TESTS TO COLLAPSE OF SIMPLE STRUCTURES AND COMPARISON WITH EXISTING CODIFIED PROCEDURES

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ABSTRACT

The results of tests on fifteen four-column frame specimens subjected to progressive unidirectional ground shaking up to collapse are analyzed. Test structure performance is compared with proposed limits for minimizing P-Δ effects in highway bridge piers. The stability factor is found to have a strong relation to the relative structural performance in this regard. Specimens with θ less than 0.25 satisfied the proposed criteria for at least a portion of their respective test schedule. The limit is violated prior to the final test in which complete specimen collapse occurs. Finally, a series of shake table tests are compared with an analytical model using two methods.

Introduction

The arbitrary lateral drift limits prescribed by modern design codes to limit non-structural damage also indirectly ensure that structural performance is minimally affected by the effect of gravity on the lateral force resistance of structures (a.k.a. P-Δ effect). However, as conventional drift limits are progressively eliminated and replaced by other performance-based limits, inelastic behavior is relied upon to a greater extent in the dissipation of seismic input energy. Therefore, accurate quantification of the destabilizing effect of gravity is becoming more significant for structural design. As a result, it may be desirable to investigate the behavior of those structures in order to enhance our understanding of the condition ultimately leading to their collapse, and to ensure public safety during extreme events. While many experimental studies and theoretical damage models support these calculated values, it remains that few experimental studies have pushed the shake table tests up to collapse.

A recently published report (Vian and Bruneau 2001) discusses a program of shake-table testing of simple frames through collapse. This series of tests provide well-documented data, freely available on the world wide web, which may be used to develop or validate algorithms capable of modeling the inelastic behavior of steel frames up to collapse. Fifteen specimens having various properties were tested in an attempt to identify some of the general parameters responsible for trends in behavior of Single-Degree-of-Freedom (SDOF) structures to collapse under earthquake loading.

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In this paper, some basic concepts of P-Δ behavior are summarized, allowing for comparison with the recorded results of the above-mentioned testing program. This is followed by a preliminary investigation of behavioral trends observed from the shake table results. In particular, peak responses are compared with limits proposed by others to minimize P-Δ effects in bridge piers. Finally, experimental results are compared with those obtained using a simple analytical model, to illustrate use of the generated experimental data for to develop or calibrate models of inelastic behavior to collapse. Progressive bilinear dynamic analyses are performed and compared with shake table test results. A basic, bilinear model is used for illustration purposes, and some observations are made with respect to the effect of damping estimates, on analysis results, and a few other issues.

P-Δ Behavior

Some concepts for characterizing P-Δ effects in inelastic SDOF structures under lateral load are described below, along with an overall view of the fundamental structural behavior. Fig. 1a shows a column from a single bay, single story structure, with an infinitely stiff beam, thereby resulting in a lateral stiffness of the column, ignoring P-Δ, of $K_o = 12EI / L^3$. A bilinear, SDOF model is shown in Fig. 1b. Elastic-perfectly plastic structural response (neglecting P-Δ) is shown, as well as the response modified by the influence of P-Δ. MacRae, Priestley and Tao (1993) provided a summary of additional concepts on P-Δ effects on simple structures during earthquakes.

