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COMPARISON OF DETAILED AND SIMPLIFIED ANALYTICAL MODELS WITH SHAKE TABLE TEST FOR RC FRAMES

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ABSTRACT

Performance-based design methodologies require a thorough understanding of the lateral and vertical load behavior of the structural elements for all levels of performance, including collapse prevention. It is, therefore, important to assess the accuracy of analytical models used to predict the collapse behavior of structural elements. The current paper presents analyses that simulate a shake table test performed at the National Center for Research on Earthquake Engineering (NCREE) in Taiwan. In order to validate and compare existing analytical models for non-ductile concrete buildings, a blind comparison of the test data and analytical approach is performed. This comparison illustrates the limitations, weaknesses, and strengths of the analytical models. Refining and validating the analyses with the existing experimental data provides the opportunity to compare the comprehensive modeling with the common simplified methods available in engineering practice. Effects of modeling parameters such as nonlinear columns without shear or axial springs, elastic columns with rotational springs at the ends are discussed here. The results from analysis employing modeling parameters suggested by FEMA356 are also compared with the shake table test data. Such studies will lead to a better prediction of the behavior of existing reinforced concrete structures, and consequently, more cost-effective retrofit strategies.

Introduction

Moment-resisting frames are one of the most common structural systems used for building and industrial structures. Reinforced concrete frames constructed before or during the 1970s, when modern seismic design codes were first introduced in many industrialized countries worldwide, were primarily designed and detailed to resist gravity loads and the amount of transverse steel to resist shear and provide confinement in the columns was very limited. It has been demonstrated that the behavior of reinforced concrete structures is very dependent on the amount and distribution of the longitudinal and transverse steel. Consequently, these older buildings can exhibit brittle behavior and may lack the lateral strength and ductility necessary to withstand an earthquake without damage, or potentially collapse.

With the recent efforts to develop performance-based seismic design methodologies, and in order to strengthen susceptible reinforced concrete structures against seismic loading, it is important to

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understand the mechanisms causing collapse in such structures under both gravity and seismic loads. Since experimental tests are often costly and time consuming, developing analytical models that can reasonably predict the behavior of structural elements up to the stage of collapse is of great interest.

The purpose of this study was to simulate the behavior of a one-third scale model of a three-bay frame with two shear-critical columns and two ductile columns. Empirical capacity models were used to predict the hysteretic response of shear-critical reinforced concrete columns under gravity and seismic loading. In particular, the shear failure and axial load collapse of these columns were closely examined. The finite element program *OpenSees*, developed by the Pacific Earthquake Engineering Research Centre, was employed to conduct the analyses. A blind prediction of the behavior of the columns was carried out through non-linear static and dynamic analyses. The model was later modified based on the data obtained from the shake table test and the refined model was then compared with the analytical models commonly used in engineering practice.

Shake Table Test

Specifications of Specimen

A shake table experiment at the National Center for Research on Earthquake Engineering in Taiwan in August 2005 was employed for this study. The test was designed to observe the process of shear and axial load failure in reinforced concrete columns subjected to ground shaking. The test specimen was composed of four columns (Fig. 1). They were fixed at their bases and interconnected by a beam at the upper level. The columns were constructed at approximately 1/3 scale. Columns A and B were designed and detailed to satisfy most current design codes for a high seismic zone, with a transverse reinforcement ratio of ρ”=1.5% in the plastic hinge zone. In contrast, columns C and D, with ρ”=0.2%, were typical of columns designed without seismic considerations, and hence were considered vulnerable to shear failure and subsequent axial load failure during testing. The connecting beam and footings were made relatively stiff and strong to remain elastic during testing.

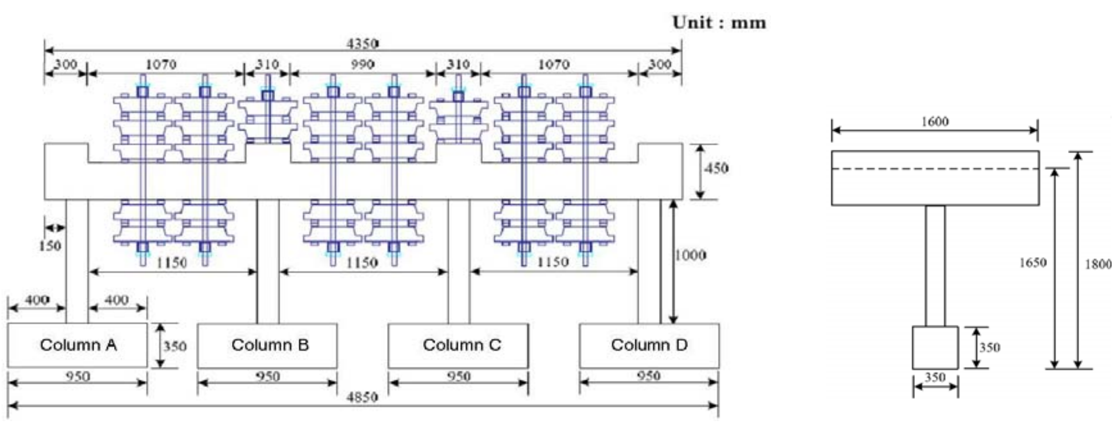


Figure 1. Dimension of three-bay frame and loading on the beam.

