ANALYTICAL STUDIES OF A HALF-SCALE 3-STORY NON-SEISMIC DETAILING REINFORCED CONCRETE BUILDING SHAKEN TO NEAR-FAULT EARTHQUAKES

Y. T. Weng¹, S. J. Jhuang² and C. C. Chou³

ABSTRACT

A half-scale three-story reinforced concrete frame building was shaken to collapse the shaking table in NCREE Tainan Laboratory in Taiwan in August 2017. This building was used as a case study to investigate modeling techniques for near fault effects on building structures, nonlinear structural simulations. The analytical studies suggest that the fiber-model beam-column element considering RC column effects simulates the responses of the three-story building specimen. Moreover, considering the effects of the stiffness reduction of the RC beam and column members can further enhance the accuracy of the response simulations. The RC column model incorporating fiber-beam–column element using the stress-strain relation for monotonic loading of confined and unconfined concrete - proposed by Mander et al. (1988b) in this study can satisfactorily simulate the degrading force versus deformation relationships of the RC column model. A frame model incorporating the same type of fibered column element could predict the key maximum experimental responses of the test building. Based on these nonlinear response analyses, it is illustrated that the responses of the building can be estimated satisfactorily by using the modeling techniques presented in this study.

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A half-scale three-story reinforced concrete frame building was shaken to collapse the shaking table in NCREE Tainan Laboratory in Taiwan in August 2017. This building was used as a case study to investigate modeling techniques for near fault effects on building structures, nonlinear structural simulations. The analytical studies suggest that the fiber-model beam-column element considering RC column effects simulates the responses of the three-story building specimen. Moreover, considering the effects of the stiffness reduction of the RC beam and column members can further enhance the accuracy of the response simulations. The RC column model incorporating fiber-beam–column element using the stress-strain relation for monotonic loading of confined and unconfined concrete - proposed by Mander et al. (1988b) in this study can satisfactorily simulate the degrading force versus deformation relationships of the RC column model. A frame model incorporating the same type of fibered column element could predict the key maximum experimental responses of the test building. Based on these nonlinear response analyses, it is illustrated that the responses of the building can be estimated satisfactorily by using the modeling techniques presented in this study.

Introduction

Experimental tests serve two purposes in structural engineering. First, they serve to provide a database for the calibration of analytical and numerical models. For example, by taking advantage of the remarkable advancements in computer technology, general-purpose structural analysis computer programs have been developed for the analysis of linear and nonlinear responses in both the static and dynamic domains for large structural systems. Before these computer programs can be utilized with confidence, it is vital that high-quality test data be available for calibration. Second, structural testing has played a very important role in the development of design rules for code implementation in the past, and this role is expected to expand with the development of performance-based engineering. In particular, experiments on full scale structures incorporating realistic boundary conditions and loading conditions are essential for the implementation of the aforementioned earthquake resistant technologies.

In August, 1999 an earthquake with a magnitude of 7.3 occurred in Taichung, Taiwan. A total of about 2,300 people in the surrounding area lost their lives. The earthquake also caused huge

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economic losses and left a lot of homeless. As a result, in 2014 the National Center for Research on Earthquake Engineering (NCREE) started the construction of The NCREE Tainan Laboratory which is composed of a research building and a laboratory building, is located in Tainan City. It features a high performance seismic simulation testing system, and a biaxial dynamic testing system (BATS). In addition to these facilities, a reaction wall and strong floor test system is constructed as well. The dimension of the reaction wall is 12m (height) x 15m (width) x 5m (thickness). It was completed in 2017. In August 2017, the NCREE-Tainan shaking table was utilized for experimental tests of a half-scale three-story RC building shaken to collapse. Before the tests were executed. For the purposes of enhancing the structural modeling techniques, the analyses made after the tests using a general purpose nonlinear structural analysis program PISA3D incorporating with fiber elements.

Tests of a Half-scale Three-story RC Frame Specimen in NCREE-Tainan Lab.

The photo of the half-scale three-story RC building are shown in Fig. 1. In order to predict the key maximum experimental responses of the specimen under the two-directional elastic-inelastic level shaking. Before the three-story frame tests, cyclic loading test results of two column specimens were also provided by NCREE to gain a better insight into the force-deformation characteristics of individual structural members.

