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Collapse performance prediction for RC frame structures

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ABSTRACT: Reinforced concrete frames with light transverse reinforcement may be susceptible to shear and subsequent axial load failures. An experimental program examined the behaviour of two half-scale, one-story frames with axial loads representative of those expected for the lower story of a seven-story building. The frames were subjected to unidirectional simulated earthquake motion applied at the base. Shear failures of an interior column led to axial load failure and redistribution of internal forces to adjacent framing components. Analytical models are proposed to identify onset of shear and axial failure. The models are incorporated in a computer framework for numerical simulation of nonlinear dynamic response under earthquake base motion.

1 INTRODUCTION

Experimental research and post-earthquake reconnaissance have demonstrated that reinforced concrete columns constructed with light or widely spaced transverse reinforcement are vulnerable to shear failure during earthquakes. Such damage can also lead to a reduction in axial load capacity, although this process currently is not well understood. The resulting redistribution of gravity loads to the neighboring elements may play a role in progressing the collapse of the building frame. Shake table tests were conducted in an effort to investigate the process of column shear and axial load failures and the effect such failures have on the rest of the building frame.

2 DESIGN OF SHAKING TABLE TESTS

Shake table tests were designed to observe the process of dynamic shear and axial load failures in reinforced concrete columns where an alternative load path is provided for load redistribution. The test specimens were composed of three columns fixed at their base and interconnected by a beam at the upper level (Figure 1). The central column had wide spacing of transverse reinforcement making it vulnerable to shear failure, and subsequent axial load failure, during testing. As the central column failed, shear and axial load would be redistributed to the adjacent ductile columns.

Two test specimens were constructed and tested. The first specimen supported a mass of 300 kN, producing column axial load stresses roughly equivalent to those expected for a seven-story building. The second specimen also supported a mass of 300 kN, but pneumatic jacks were added to increase the axial load carried by the central column from 128 kN (0.10 f'_cA_g) to 303 kN (0.24 f'_cA_g), thereby amplifying the demands for redistribution of axial load when the central column began to fail.

Specimens were constructed of normal-weight aggregate concrete (10-mm maximum aggregate size). See Table 1 for information on material properties.

The stiffness of the beam in the three-column frame was selected such that the scaled maximum vertical deflection after axial failure of the center column for the first test specimen would be the same as the maximum deflection of the second story in a seven-story prototype building after axial failure of an interior first story column. The beam reinforcement was selected such that the ratio of the yield strength of the beam to the maximum moment demand from plastic analysis after axial failure of the center column was 1.59 for the Specimen 1 and 0.82 for Specimen 2.

Table 1. Properties for shake table test specimens

f'_c (columns and beam, Specimen 1)	24.5 MPa
f'_c (columns and beam, Specimen 2)	23.9 MPa
f_y (centre column longitudinal bars)	479 MPa
f_y (outside column longitudinal bars)	424 MPa
f_y (centre column transverse bars)	718 MPa
r_l (centre column)	2.5 %
r_l (outside column)	2.0 %
r_h (centre column)	0.18%