Loading

The total weight of the lead packet stacks, used as gravity load on the beam (Fig. 1), was 37.5 Kips. Sixty eight lead packets were used, with each packet weighing 550 Pound. The distance between supports of each packet was 9.85 inch to distribute the load as a series of point loads on the beam and the top of columns B and C. Table 1 shows the loads on the specimen as well as the weight of the elements.

Table 1. External load on the structure and weight of the elements.

	Lead Packet stacks	**Beam**	**Columns**	**Footings**	**Total**
Weight (Kips)	37.5	12.6	0.49	2.47	53.06

Material Properties

The stress-strain model developed by Mander et al. (1988) was used to determine the constitutive relationship for confined and unconfined concrete for the columns. The steel with parabolic strain hardening model was employed for reinforcement steel. The material properties used in the frame analysis are shown in Tables 2 and 3. Although columns C and D were not well confined, the light transverse reinforcement resulted in a slightly higher concrete strength in the core concrete compared to the cover concrete.

Table 2. Properties of steel used in section analysis of the columns and beam.

	6 mm	#3	5 mm	3.2 mm
Steel Yield Strength (ksi)	33.6	68.3	96	79.5
Steel Ultimate Strength (ksi)	55.3	102.7	100.3	83.8

Table 3. Properties of concrete used in section analysis of the columns and beam.

	Beam	Column A	Column B	Column C	Column D
Concrete Strength on Test Day (psi)	5139	4683	4683	4683	4683
Confined Concrete Strength, f'cc (psi)	--	7785	7785	5060	5060
Compressive strain at f'cc , εcc	--	0.006	0.006	0.002	0.002

Instrumentation

The instrumentation consisted of force transducers (load cells), displacement transducers (Temposonics), and accelerometers. The force transducers were used to measure the axial loads and the shear forces in the columns. Displacement transducers (Temposonics) were utilized to measure the global vertical and horizontal displacements of the mass and local deformation of the columns. Accelerometers were used to measure the vertical and horizontal acceleration of the mass.

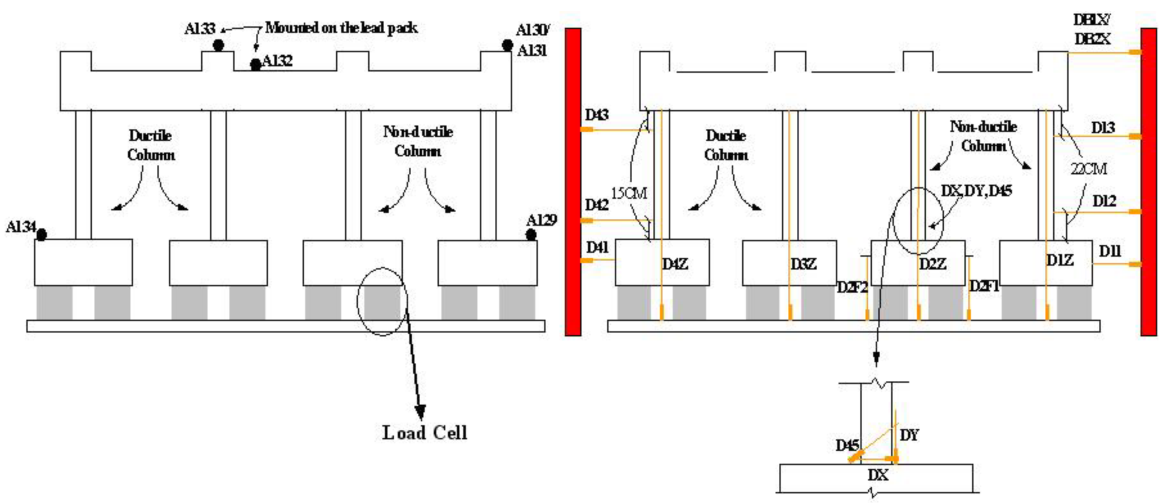


Figure 2. Location of Accelerometers and Temposonics.

