis contest has been carried out in conjunction with the test to stimulate development of computational methods and efficient modeling techniques for collapse analysis of steel frame buildings. See Ohsaki *et al*. (2008) for details of contest rules and results. The contest has two categories of 2D-analysis and 3D-analysis, which are further divided to the categories of researchers and practicing engineers by the types of the participants. In this study, the results of the award-winning groups in the categories of 3D-analsysis are reported.

2. AWARD-WINNING RESULTS BY RESEARCHER – 1

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2.1 Modeling details and limitations

The primary objective of the researchers was to create a realistic and practical model using commercial software – SAP2000, which was chosen due to familiarity from previous projects. The decision on which details, furnished during the experimental program, to eliminate and which ones to retain and implement in the model were based on the options a practicing engineer would be faced with when starting work on such a project.

The beams and columns were modeled as frame elements with properties specific to Japanese sections. The connections were modeled as simple moment connections. Both the floor and roof slabs were modeled using membrane elements. The model contained 168 frame elements and 48 shell elements. All columns were considered fixed at the base, resulting in a total of 105 joints and 306 equilibrium equations, and hence the computer time required for the model was relatively low with a typical analysis for all the modal and time history computations taking two to three minutes. The low computational demand, for a relatively simple structure, allowed for numerous trials prior to the submission of results. The contestants decided to exclude the effect of connection-specific test data because it was impractical to gage its effectiveness given the short response time for the results. Floor subsystems, such as the collapse prevention structure and the walls, were taken as a lumped mass at the center of the floor area.

2.2 Time History Analysis of the Frame

Three sets of time histories, 40%, 60%, and 100% Takatori waves were input in SAP 2000 as user-defined data. Each set consisted of a scaled version of the eventual time history intensity. The peak intensities were more than 0.85 g in the EW and NS directions, while the UD accelerations had a peak value of about 0.4g. Each of the three intensities was run as load cases in which the results from one run were transferred to the other. However, the analysis did not explicitly account for any loss of stiffness in the structure due to cracking and damage after each step. The analysis was run using the nonlinear modal time history analysis method with a constant damping for both mass and stiffness of 0.02. The periods of four lowest modes were 0.869, 0.821, 0.623 and 0.284 sec.

The figures shown below provide the following observations.

a. The maximum relative displacements were predicted very accurately in the Y-direction but overpredicted in the X-direction.
b. The maximum absolute accelerations were predicted reasonably well in the Y-direction, while in the X-direction they were underpredicted at the second floor level and overpredicted at the fourth and fifth floor levels.
c. The story shears were predicted accurately in the Y-direction but overpredicted in the X-direction.
d. The maximum relative drift angles in the X- and Y-directions were predicted very closely.

As frame elements were used for columns, we could not submit a prediction for the axial strain in them at the base level. One observation that stands out is that the predictions were very close in the Y-direction but off in the X-direction. There are two reasons to which we attribute this, namely a) damping effects due to mass and stiffness were not given very careful consideration, and b) panel zone effects at beam-column junctions were not incorporated. Further study of the damping effects is underway. It was observed from the final collapse mechanism of the experimental structure that the failure was probably due to plastic hinge formation in the X-direction at the base of the structure. Hence, inclusion of the panel zone and plastic hinge effects could have resulted in a more accurate collapse prediction.

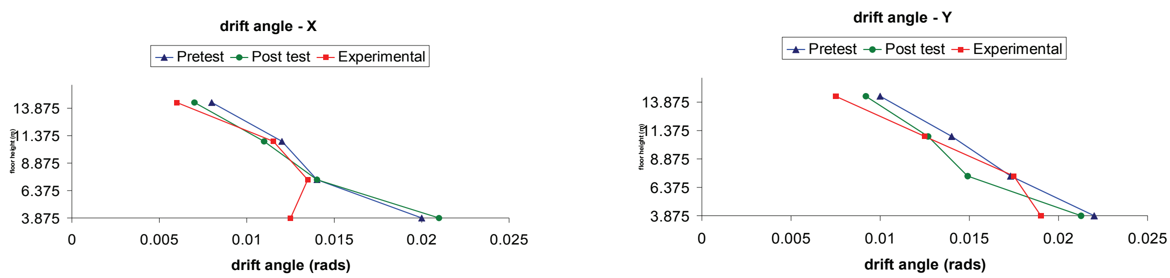

Figure 1 Maximum story drift angles.