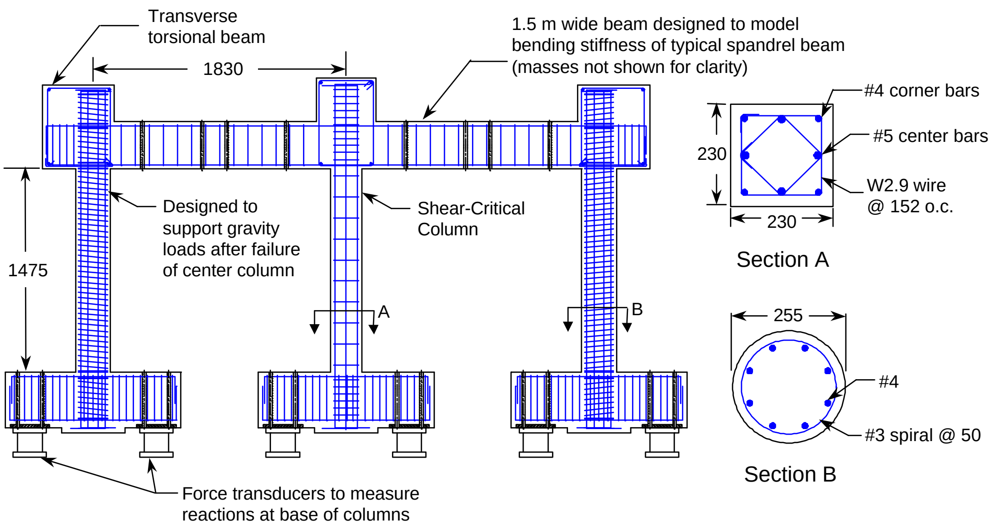

Figure 1. Shake table test specimen (units in mm). #3, #4, and #5 bars have nominal diameters of 9.5, 12.7, and 15.9 mm.

Each test specimen was moved to the earthquake simulator prior to testing and supported on force transducers that monitored axial load, shear, and moment (Figure 1). The beam supported lead weights. Pneumatic jacks applied additional load to Specimen 2.

The planar frame specimens were to be subjected to one horizontal component from a scaled ground motion recorded at Viña del Mar during the 1985 Chile earthquake (Figure 2). An out-of-plane bracing system was developed to restrain motion out of the plane of the specimen; otherwise the bracing system allowed unrestrained in-plane horizontal and vertical motion.

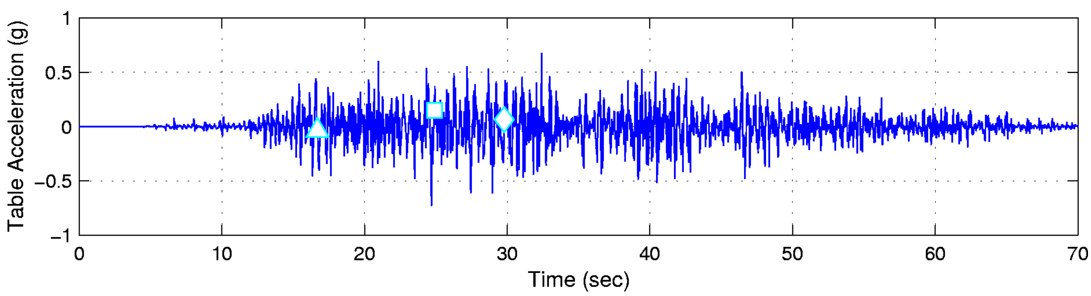

Figure 2. Table acceleration record (scaled Chile, 1985)

3 TEST RESULTS

Selected results from the shake table tests are plotted in Figures 3 and 4. The triangular marker indicates the approximate time at which the center column shear for Specimen 2 begins to drop off relative to the center column shear for Specimen 1. Also at this time, the center column axial load for both specimens drops by approximately 40 kN. This drop in load coincides with the development of significant cracks in the outside and center columns, and is thought to be caused by redistribution of gravity loads as the lengths of the columns change owing to flexural response.

The square marker indicates the pulse that initiates the axial failure of Specimen 2. Figure 4 demonstrates that by the time indicated by the square marker the center column shear capacity for Specimen 1 has only just begun to degrade, while the center column shear capacity of Specimen 2 has degraded to less than one-half of the previously attained center column shear.