Table Motions

Two shake table tests were performed on the same specimen. For both tests, the specimen was subjected to a scaled horizontal ground motion recorded during the 1999 Chi-Chi Earthquake. Although the specimen did not collapse under the first table motion, it experienced permanent damage. In a subsequent test, the deformed specimen was subjected to the second table motion which caused the specimen to collapse. The same input motion was used for both tests, however, the resulting table motion varied slightly within the short period range, as shown in Fig. 3.

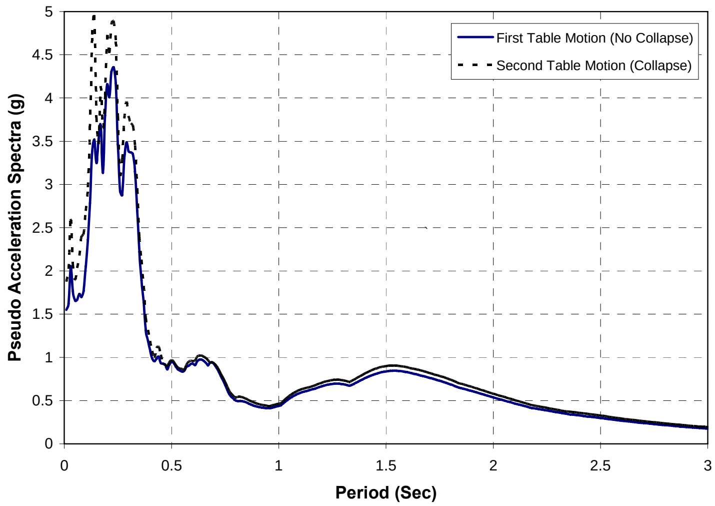


Figure 3. Pseudo-acceleration response spectra for table motions.

Analysis of the Shake Table Test

In an effort to reproduce the response of the shake table test specimen described above, an analytical model was developed using OpenSees, a finite-element analysis platform designed for earthquake engineering simulation, developed by the Pacific Earthquake Engineering Research Center. The model described below was developed without prior knowledge of the results from the shake table tests in order to provide a blind comparison of the analytical and experimental results and objectively evaluate the accuracy of the selected modeling technique.

Drift Capacity Model

To identify the initiation of shear and axial strength degradation during the analysis, the drift capacity models developed by Zhu (2005) were implemented as a Limit State Material in OpenSees. The Limit State Material uses the drift capacity model to determine the point of shear or axial failure for a column and subsequently controls the post-failure response of the element resulting in strength degradation (Elwood, 2004). The median drift capacity at shear failure can be obtained from:

$$(\delta_s)_{median} = 2.02\rho'' - 0.025\frac{s}{d} + 0.013\frac{a}{d} - 0.031\frac{P}{A_g f'_c} \quad (1)$$

where *ρ”* denotes the transverse reinforcement ratio, *P* is the axial load, A_g is the gross cross-sectional area, f'_c is the concrete strength, *s* is the spacing of the transverse reinforcement along the height of the column, and *a/d* is the aspect ratio of the column. The median prediction of drift ratio at axial failure can be obtained from:

$$(\delta_a)_{median} = 0.184\exp(-1.45\mu) \quad (2)$$

μ, the effective coefficient of friction, is given by:

$$\mu = \frac{\dfrac{P}{A_{st} f_{yt} d_c / s} - 1}{\dfrac{P}{A_{st} f_{yt} d_c / s} \dfrac{1}{\tan\alpha} + \tan\alpha} \qquad \text{where } \alpha = 65^\circ \tag{3}$$

where A_{st} is the area of transverse reinforcement in the direction of loading, f_{yt} is the yield strength of steel, and d_c is the depth of the column core measured from centre line of the perimeter tie.

Analytical Model and Results

The layout of the nodes and elements for the analytical model is shown in Fig. 4. Due to relatively large size of the connecting beam and footings, they were assumed to behave as linear-elastic members. Each column consisted of a nonlinear beam-column element and two zero-length sections located at the top and bottom of the beam-column element to attach it to the beam and footing, respectively. The top zero-length sections for non-ductile columns contained the Limit State Material models for shear and axial failure (this is shown in Fig. 4 as Zero-Length Element). Each material model could be interpreted as a spring in series with the nonlinear beam-column element, and hence, modeled the additional deformations, either shear or axial, that occurred after failure was detected by the drift capacity models described above. To account for the displacement due to slip of the longitudinal reinforcing bars from the beam and footing, elastic slip springs were included in zero-length sections at the ends of each of the column elements.