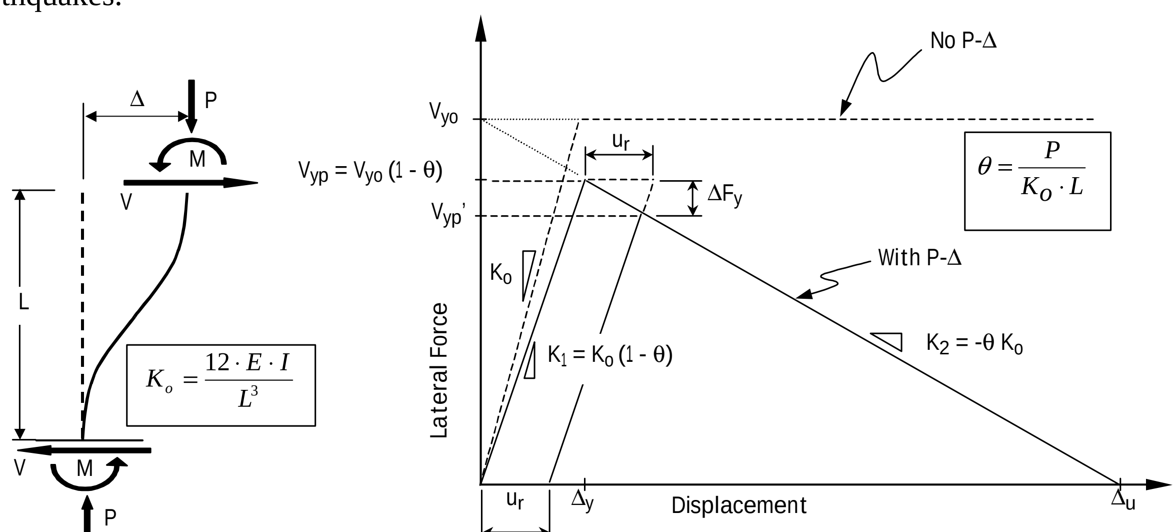

Figure 1. Bilinear Lateral Force versus Displacement model for SDOF structure

Overview Of Experimental Program

A SDOF shaking table located at the University at Buffalo Structural Engineering and Earthquake Simulation Laboratory (UB-SEESL) was used to conduct the testing program from which the results are analyzed in this paper.

The ground acceleration time history for the El Centro S00E Imperial Valley earthquake of May 1940 was used in this study. A displacement record was generated from this time history for use as input to the displacement-controlled actuator. Note that unscaled ground motions were used as the specimens were designed to fit actual parameters of interest, and not intended to be scaled models of actual structures.

Description Of Specimens

Fifteen specimens, each having four columns, were fabricated at the University of Ottawa and tested to failure at the UB-SEESL in the course of this research. These fifteen specimens were subdivided into three groups of five with slenderness ratios of 100, 150, and 200. A range of values for column axial capacity versus demand was chosen for each slenderness ratio.

A number of methods were used to analyze each specimen prior to testing. This data was used to determine the scaling factors for the earthquake excitation in developing a testing schedule for each specimen. Each column in each specimen was measured in numerous ways prior to testing, in order to fully document considered imperfections (Vian and Bruneau 2001).

Instrumentation

Instrumentation was designed to record structural response in a number of ways. A strain gage was mounted on one column of each specimen, and located at a distance of one-third of the column height from its bottom base plate for estimation of forces during testing as detailed in the available report. Displacement transducers (LVDT's) were used to measure displacement of the table (labeled "UG"), vertical displacement of the mass (labeled "Vert"), and total horizontal displacement at the East and West side of the mass (labeled "HorEast" and "HorWest"). Special modifications to the LVDT's setup were required to allow measurement of the structural mass displacement during the entire response history, including much of the collapse. A schematic of the test setup and instrumentation is shown in Fig. 2.