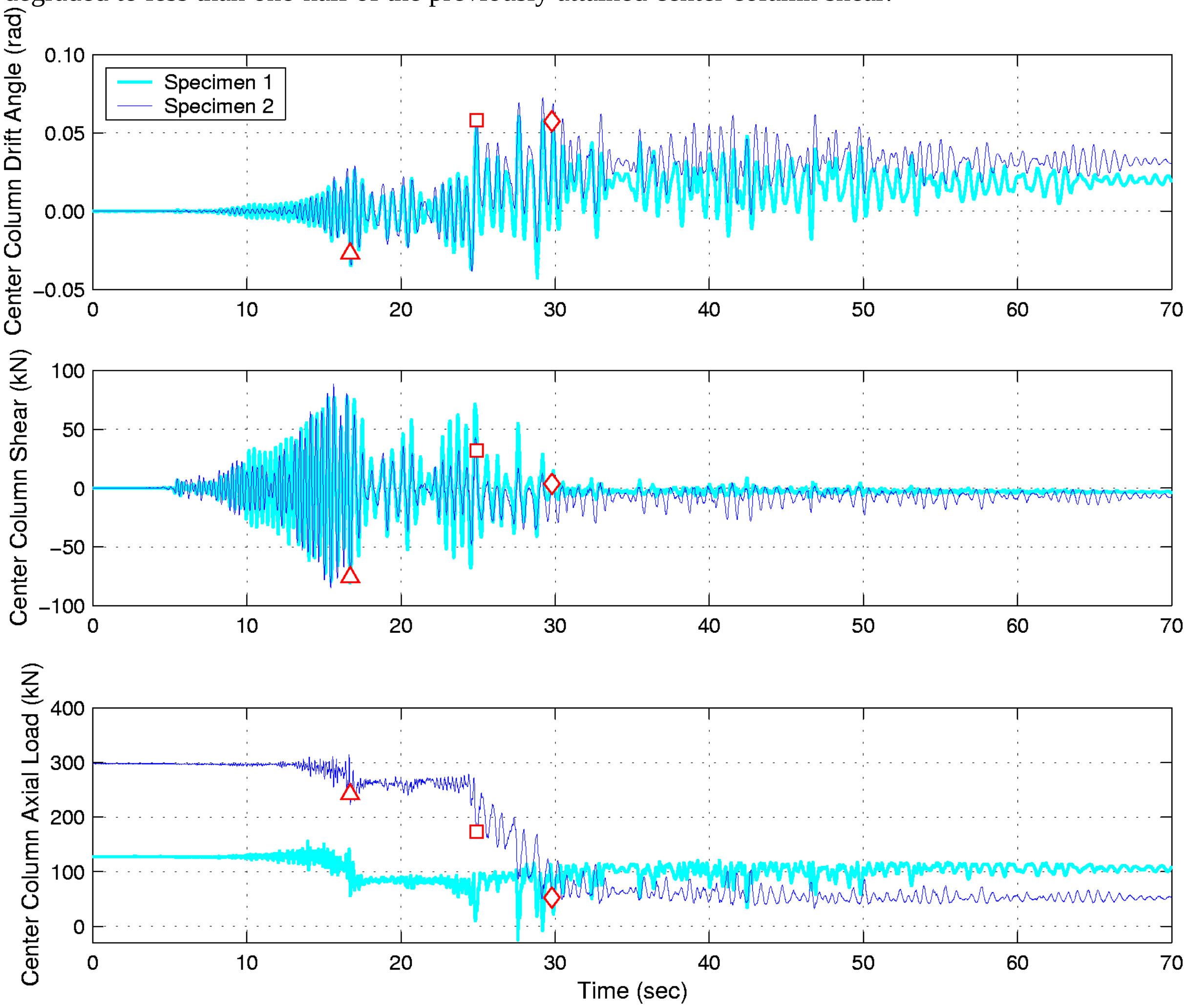

Figure 3: Response histories for shake table tests

The diamond marker indicates the approximate time at which the minimum center column axial load is reached for the first time. By this point the center column shear capacity has all but disappeared for both specimens.

Figure 5 shows the state of both columns at the end of the tests.

4 DEFORMATION AT ONSET OF SHEAR FAILURE

Several models have been developed to represent the degradation of shear strength with increasing inelastic deformations (Watanabe and Ichinose, 1992; Aschheim and Moehle, 1992; Priestley et al. 1994; Sezen, 2002). While these shear strength models are useful for estimating the column strength as function of deformation demand, they are less useful for estimating displacement at shear failure. For example, the model by Sezen represents shear strength as a function of displacement ductility using the relation in Figure 6, where V_m = shear strength as function of displacement ductility and V_0 = initial shear strength. A small variation in shear strength (shown by mean plus or minus one standard deviation bounds) or similar variation in flexural strength (not shown) can result in large variation in estimated displacement capacity, Dm_d. These models also suggest misleading trends in the relation between some critical parameters (such as axial load) and displacement capacity.