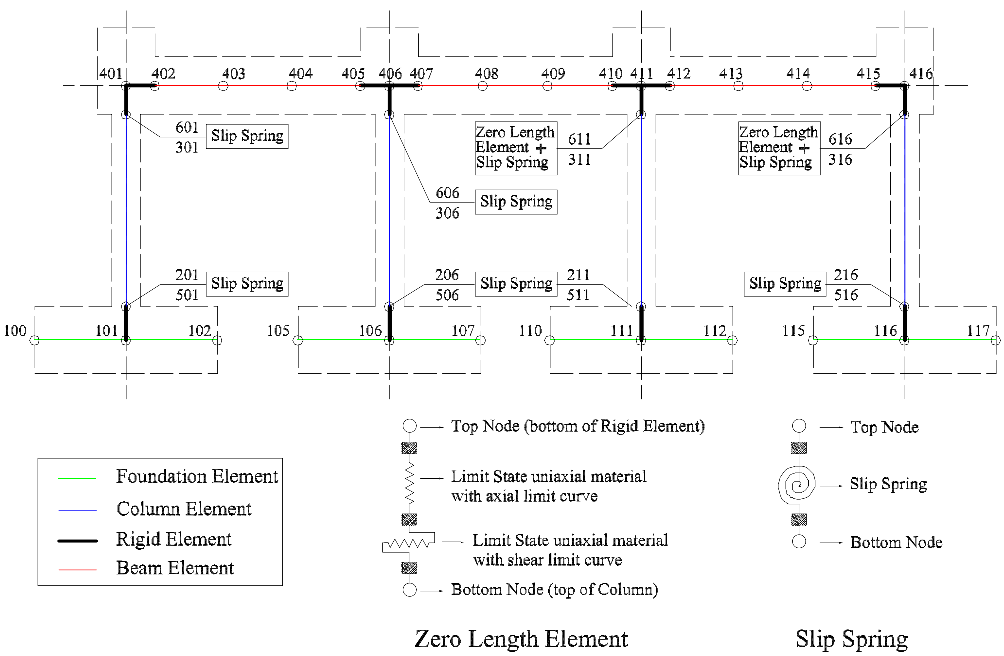


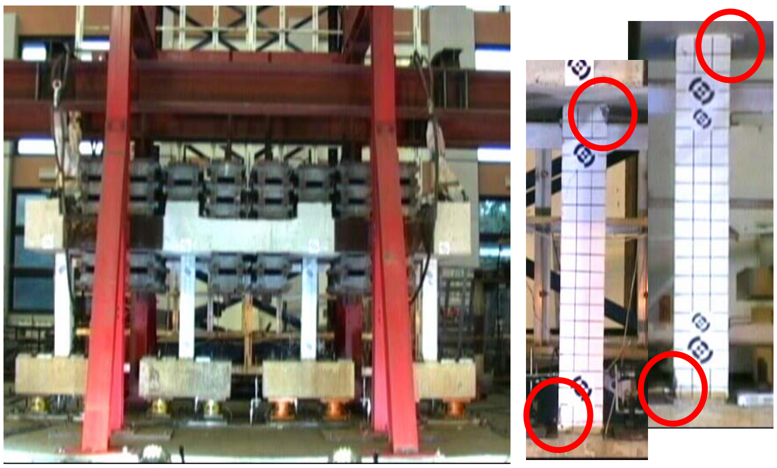
Figure 4. Model of shake table specimen.

The dead loads were distributed according to the measured weight of the lead packet stacks and the calculated weight of the beam. The lead packets load was applied as a series of point loads to the beam at the supports.

Based on the results of the analysis, the fundamental period of the structure was determined to be 0.75 second. The equivalent viscous damping was chosen as 2% of critical for the fundamental mode of the frame. Note that Rayleigh damping and stiffness-proportional damping could not be used in this model. At shear and axial failure of the zero-length springs, the response is changed suddenly which causes a large increase in velocity. This induces unrealistically large damping forces at the node connecting the springs to the beam-column element. Since no mass was modeled at this node, the increase in velocity does not influence the mass-proportional damping forces.

The model was subjected to the two sequential table motions measured during the experimental tests. To study the behavior of the structure subjected to each of the table motions, the data were recorded in two sets; referred to here as “Did not Collapse” and “Collapsed”.

Appreciating the fact that the shake table data is not available in evaluation of exciting structures, a blind prediction of the specimen behavior was conducted at the first step of the study. The results of the analyses are shown in Figs 6 to 8. It was assumed in blind prediction that the concrete cover of the columns would spall off at strain of 0.006. However, as demonstrated in Fig. 5, only small zones at the top and bottom of columns C and D experienced partial spalling during the first test. It was felt that this limited degree of spalling should not significantly affect the load-deformation response of the column; hence, spalling of the concrete was ignored in the refined model.