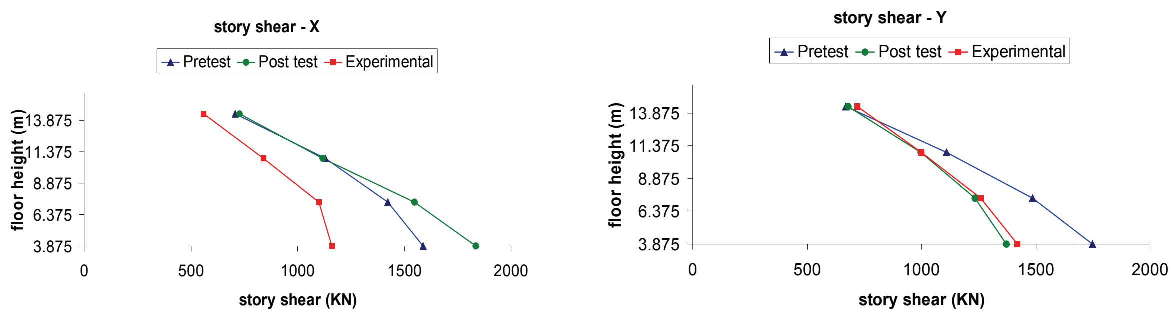

Figure 2 Maximum story shears.

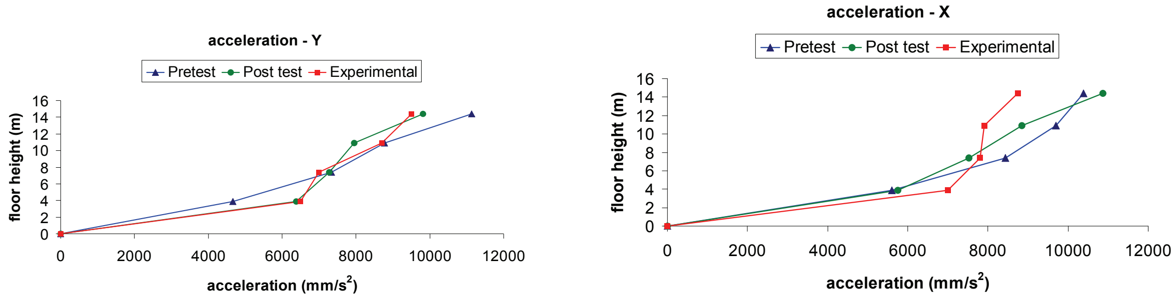

Figure 3 Maximum absolute accelerations.

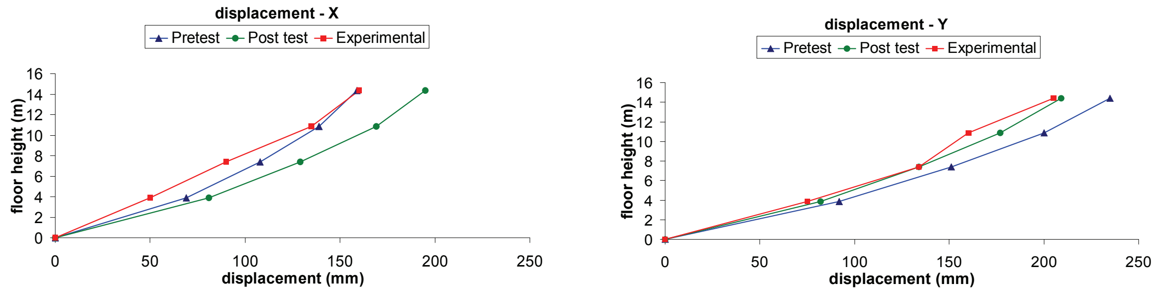

Figure 4 Maximum relative displacements.