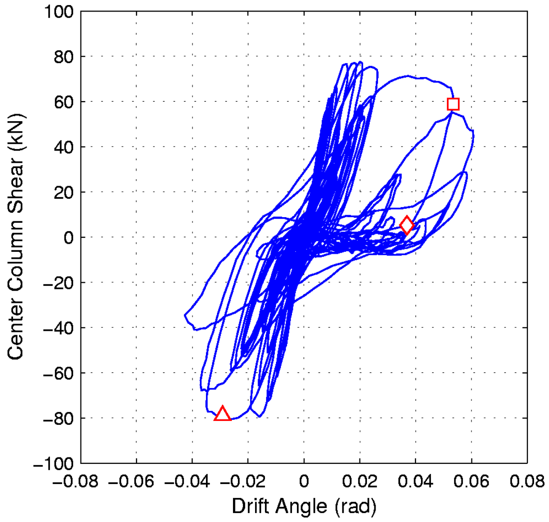

Figure 4a. Specimen 1 center column shear hysteretic response

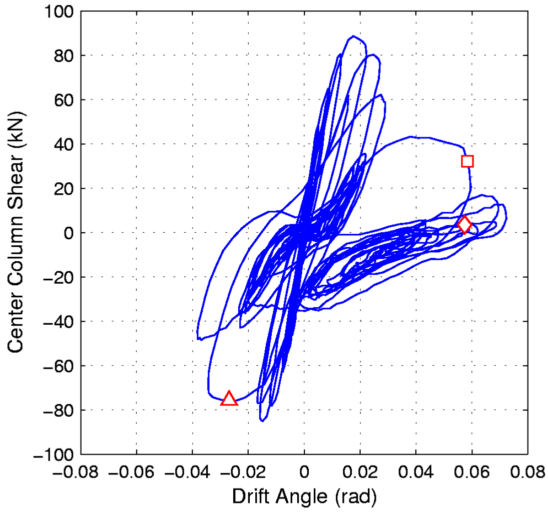

Figure 4. Specimen 2 center column shear hysteretic response

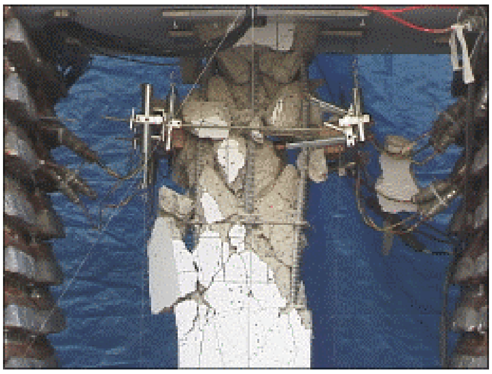

Figure 5a. Top of center column, Specimen 1 at end of test

Figure 5b. Top of center column, Specimen 2 at end of test

Pujol et al. (1999, 2000, 2002) have proposed drift capacity models for columns failing in shear. These models make an important contribution by focusing attention directly on displacement capacity and by analyzing data for model development. The database of Pujol et al. includes columns with transverse reinforcement ratios exceeding 0.01, which is larger than that which is of interest in the present study.

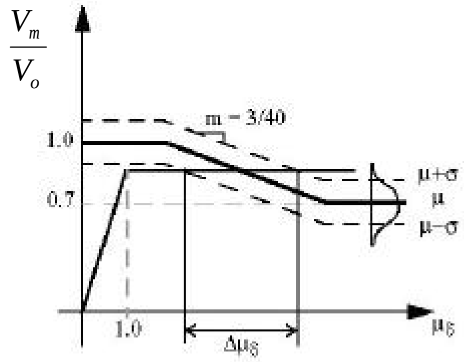

Figure 6. Displacement at shear failure as function of model variability

In the present study, a database of 49 columns having lower transverse reinforcement was studied. The database, compiled by Sezen (2002), consists of column specimens with observed shear distress at failure and tested in single or double curvature with the following range of properties: shear span to depth ratio: 2.0 < *a/d* < 4.0; concrete strength: 17 < $f_c^{'}$ < 45 MPa; longitudinal reinforcement nominal yield stress: 270 < f_{yl} < 550 MPa; longitudinal reinforcement ratio: 0.01 < r_l < 0.08; transverse reinforcement ratio: 0.01 < $\frac{r^{''} f_{yt}}{f_c^{'}}$ < 0.12; maximum shear stress: 0.17 < $\frac{v}{\sqrt{f_c^{'}, MPa}}$ < 0.75.