(a) **(b)** **(c)**

Figure 5. The shake table test after being subjected to first table motion (a) the specimen (b) cover spalling, column C (c) cover spalling, column D.

The drift histories and base shear hysteretic responses obtained from experimental test data, blind prediction, and the response of the refined analytical model are compared in Fig. 6. The analytical model generally performed very well and captured the trend of the drift and shear response in both tests. Despite the lack of agreement early in the drift ratio history for the first test, the analytical model provides a very good prediction of the drift response during the period of strongest shaking from 17 through 21 seconds. The peak drift for the first test is slightly over predicted by the model, while the peak shear is slightly under-predicted. The residual drift recorded from the experimental test was higher than what was predicted by the analytical model. This influenced the initial drift ratio for the second test in which the structure collapsed. It is observed that the refined model predicts the drift of the specimen much better than the blind prediction model. This is more noticeable for the second test. For the second test, the analytical model for blind prediction went through more cycles than the shake table specimen and detected collapse of the structure at approximately 21 seconds. Unlikely, the refined model captured the failure of the structure at 18.5 seconds in the second table motion. This is very close to the point of failure for the specimen, recorded at approximately 19 seconds in the shake table test. Although the refined model resulted in a better estimate of the drift, there is not much improvement in the estimation of the residual drift, observed at the end of the first test.

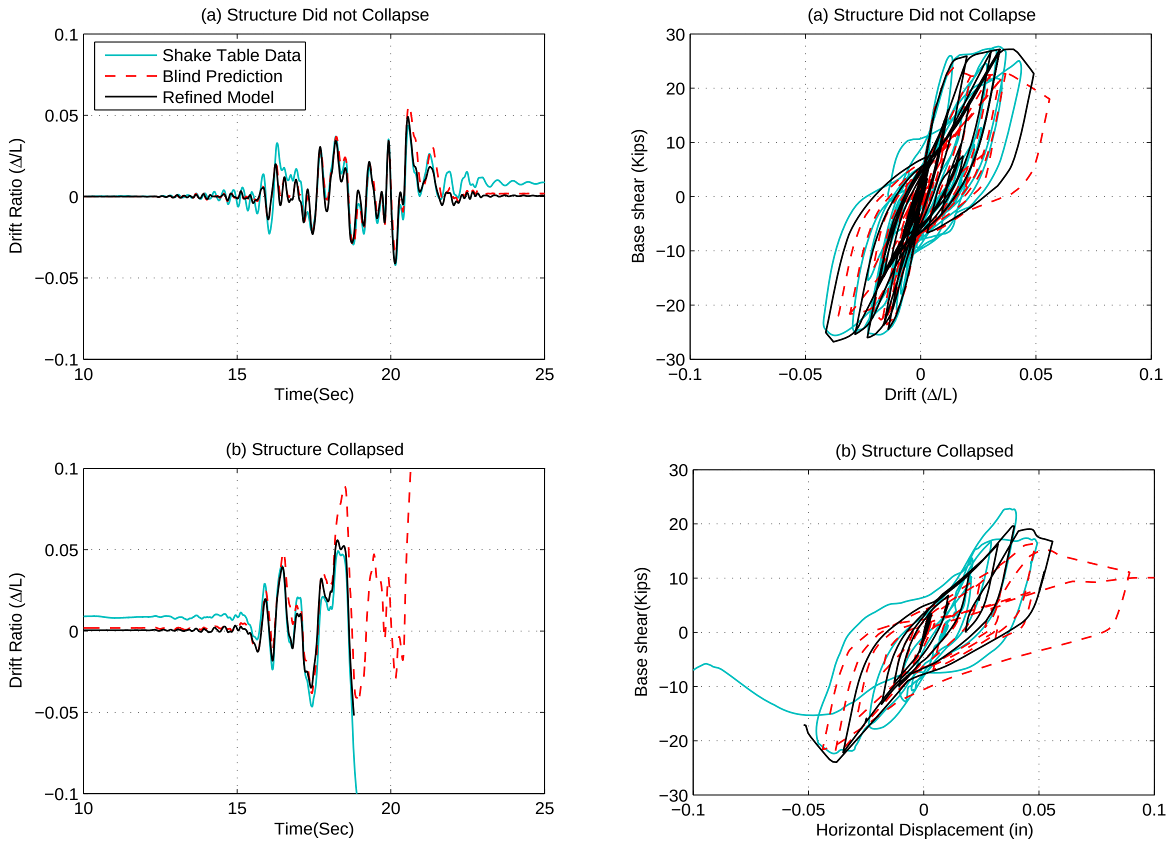


Figure 6. Drift ratio histories and base shear hysteretic responses (a) first table motion (structure did not collapse) (b) second table motion (structure collapsed).