3. AWARD-WINNING RESULTS BY RESEARCHER – 2

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3.1 Finite Element Model

The 3D finite element analysis using an open-soruce code OpenSees Ver. 1.7.1 (McKenna and Fenves, 2002) was carried out to simulate the shaking table test results of the four-story steel moment frame. OpenSees Navigator (Schellenberg and Yang, 2005) was employed for pre- and post-processing in this work. The details of the geometrical and mechanical properties of the frame is described in Tada *et al*. (2007). As shown in Figure 5, the numerical model of the structure was mainly simulated by beam-column elements. Nonlinear displacement-based beam-column elements with fiber sections were adopted. Elements were refined at beam-column joints. Column bases were modeled by linear rotational springs. Two specious protection elements using OpenSees gap material were used to prevent unlimited falling after collapse.

Each beam component was modelled by a displacement-based beam-column element with five integration points. Mesh refinement was not applied to beams because of a presumption that beams would not suffer severe nonlinearity according to a simulation result (Tada et al., 2007). Fiber sections used in the beam elements consists of patches simulating concrete slabs, steel webs and flanges, as shown in Figure 6. Considering that the real concrete slab does not perfectly act with steel section in a plane-remain-plane manner as that in numerical model does, concrete patches in the numerical model were smaller than they actually were. The effective height and width of concrete slab were assumed to be 0.1 and 0.6 m, respectively. There were 20 (20 by 1) integration points distributed along each web or flange patch, and 36 (6 by 6) integration points in each concrete slab patch. OpenSees Steel02 and Concrete01 material models were used for the steel and concrete patches, respectively. The material properties were tuned to roughly match the component test result. To match the component test result, the steel Young's module and strength were set to 160 GPa and 320 MPa, respectively, while the concrete compressive strength and strain were -34.3Mpa and -0.004, respectively. Figure 7 shows the comparison hysteresis loops of real beam test (black) and numerical beam test (blue).

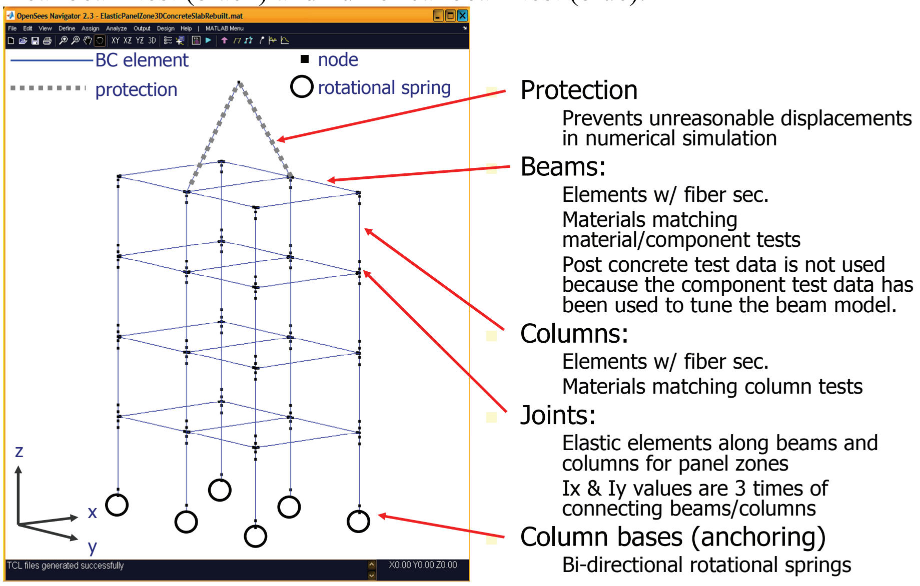


Figure 5 Finite element model by OpenSees.

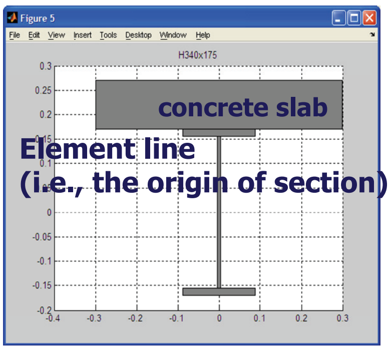


Figure 6 Fiber section for beam section type: H340x175.