Introduction of the test frame

The photo of the 1/2-scale three-story RC building are shown in Fig. 1. 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 3.5 m each in the Y direction and one-bay of 3.5 m in the X direction. Each story height is 1.5 m except the first story which is 3.0 m. The thickness of the concrete slab is 100 mm for the each floor. The weight and story height of the frame for each floor is given by the organizer and is listed in Table 1. It includes floor slabs, RC frame, exterior walls, additional steel plates and the safeguard system. The test specimen had a 0.73 m thick posttensioned RC foundation designed to remain linear elastic during all seismic tests. The building was anchored to the shake table through posttensioned rods installed along the foundation perimeter, providing the fixed-base test configuration during the all phases of the seismic testing. The steel material is SD420W and SD280W for the frame beams and columns. Tables 2 and 3 show material properties used in the frame specimen. The average cylinder strength of concrete obtained from the concrete cylinder samples was 23.3 N/mm². The D10 and D19 reinforcing bar had average yield strengths of 349.7 N/mm² and 454.3 N/mm², respectively. The sizes of the beams and columns are summarized in Fig. 3. The building was designed following the Taiwan specifications and practices before 1999.

Figure 1. Views of the 3-story RC building specimen.
Table 1. Floor weight and story height

<table>
<thead>
<tr>
<th>Floor number</th>
<th>Floor weight (kN)</th>
<th>Story</th>
<th>Story height (mm)</th>
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</thead>
<tbody>
<tr>
<td>4</td>
<td>170.99</td>
<td>3</td>
<td>1500</td>
</tr>
<tr>
<td>3</td>
<td>193.26</td>
<td>2</td>
<td>1500</td>
</tr>
<tr>
<td>2</td>
<td>205.23</td>
<td>1</td>
<td>3000</td>
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</tbody>
</table>

Table 2. Concrete properties

<table>
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<tr>
<th>Specimen</th>
<th>Compressive strength (N/mm$^2$)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>1$^{st}$ story and 2$^{nd}$ story</td>
</tr>
<tr>
<td>Specimen 1</td>
<td>23.8</td>
</tr>
<tr>
<td>Specimen 2</td>
<td>22.5</td>
</tr>
<tr>
<td>Specimen 3</td>
<td>23.7</td>
</tr>
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</table>

Table 3. Steel properties

<table>
<thead>
<tr>
<th></th>
<th>D10</th>
<th></th>
<th>D19</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_y$ (MPa)</td>
<td>$f_u$ (MPa)</td>
<td>$f_y$ (MPa)</td>
<td>$f_u$ (MPa)</td>
<td></td>
</tr>
<tr>
<td>348</td>
<td>479</td>
<td>456</td>
<td>642</td>
<td></td>
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<td>348</td>
<td>466</td>
<td>452</td>
<td>639</td>
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<tr>
<td>353</td>
<td>478</td>
<td>455</td>
<td>644</td>
<td></td>
</tr>
</tbody>
</table>
Cyclic loading test of reinforced concrete columns

Before the three-story frame tests, cyclic loading test results of two column specimens which are the same as the column C1 and C2 respectively, were also provided by NCREE to gain a better insight into the force-deformation characteristics of individual structural members. Two column specimens with the same structural property (aspect ratio is about to 8.66) were constructed and tested under cyclic loading. Fig. 4 show the details of the two column specimens which are labeled as Column C1 and Column C2, respectively. Longitudinal reinforcement D19 for these two column specimens were characterized by a yield strength $f_y$ of 454 MPa. The transverse reinforcement of all specimens comprised of steel stirrup D10 was characterized by a yield strength $f_y$ of 355 MPa. The average compressive strength of concrete, $f_c$, obtained from the concrete cylinder samples, was found to be 22.0 MPa for both Specimens C1 and C2. These two specimens C1 and C2 are subjected to axial force $0.12 Af_c$ and $0.06 Af_c$, respectively.

![Figure 4. Dimensions and reinforcing details of Specimens C1 and C2 (in cm)](image)

Fig. 5 shows the test setup. The test specimens were subjected to quasi-static load reversals that simulated earthquake loading. The bottom of the column was fixed to the strong floor of the laboratory and the ends of the column was connected to this strong floor by RC links which was not permitted rotations and free horizontal translations of the column, and thus provided the vertical reactions to the column. The cyclic loading history showing applied cycles versus the drift ratio is shown in Fig. 6.
Experimental Results and Observations

Figure 6. Cyclic horizontal loading history

Figure 7. Comparison of hysteretic behavior between the experimental and the analytical results:
Specimen C1

Figure 8. Observed cracking of Specimen C1 at the (a) start, (b) drift ratio of ±3%, and (c) drift ratio of ±8%

The shear force versus horizontal displacement relationship of Specimen C1 is shown in Fig. 7. In the loading directions, the maximum capacity attained was approximately 96.9 kN, corresponding to a drift ratio of 3% as illustrated in Fig. 8b. When the specimen was loaded to a drift ratio of ±0.5%, the column experienced the initiation of flexural cracks. In the subsequent loading run corresponding to drift ratios of 1.0 and 2.0%, the specimen developed some flexural cracks along the column. At a drift ratio of 8%, the specimen experienced a substantial loss of more than 24%
of its load-carrying capacity, which led to the test being stopped. At this stage, spalling of concrete at the column end due to the opening of flexural cracks and at the column compression zone due to crushing was observed as illustrated in Fig. 8c.