Shear Response

During the first test, the analytical model detected shear failure for the non-ductile columns C and D. The drift at point of shear failure determined by the analytical model perfectly aligns with the one obtained from the experimental test data (Fig. 7a). Note that the detection of shear failure, and the subsequent strength degradation, contributed to the over-prediction of the drifts in the “Blind Prediction”. Limited strength degradation, accompanied by shear cracking, was observed in the experimental results for non-ductile columns C and D.

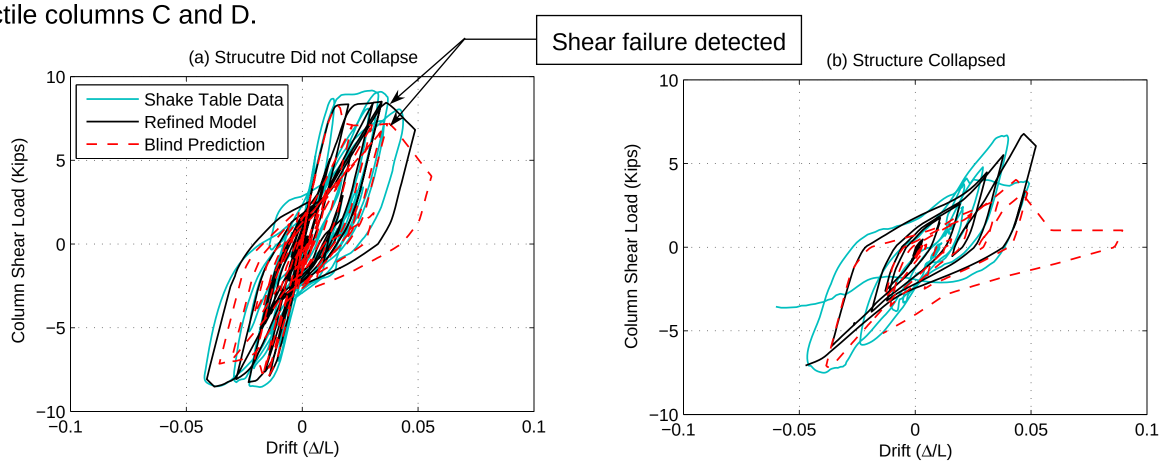


Figure 7. Shear hysteretic response for the first and second tests (a) first table motion (structure did not collapse) (b) second table motion (structure collapsed).

Fig. 7b demonstrates the shear hysteretic response for non-ductile column C during the second test. Despite the lack of agreement between the analytical model response and the experimental test results, the shear degradation slopes are similar. The poor match between the hysteretic results from the analytical model and the experimental test, particularly for “Blind Prediction”, was influenced by the cumulative errors that occurred in capturing the drift ratio and shear strength for the second test.

Axial Response

The axial response of the shake table specimen was influenced by three primary factors: the axial lengthening of the ductile columns, the initial axial lengthening of non-ductile columns and subsequent axial shortening due to shear failure, and the bending of the beam resulting from the change in the column lengths. The analytical model was able to capture the columns lengthening and shortening leading to redistribution of the gravity load.

The axial responses of the columns, during the first and second tests, are shown in Fig. 8. Similar to what was observed for shear response, the analytical model predicted the axial responses of the columns for the first test better than for the second. Several modeling assumptions and initial conditions including the spalling of the whole concrete cover (Blind Prediction), the residual shear and drift ratio from the first test, and approximations considered in the definition of the model for axial failure (Eqs. 2 and 3) influenced the disagreement of the results from the analytical model and the experimental test.

Due to its large size, the beam was expected to show very small deformations, yet, it affected deformations at the joints and consequently the shear and axial response of the columns. The ductile columns A and B elongated more than columns C and D due to their lower longitudinal reinforcement ratio.