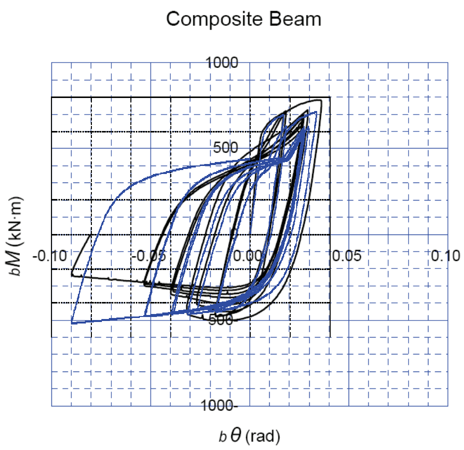


Figure 7 Matching numerical beam to tested beam.

Each column of each floor was modelled by three displacement-based beam-column element, each with five integration points. Considering the plastic hinges typically occur at two ends of a column, the two end elements were only 0.3 m long while the middle element was much longer. Fiber sections used in the column elements consists of patches simulating steel sections. There were 20 (20 by 1) integration points distributed along each side of a steel section. OpenSees Parallel material encapsulating a Steel02 model and a hysteretic model was used to simulate the complicated nonlinear local buckling behaviors. The material properties were tuned to roughly match the column test result. To match the component test result, the steel Young's module and strength were set to 126.7 GPa and 160 MPa, respectively, while the three controlling points of the hysteretic were 200MPa/0.0021, 215MPa/0.011, 0/0.015, respectively. Figure 8 shows the comparison hysteresis loops of real column tests (black) and numerical column tests (blue).

The beam-column connections were modelled by linear-elastic beam-column elements as shown in Figure 9. It was assumed that dominant plastic hinges occur near but not within connections, therefore, the numerical models of the connections were set to be stronger than their connecting beams and columns. The I and J values of the beam-column elements are three times of their connecting beams or columns.

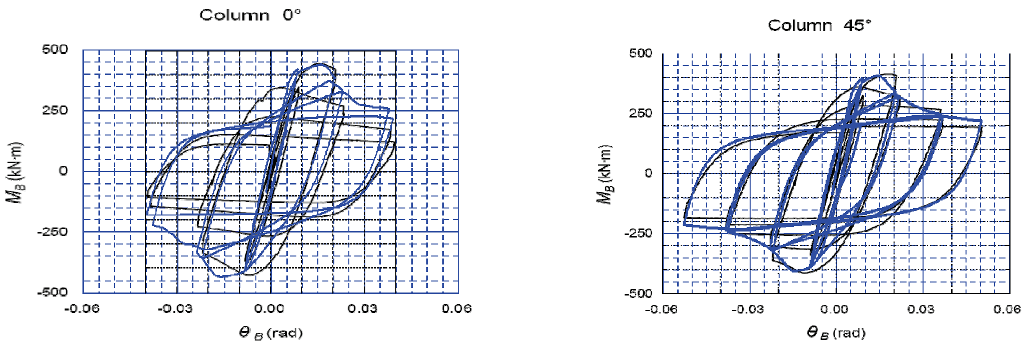


Figure 8 Matching numerical column to tested column.

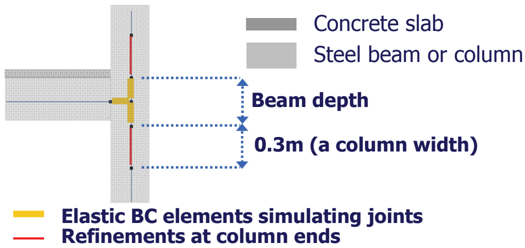


Figure 9 Beam-column connection model.