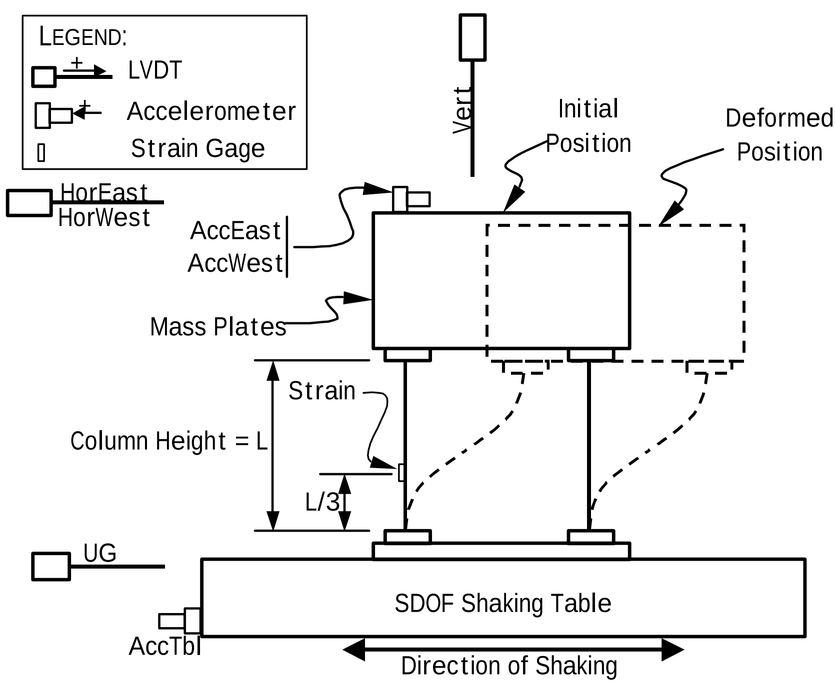

Figure 2. Schematic of Test Setup and Instrumentation (Looking West)

Shake Table Testing

A free vibration test was performed on each specimen prior to the initiation of its respective schedule of shake table tests. Shake table test schedules were established for each specimen, creating a series of progressively more severe shake table tests until the structure collapsed. Approximately five levels of the ground motion were selected for application to each specimen.

Analysis Of Experimental Results

The shake table tests briefly described above are analyzed with regard to some of the concepts described above. All specimens are investigated in the perspective of the entire series of tests to which they were subjected.

Behavioral Trends

The value of the stability factor, θ, has a significant effect on the response of the structure. In practical bridge and building structures, θ is unlikely to be greater than 0.10, and is generally less than 0.060 (MacRae, Priestley and Tao 1993). Specimen 1 is found to be the only one here that has a θ value near that suggested practical range for the stability factor, with a value of 0.065. Specimens 2,6, and 11 have stability factors slightly larger than the likely upper limit, at 0.123, 0.101, and 0.138, respectively. All other specimens have a value of $\theta \geq 0.155$.

Results of a graphical study of peak response parameters are summarized below. Three response parameters, $S_{a\text{-}final}$, μ_{final}, and γ_{final}, the spectral acceleration, displacement ductility, and drift, respectively, of the penultimate shake table test, were compared with the stability factor, θ. These response quantities are also compared with the static stability limit, μ_s, defined as the structure's ductility at ultimate displacement, Δ_u. From the information given in Fig. 1b, μ_s can be shown to be the inverse of the stability factor. The following general observations can be made:

- The elastic spectral acceleration, S_a, ductility, μ, and percent drift, γ, were observed to have inverse relationships with θ. Spectral acceleration is plotted versus the stability factor in Fig. 3 for the penultimate test (given subscript "final"). This suggests that structures may be less able to undergo large inelastic excursions before imminent instability as the stability factor increases. Specimens 1 and 6, which have the lowest values of θ tested, were the only specimens able to withstand $S_a > 0.75$ g.

- Specimen 1 was the only specimen that underwent both a ductility greater than five (20.35), and a drift larger than 20% of the specimen height (64%), prior to collapse. Recall that this is the only specimen that has a value of θ less than 0.1.