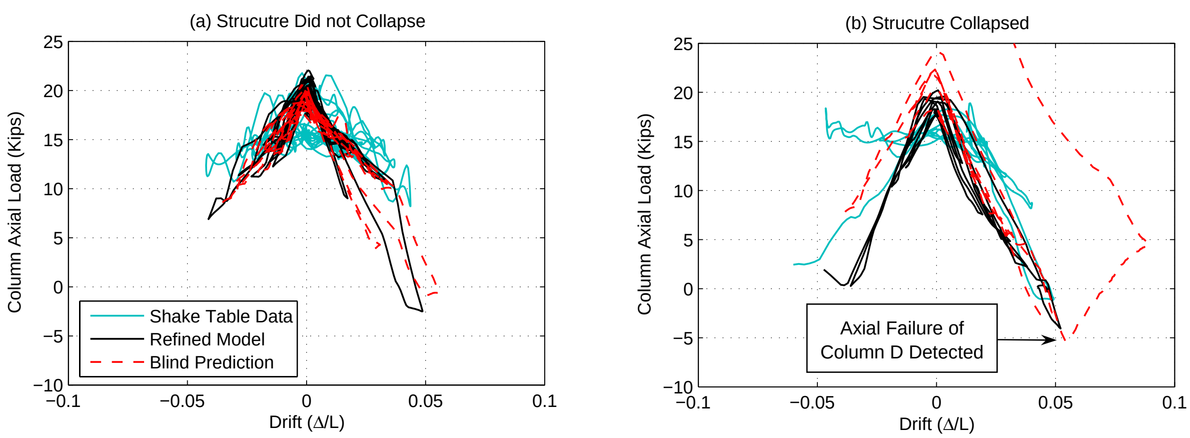


Figure 8. Axial hysteretic response for the first and second tests (a) first table motion (structure did not collapse) (b) second table motion (structure collapsed).

The axial hysteretic responses obtained from the refined model demonstrated a better agreement with the experimental test results. However, further studies are required to refine the analytical models in order to be able to predict the axial behavior of columns more adequately. Note that the sudden decrease in the calculated axial load for the first test at a drift of 4% is due to bending of the beam as the ductile columns lengthen, while column C ceases to increase in length after shear failure.

Comparison of Simplified Modeling Methods

Although the results of the above-mentioned analyses, especially for the shear behavior, were reasonably in agreement with the data from the shake table test, the procedure is very comprehensive and time consuming for practical purposes. In order to observe the impacts of simplification in modeling on the accuracy of the results, a few analyses using common simplified models were carried out. The employed models include 1) Nonlinear columns without shear, axial, and slip springs 2) Elastic columns with concentrated rotational springs at the ends which follow the FEMA356 backbone 3) Elastic columns with concentrated rotational springs at the ends having rigid-perfectly-plastic behavior. Based on FEMA356, the effective stiffnesses for the elastic columns were considered as 50% of the gross stiffnesses.

To observe the effects of individual axial, shear, and slip springs on total response of the structure, a series of analyses were carried out where one spring type was eliminated at each analysis. Nonlinear beam-column elements with fiber sections were considered for the columns in these analyses. As shown in Fig. 9a, the model with all springs was the only one that could predict the point of collapse accurately. However, eliminating the axial spring had negligible effect on capturing the lateral response of the structure. The slip spring elimination influenced the drift of the structure in the elastic range (starting of the first table motion), but did not have a significant effect on prediction of the drift of the structure after yielding. Although absence of the shear spring in the model did not change the maximum displacements of the structure, it affected the shear behavior of the non-ductile columns. As demonstrated in Fig. 9b, considering shear springs in column C caused the model to follow the degradation path after the point of shear failure which made the shear response very similar to the recorded one from the shake table test.

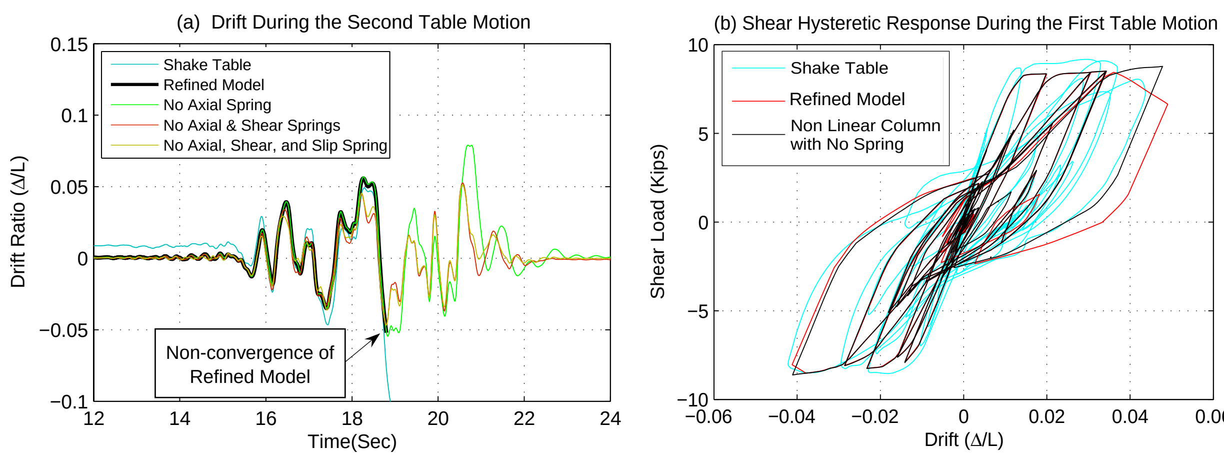


Figure 9. Effects of elimination of springs on the response of the structure (a) drift time history for the second table motion (structure collapsed) (b) shear hysteretic response for the first table motion.