A lumped mass node was placed around the center of each floor. Eccentricity of each mass point was calculated considering the nominal masses of construction material and unsymmetric deployment of walls. The height of masses are the same as the central lines of beams, ignoring that the actual height of the equivalent mass point should be higher. Six degrees of freedom of each lumped mass was calculated considering the translational and rotational effects based on the rigid-diaphragm assumption. It is also assumed that masses of all objects were rigidly connected with the mass point of each floor. The column bases were modelled by rotational springs. It was assumed that the column bases provides sufficient translational and torsional stiffness and can be considered as rigid. The rotational stiffness of 4.89×10^7 kN-m/rad suggested by a simulation result was adopted for the springs along both horizontal directions. The numbers of elements and nodes are 242 and 211, respectively, making the total number of variables around 1200. A direct sparse solver with diagonal pivoting is adopted for solving linear equations. Computation was carried out on a desktop personal computer with a single Pentium 4 CPU 3.4GHz with 1.5GB main memory.

3.2 Time-history Analysis of the Frame

Newmark's average acceleration method with adaptive time increment of 0.005 sec. was used for time integration. Modified Newton's method with convergence criterion of 10^{-6} of applied effective force were applied for nonlinear iteration. Rayleigh's damping coefficients were calculated based on assumption of damping ratio of 3% between its 1st and 3rd natural periods. The three lowest natural periods (sec.) are 0.853, 0.810 and 0.579.

The actual acceleration records in three directions measured on the shake table during the series of tests were used as the input acceleration of ground motion. The ground motion consisting of three tests was applied at a time. The time period of 0 to 40.96 seconds represents the first test; while 49.96 to 81.92 seconds and those afterward represents the 2nd and 3rd tests, respectively. Time histories of the 1st-story drift angles in X- and Y-directions are plotted in Figure 10. The CPU time for this analysis were around 3700 seconds.

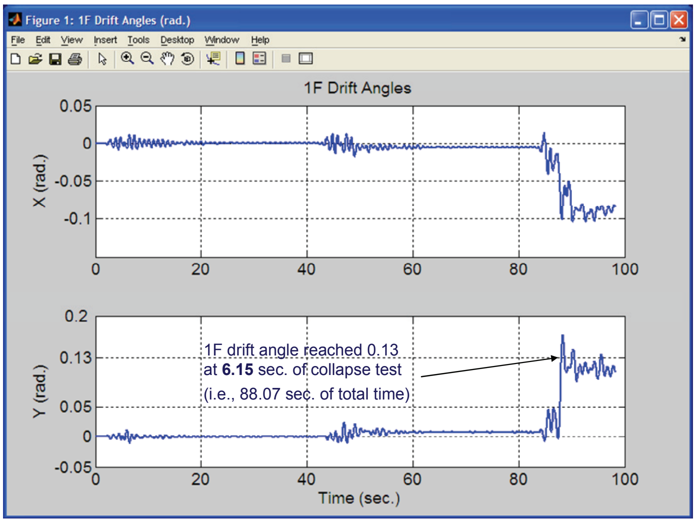
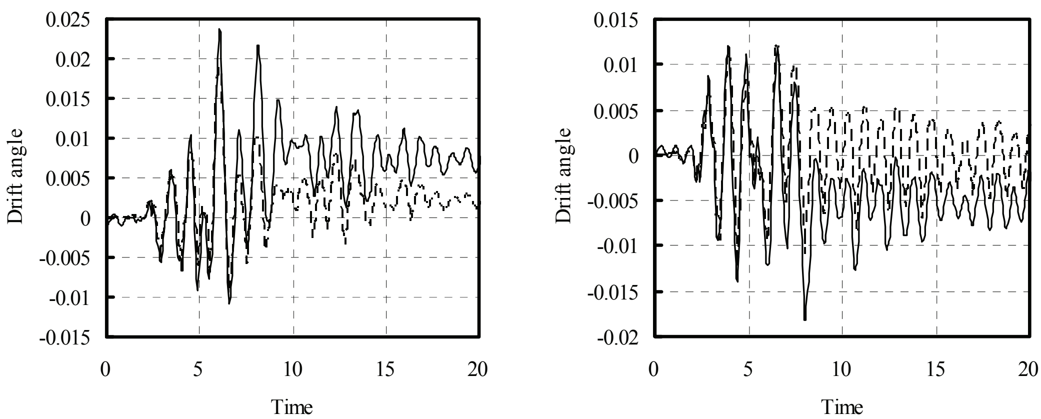


Figure 10 Time history of drift angles of the 1st story.