The model of Sezen (2002) can be used to estimate mean shear strength as a function of displacement ductility. As suggested by Figure 6, the intersection of the shear corresponding to flexural strength with mean shear strength can be interpreted to indicate the expected displacement at shear failure. Figure 7 compares results obtained by this procedure with those actually observed during the tests. The correlation seems unsatisfactory.

A reanalysis of the data from a displacement-capacity perspective resulted in the following relationship to estimate displacement at onset of shear failure:

$$\frac{\Delta_s}{L} = 4r^{''} - \frac{1}{40}\frac{v}{\sqrt{f_c^{'}}} - \frac{1}{40}\frac{P}{A_g f_c^{'}} + \frac{3}{100} \geq \frac{1}{100} \qquad (1)$$

where $\frac{\Delta_s}{L}$ = drift ratio at shear failure, $r^{''}$= transverse steel ratio, v = nominal shear stress (in MPa), $f_c^{'}$ = concrete compressive strength (in MPa), P is the axial load on the column, and A_g is the gross cross-sectional area. Figure 8 compares measured and calculated displacements at shear failure according to Equation (1). The correlation is improved compared with that reported in Figure 7.

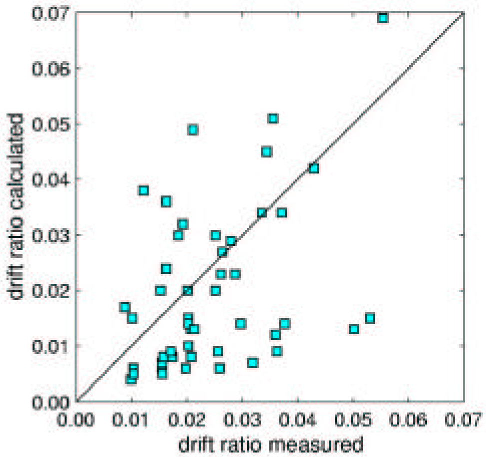

Figure 7. Measured versus calculated drift at shear failure, interpreted from Sezen shear strength model.

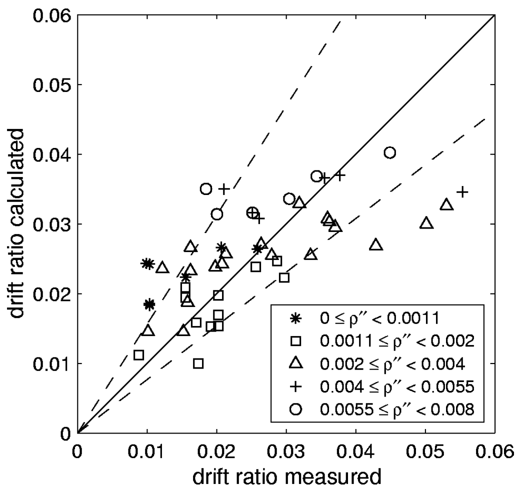

Figure 8. Measured versus calculated drift at shear failure, equation (1)

5 DEFORMATION AT LOSS OF AXIAL-LOAD CAPACITY

Elwood (2002) has extended a shear-friction model, first presented in Moehle et al. (2001), to represent the general observation from experimental tests that the drift ratio at axial failure of a shear-damaged column is inversely proportional to the magnitude of the axial load. Considering a free-body diagram of the upper portion of a column under shear and axial load, the classic shear-friction equation from ACI 318 (1999) ($V_{sf} = Nm$), and several simplifying assumptions, Elwood (2002) developed relations among axial load, transverse reinforcement, and drift ratio at axial load collapse as

$$\left(\frac{\Delta}{L}\right)_{axial} = \frac{4}{100}\frac{1+\tan^2 q}{\tan q + P\left(\dfrac{s}{A_{st} f_{yt} d_c \tan q}\right)} \tag{2}$$

in which q = critical crack angle assumed = 65 deg, P = axial load, A_{st} = area of transverse reinforcement parallel to the applied shear and having longitudinal spacing s, f_{yt} = transverse reinforcement yield stress, and d_c = depth of the column core measured parallel to the applied shear. Results of tests by Sezen and Lynn are shown in Figure 9. While useful as a design chart for determining drift capacities, Figure 9 must only be used with a full appreciation for the limited accuracy of the results as discussed in Moehle et al. (2001) and the limitation that the results are based on unidirectional, pseudo-static tests.