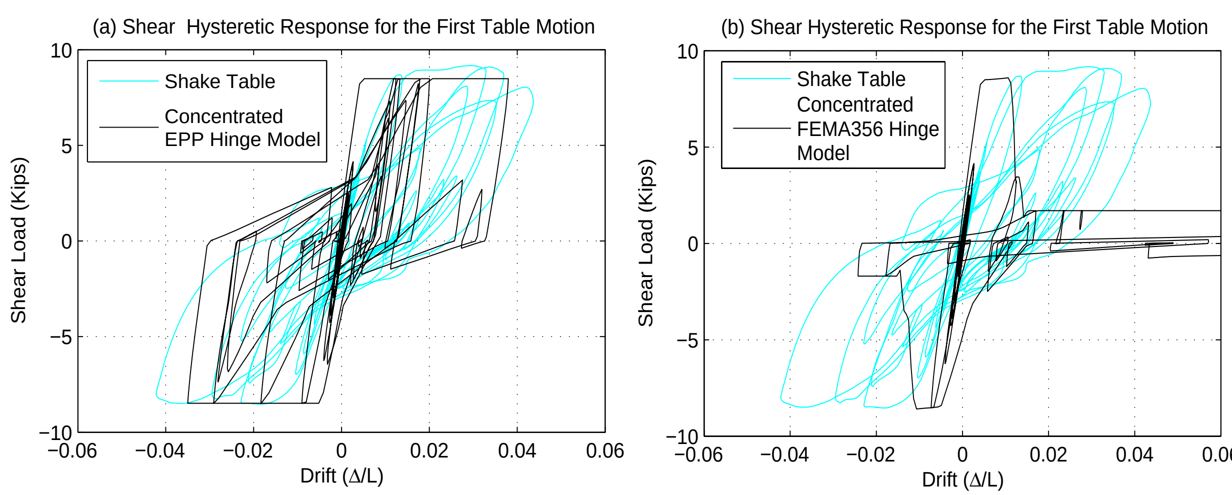


Figure 10. Comparison of shear hysteretic response for column C (a) concentrated elastic-perfectly-Plastic hinge model (b) concentrated FEMA356 hinge model.

Fig. 10 compares the shear response from the shake table test and results from analysis using concentrated hinges at the column ends to model inelastic behavior. The shear hysteretic response shown in Fig. 10a was obtained using rotational springs with rigid-perfectly-plastic behavior. As demonstrated, the hysteretic response of the column was not completely captured in the positive direction. However, the overall behavior is in agreement with the shake table test and the peak displacement is only underestimated by 12%. The results from the analytical model using the rotational sprigs with the FEMA356 backbone is compared with the shake table test data in Fig. 10b. It is observed that the backbone suggested by FEMA356 significantly underestimates the maximum drift capacity of the structure. The collapse prediction acceptance criteria from FEMA356 is equivalent to a drift ratio of 0.024. However, according to the shake table test data, specimen did not collapse during the first table motion (maximum drift ratio of 0.043). Furthermore, the very sudden degradation in shear resistance included in the FEMA356 model significantly overestimates the drift demand on the structure.

Conclusions

The goal of this work was to study the accuracy of currently available empirical capacity models in predicting the inelastic response, and in particular the failure mechanisms, of existing shear-critical reinforced concrete columns subjected to gravity and seismic loading, and to validate these models with experimental test data.

The shear hysteretic response of the shear-critical columns, obtained by using deformation components and shear capacity models developed by Zhu (2005), showed a reasonable agreement with the results from the experimental tests. The analytical model provided a very good estimate of the measured drifts through the point of shear and axial failure. Comparison of the measured axial response of the test specimen with the results for the analytical model demonstrates that the axial behaviors of the columns were not captured very well and further calibration of the OpenSees analytical model for axial capacity of shear-critical RC columns is required.

Comparison of the model with some of the common simplified models revealed that shear, axial, and slip springs are required to capture the point of collapse of the structure. It was also observed that FEMA356 predicts the failure of the structure very conservatively.

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