**Introduction of the PISA3D program**

The Platform of Inelastic Structural Analysis for 3-D Systems (PISA3D) (Lin et al. 2009) 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

PISA3D is based on the small displacement theory. In a PISA3D analysis, nodal coordinates of the deformed structure are not updated. However, the geometric nonlinear effects are considered by adding geometric stiffness matrices into the element stiffness matrices. PISA3D adopts the Newmark scheme for transient response analysis. In each time step, the updated element stiffness at the beginning of each time step is used to calculate the responses. However, the approach is non-iterative and any unbalanced forces resulting from the element stiffness change in any time step are incorporated into the external loads in the next time step. In this paper, PISA3D analytical results are adopted to present the maximum responses and the collapse time simulations of the frame specimen.

**Basic models**

A basic analytical frame models with different modeling techniques for beams and columns are presented herein. In the basic model, fiber-model beam-column element were adopted for all the column members, respectively. All the frame mass is considered as a lump mass located at the top of the slab of each floor in these basic models. However, the location of the mass is shown in Figure 2a and Table 1 show the estimated mass computed from the information provided by the organizer. The Newmark method of constant average acceleration scheme (β=1/4) is used for the time integration. According to the information provided by the organizer, Rayleigh damping is adopted, and the damping ratios are assumed to be 4.97% for the first and second modes of each 3D model.

![Figure 9. Stress-strain relation for monotonic loading of confined and unconfined concrete - Mander et al. (1988b)](image1)

![Figure 10. Stress-strain relation for monotonic loading of steel bar](image2)
Fiber-model RC column element (FC)

The FC model adopted fibered beam-column element to represent the RC column. The fibered beam-column element in 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 (Spacone et al. 1996; Neuenhofer and Filippou 1997).

Figure 11. The stress versus strain relationships of the concrete and steel bar for the Specimen C1

As Figure 11a shown, the steel longitudinal reinforcement consists of 8 fibers using the steel yield strength provided by the organizers for the degrading steel material which utilizes the steel coupon test to obtain the stiffness and post-yield stiffness ratio as shown in Figure 10. The “stress versus strain” relationship of the concrete provided by the organizer was used for concrete fibers. The concrete material follows the recommendations provided by Mander et al. (1988b). According to ASCE/SEI 41-13 (2013), the initial stiffness is defined considering flexural and shear deformations of beams and columns. Joint deformations were ignored in ASCE/SEI 41-13 (2013) guidelines. The flexural rigidity is defined as $0.3E_c I_g$ for RC columns with low axial loading, and shear stiffness for rectangular cross sections is defined as $0.4E_c A_w$ as suggested by ASCE/SEI 41-13 (2013) guidelines. However, measured effective flexural rigidity for low axial loads could be
approximated as $0.2E_{c,0}$ for RC columns. Considering the concrete linear elastic modulus $E_c$ of concrete material was reduced to $0.2E_c$, the corresponding strain at the maximum compressive strength of confined concrete $\varepsilon_{cc}$ has to be modified to $5\varepsilon_{cc}$ in the Mander’s stress versus strain relationship. The experimental initial stiffness of the specimen Column C1 was estimated using the secant of the horizontal shear force versus the horizontal displacement relationship passing through the point at which 75% of the theoretical strength was obtained. The initial stiffness of the specimen Column C1 obtained from the cyclic loading test and analytical results computed from PISA3D using fiber-model were 2.40 kN/mm and 2.76 kN/mm, respectively. The unloading/reloading responses according to the work of Karsan and Jirsa (1969) are shown in Fig. 11b. Five integration points along the fibered beam-column element were chosen to integrate the element responses. This fibered column modeling could be viewed as the best “blind prediction” an engineer could do without having to use the subassembly test results. However, utilizing the cyclic loading test results described as above, a more realistic effective stiffness could be obtained from calibrations. The results are incorporated into the refinement of the frame model as described below.

**Comparison between Analytical Simulation and Test Results**

According to the above-mentioned different modeling techniques for beams and columns, the corresponding analytical frame models were constructed. Figure 7 shows that analysis using the fiber-model RC column element had well agreement with cyclic loading test in hysteretic response of Column C1.