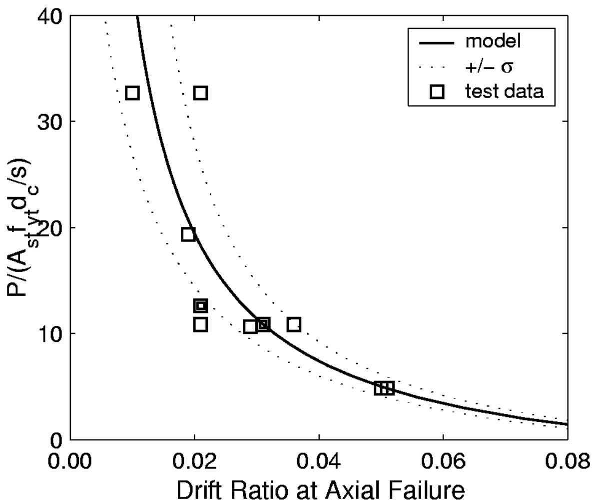

Figure 9. Drift capacity curve based on shear-friction model

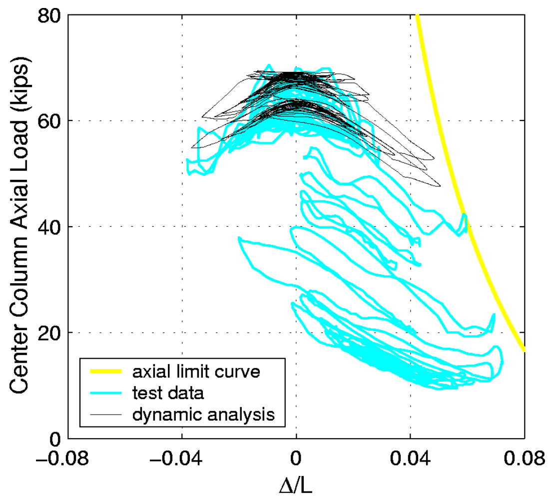

Figure 10. Comparison of shear-friction model with shake table test data from Specimen 2, 1 kip = 4.448kN

Figure 10 plots the results from the shake table test (Specimen 2) along with the drift capacity curve for the centre column based on the model discussed above. The intercept of the centre column response and the model occurs at approximately 24.9 seconds, as indicated by the square marker (the same square marker appears in Figures 3-4). At 24.9 seconds significant distortion of the top of the centre column, possibly due to sliding along the diagonal shear failure plane, could be observed visually. The centre column response for Specimen 1 (not shown) lies entirely below the drift capacity curve, indicating that the shear-friction model correctly predicts no axial failure for the test column with low axial load.

6 IMPLEMENTATION IN NONLINEAR DYNAMIC ANALYSIS

An analytical model for reinforced concrete columns vulnerable to shear and axial load failures has been developed for the OpenSees (2002) analytical platform. The analytical model incorporates the capacity model defining shear failure in addition to the capacity model described above for axial load failure. The column model consists of a beam-column element in series with zero-length shear and axial springs. As implemented, all deformations are accounted for by the beam-column element before shear or axial failure. Force and deformations in the beam-column element are used to trigger nonlinear response of zero-length springs. The displacement at shear failure for the shear failure model is based on the deformation model given by Equation (1). Since the springs are in series with the beam-column element, the springs will act as a fuse by limiting the loads carried by the entire column model. In the case of shear failure, the deformations of the entire system after the onset of failure is detected will be dominated by the shear deformations. A similar approach is implemented for axial failure using the interaction surface defined in Figure 9.

The column model described above was used to model the shear failure observed in the center column of Specimen 2. All columns were modeled using a fiber model to represent the interaction of flexural and axial response. The model was subjected to the input base motion recorded during the test of Specimen 2. Equivalent viscous damping was set equal to two percent of critical.

The computed and measured base shear and drift ratio histories are compared in Figure 11. The analytical model adequately represents the measured response in terms of apparent vibration period and force amplitude throughout the test. The drifts are well predicted by the analytical results up to the point of axial failure (approximately t = 25 sec), at which point the permanent offset in the drifts observed in the test is not captured by the analysis. The analysis did not detect axial failure of the center column due to the underestimation of the lateral displacements.