Figure 11 shows time history of drift angle of the 1st story under target 60% Takatori wave. The computed maximum absolute values of drift angles in X- and Y-directions are 0.018 and 0.024, respectively, which overestimated the results 0.012 and 0.019 of experiments. However, the phases and the amplitudes are accurately simulated by analysis.



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

Figure 11 Time history of drift angles of the 1st story for 60% Takatori.

4. AWARD-WINNING RESULTS BY PRACTICING ENGINEER

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  Maeda Corporation , Japan

4.1 Outline of the analytical model

It is assumed that the factors causing the collapse in this shake-table test are the following two phenomena. The first phenomenon is the deterioration caused by local buckling of the column. The stress-strain relationship of the steel material considering local buckling is modeled depending on the short column compression tests of box-section steel by Yamada *et al.* (2002). Section analysis using this stress-strain relationship is conducted under constant axial load, and the obtained moment-curvature relationship is converted into tri-linear skeleton.

However, when this stress-strain relationship is applied at all sections, the deterioration is overestimated for a bending member. So the stiffness deterioration ratio is taken as 1/2 of the original value. The skeleton of a column is defined using static pushover analysis of 3-D frame with positive and negative loading considering varying axial force. An example of the 1st-story column in compression is shown in Figure 12. As it was thought that the influence of the strain rate dependency could not be neglected, and the increase of the bearing force is assumed to be 20 percent as observed in the study of Yamada et al. (2002).

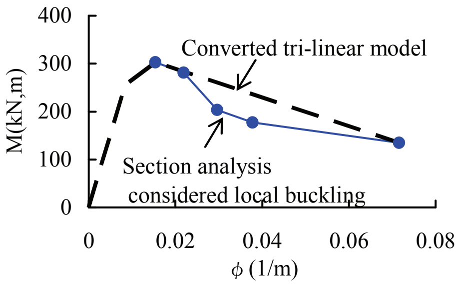


Figure 12 M-Φ relation of column
（Compressive Column of 1F, N=0.2Ny）.

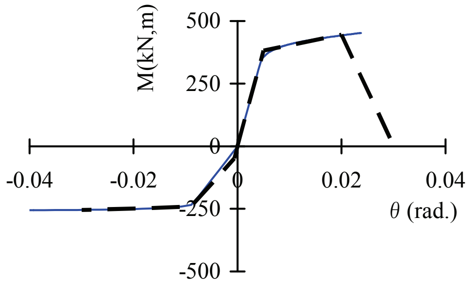


Figure 13 M-θ relation of beam
(H350×175).

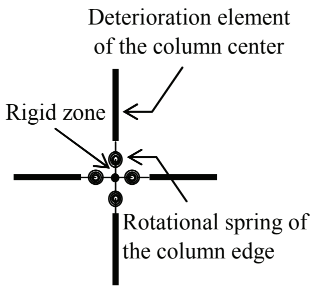


Figure 14 Connection model.

The second phenomenon is the fracture of the lower flange in a composite beam caused by strain concentration. It is assumed that the lower flange will be broken at 0.02 rad., and the resisting force would be lost at 0.03 rad. as observed in the tests conducted by Okada et al. (2001). From reverse-symmetric analysis using multi-fiber model, the result is replaced with tri-linear skeleton under above-mentioned rules. The modeling example is shown in Figure 13.

A general-purpose elasto-plastic analysis program CANNY (Li Kangning, 1993) is used for the analysis. Bending behavior of columns and beams are modeled using one-component model or multi-fiber model in this program. Since the deterioration of a steel member cannot be expressed by the prepared history rule, one member is expressed using three elements as shown in Figure 14. The rotational spring at the edge of the member is expressed by tri- or bi-linear skeleton which does not consider deterioration, while the center element is expressed by bi-linear skeleton considering deterioration. The conversion method from one member skeleton to the skeleton of 3 elements is illustrated in Figure 15. The history rule of an individual element obeys the Ramberg-Osgood curve. An example of 1st-story column under cyclic loading is shown in Figure 16.

Nodes are installed on the rigid zones of the edge of columns and beams. The column-beam connection is assumed to be rigid, and the base of 1st-story columns are assumed to be fixed.