**NONLINEAR DYNAMIC TIME HISTORY ANALYSIS**

Each nonlinear analysis starts by applying the gravity loads quasi-statically and incrementally. The regular incremental iterative Newton method is used to solve the nonlinear static equilibrium equations for gravity loads. The nonlinear time history analysis for earthquake base excitation is then performed from the state of the structure after application of the gravity loads. Newmark average acceleration method (Chopra, 2011), with a constant time step of 0.005 sec, is used to integrate the equations of motion in time. The constant time step of 0.005 sec is selected based on a preliminary convergence study with respect to the integration time step size to ensure the accuracy of the results. The quasi-Newton (secant) method based on the Broyden–Fletcher–Goldfarb–Shanno (BFGS) stiffness update method (Matthies and Strang, 1979) is employed as the iterative method to solve the nonlinear dynamic equilibrium equations. At the end of each time step of analysis, the last obtained secant stiffness matrix is stored and used as the initial stiffness matrix at the first iteration of the next time step. The convergence criterion is based on the relative norm of the last displacement increment vector or the relative norm of the last unbalanced force vector with a convergence tolerance of $10^{-4}$, whichever occurs first, while the number of iterations per time step is limited to 30. If none of the two convergence criteria are satisfied within 30 iterations, the iterative procedure at that time step is terminated, the current unbalanced force vector is transferred to the next time step, and the analysis goes on.

The NLRHA was conducted for the elastic-level (CHY047EL: PGA=0.42g; TCU052EL: PGA=0.35g) and the inelastic-level (TCU052In1: PGA=0.8g; TCU052In2: PGA=1.0g) earthquakes.
Seismic response at elastic level

The key responses of the experimental and the analytical seismic responses are presented and compared. The peak story displacements and the peak story accelerations under the application of the CHY047EL of elastic-level are shown in Figures 14a, and 14b respectively. Figure 14a shows peak roof displacement reached 43.6 mm and 31.5 m for the experimental and analytical seismic responses, respectively. Figure 14b shows peak roof accelerations of the experimental and analytical seismic responses reached 1320.1 and 742.0 cm/sec², respectively.

The peak story displacements and the peak story accelerations under the application of the TCU052EL of elastic-level are shown in Figures 15a, and 15b respectively. Figure 15a shows peak roof displacement reached 26.7 mm and 25.0 m for the experimental and analytical seismic responses, respectively. Figure 15b shows peak roof accelerations of the experimental and analytical seismic responses reached 848.8 and 509.8 cm/sec², respectively.

Seismic response at inelastic level

The key responses of the experimental and the analytical seismic responses are presented and compared. The peak story displacements and the peak story accelerations under the application of the TCU052In1 of elastic-level are shown in Figures 16a, and 16b respectively. Figure 16a shows peak roof displacement reached 59.4 mm and 64.1 m for the experimental and analytical seismic responses, respectively. Figure 16b shows peak roof accelerations of the experimental and analytical seismic responses reached 1748.4 and 1152.0 cm/sec², respectively.

The peak story displacements and the peak story accelerations under the application of the TCU052In2 of elastic-level are shown in Figures 15a, and 15b respectively. Figure 15a shows peak roof displacement reached 92.5 mm and 97.1 m for the experimental and analytical seismic responses, respectively. Figure 15b shows peak roof accelerations of the experimental and analytical seismic responses reached 1809.8 and 2070.0 cm/sec², respectively.

Figure 14. (a) Roof displacement time histories and (b) Roof acceleration time histories in X dir. under the CHY047EL earthquake event

Figure 15. (a) Roof displacement time histories and (b) Roof acceleration time histories in X dir. under the CHY052EL earthquake event

Figure 16. (a) Roof displacement time histories and (b) Roof acceleration time histories in X dir. under the CHY052In1 earthquake event
Summary and Conclusions

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

- The cyclic force-asymmetric hysteresis responses of the RC column specimen can be much better simulated by using the fibered beam-column element than traditional multi-linear plastic hinge model. Figure 7 shows that analysis using the fiber-model RC column element had well agreement with cyclic loading test in hysteretic response of Column C1.

- For fibered beam-column element, considering the concrete linear elastic modulus $E_c$ of concrete material was reduced to $0.2E_c$, the corresponding strain at the maximum compressive strength of confined concrete $\varepsilon_{cc}$ has to be modified to $5\varepsilon_{cc}$ in the Mander’s stress versus strain relationship.

- The Mander et al. (1988b) model has enjoyed widespread use in design and research. Notwithstanding this it has several shortcomings. Since the original tests were developed in the 1980’s, there has been a marked upsurge in the use of high performance (strength) materials, in particular high strength concrete. The Mander et al. (1988b) model does not handle the post-peak branch of high strength concrete particularly well and requires some modification.

References

1. ASCE/SEI 41-13 (2013). Seismic Evaluation and Rehabilitation of Existing Buildings, America Society of Civil Engineers, Reston, V.A.