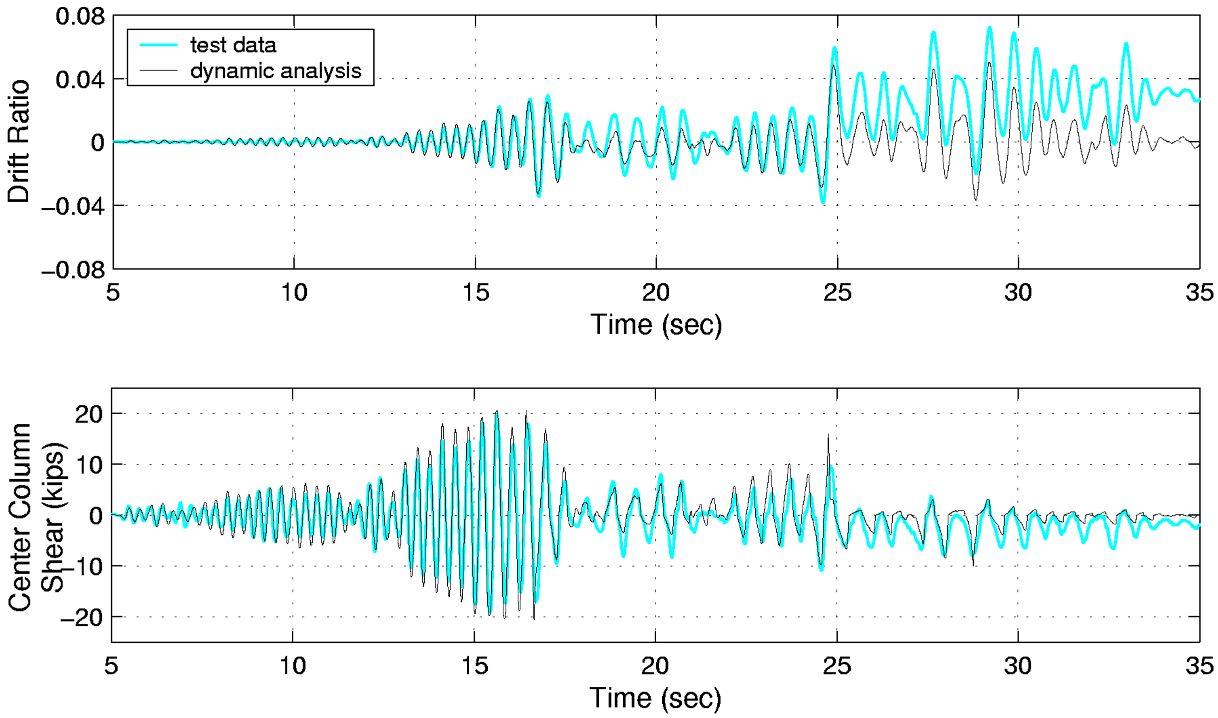

Figure 11. Comparison of experimental and analytical response histories for Specimen 2, 1 kip = 4.448kN.

Figure 10 shows the relation between centre column axial load and drift ratio. The model adequately represents the interaction initially, but fails to identify axial load failure because the analytical model underestimated the drift ratio achieved during the test. Analytical studies in which the drift ratio was artificially amplified or the axial failure envelope was reduced produced axial failure responses more closely resembling the response measured during the test. Elwood (2002) investigates these issues to identify the sensitivity of global response to details of the analytical model.

7 CONCLUSIONS

Shake table tests were conducted to observe the process of dynamic shear and axial load failures in reinforced concrete columns when an alternative load path is provided for load redistribution. The test results show that the axial stress on the column influences the behaviour of the column during shaking, particularly after shear failure. A column with an axial stress of $0.24f'_c$ failed to maintain its gravity loads, while another column with an axial stress of $0.10f'_c$ only saw minor gravity load redistribution. An axial failure model based on shear-friction compared favourably with the test results. A new analytical model for columns vulnerable to shear and axial load failures demonstrates the potential to reproduce the shake table test results analytically. Such a model would allow for the analysis of older reinforced concrete frame buildings vulnerable to gravity load collapse.

8 ACKNOWLEDGMENTS

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