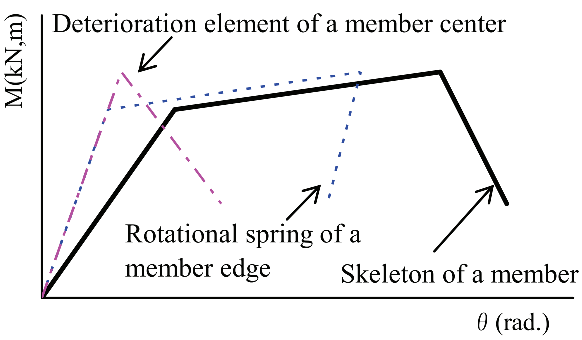


Figure 15 Conversion from member skeleton to element skelton.

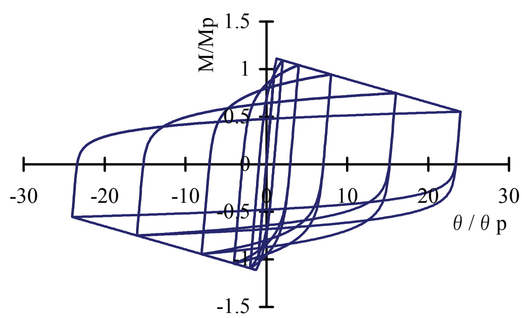
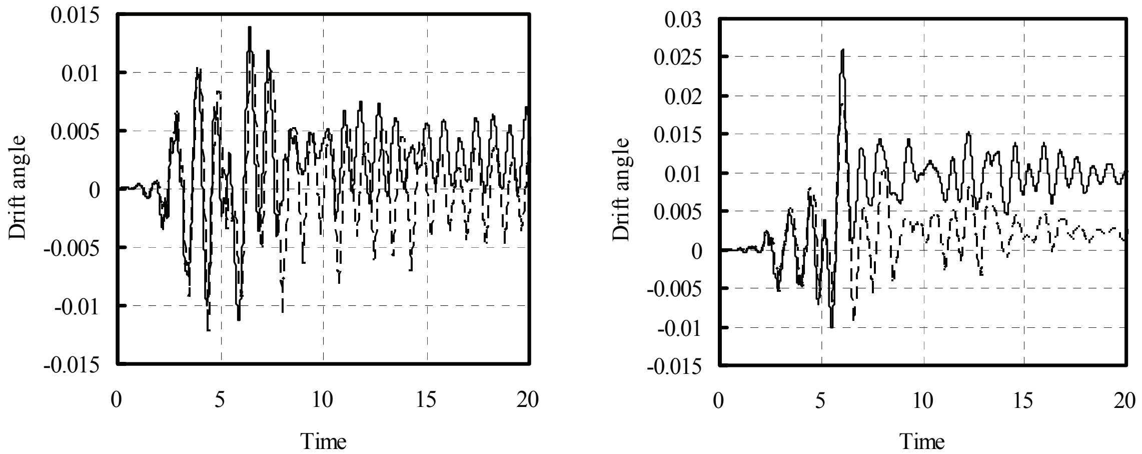


Figure 16 An example of 1st story column under cyclic loading .

4.2 TIME-HISTORY ANALYSIS OF THE FRAME

The three lowest natural periods (sec.) are 0.809 (X_dir.), 0.785 (Y_dir.), 0.741 (Tor.). Rayleigh damping is adopted, where the damping ratio is 0.02 for the first and the third mode. The input acceleration of three directions is applied on the base of the 1st-story columns, and Newmark-β method (β = 1/4) is used. The time interval is 0.005 sec. Time histories of the 1st-story drift angles in X- and Y-directions are plotted in Figure 17. In X-direction, the computed maximum absolute values is 0.0133, which is close to the results 0.012 of experiments. On the other hand, in Y-direction, the computed maximum absolute values is 0.0259, which overestimates 0.019 of experiments; however, the wave form of the computed X- and Y-directional response are similar to the measured ones.



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

Figure 17 Time history of drift angles of the 1st-story for 60% Takatori.

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