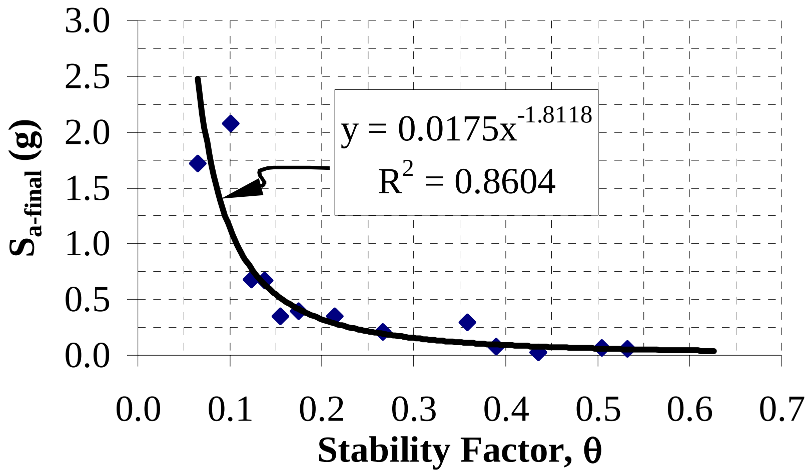

Figure 3. Spectral Acceleration versus Stability Factor

The static stability limit, μ_s, can be expressed as the inverse of the stability factor, as previously shown. The same set of parameters discussed above was compared to μ_s. A reverse trend than shown in Fig. 3 is observed, as expected. The ductility of each specimen's penultimate shake table test is plotted versus its static stability limit in Fig. 4. The line $\mu_{final} = \mu_s$ is shown for clarity. Only Specimen 1 was able to exceed its static stability limit.

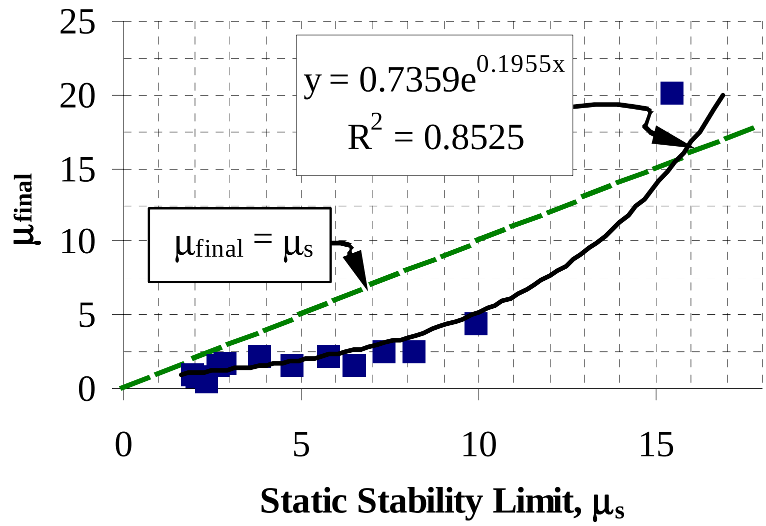

Figure 4. Displacement Ductility versus Static Stability Limit

Overall, ultimate inelastic behavior is shown by this investigation to have a high dependence on the stability factor for a P-Δ affected structure. For the specimens tested in this research having a value of $\theta \geq 0.1$, a relatively low level of inelastic behavior was exhibited before collapse. Specimens with $\theta < 0.1$ were able to withstand ground motions with higher spectral accelerations, experience larger values of ductility, and accumulate larger drifts, than those with $\theta > 0.1$. The more slender structures, characterized by a larger θ value, will undergo relatively small inelastic excursions prior to collapse.

NCHRP 12-49 Proposed P-Δ Limits for Bridge Piers

The National Cooperative Highway Research Program (NCHRP), Project 12-49, under the auspices of the Transportation Research Board, is investigating seismic design of bridges from all relevant aspects. At the conclusion of this project, proposed revisions to the current LRFD specifications for highway bridges will be presented to the American Association of State Highway Transportation Organizations (AASHTO) for review and possible implementation. Included in the proposed revisions, are how additional demands from P-Δ affect structural performance. The most recent proposed provision, as of this writing, seeks to limit P-Δ effects on bridge piers by suggesting the following:

"The displacement of a pier or bent in the longitudinal and transverse direction must satisfy:

$$\Delta_m \leq 0.25 \cdot C \cdot \left(\frac{W}{P}\right) \cdot H \qquad (1)$$

where:

Δ_m = $R_d \times \Delta$
R_d = Factor related to response modification factor and fundamental period
Δ = Displacement demand from the seismic analysis
C = Seismic base shear coefficient based on lateral strength
W = Weight of the mass participating in the response of the pier
P = Vertical load on the pier from non-seismic loads
H = Height of the pier"

For analysis of the specimens in this research, the W/P ratio is equal to unity, and the measured experimental displacements, u_{rel}, and estimated base shear coefficient, C_s^* (= V_p^*/W), can substitute for Δ_m and C, respectively. Note that V_p* is the experimentally estimated base shear, corrected for P-Δ, as described in the report.

In Fig. 5 the proposed limit is compared with the peak experimental responses. The estimated base shear coefficient, C_s*, is plotted as a function of the maximum drift, $\gamma (= u_{rel}/H)$. In Fig. 5a, the specimens for which $\theta < 0.25$ (1, 2, 4, 6, 7, 11, and 12) are shown. During the initial tests, when the proposed limit was satisfied, none of these specimens failed. However, in the subsequent tests of the schedule, due to repeated inelastic action, the cumulative drifts of the structure increased, eventually causing progressive collapse and violating the proposed limit. Collapse always occurred only after the limit was exceeded in a prior test, thus validating the proposed criterion.

As shown in Fig. 5b, the remaining specimens, for which $\theta \geq 0.25$, never satisfied the drift criteria, even for those tests that remained in the elastic range. The stability factor for these specimens, however, is well above the practical range discussed previously and, therefore, the limit violation is of no consequence.

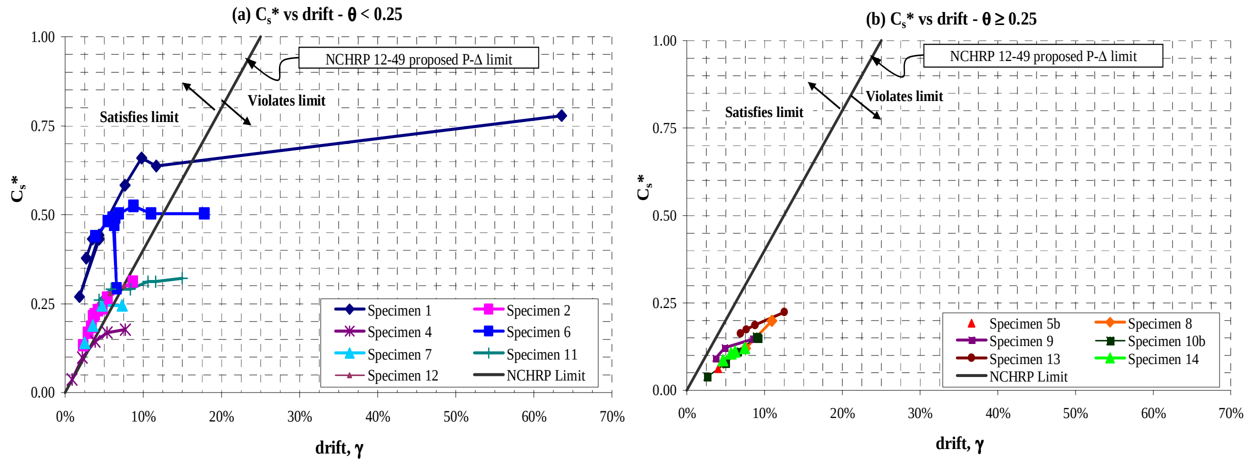

Figure 5. Test Results Comparison with NCHRP 12-49 Limits

Model Verification Example

Data from the experimental program can be used to assess the efficacy of SDOF time history analysis algorithms to model inelastic structural behavior up to collapse. This is illustrated through an example utilizing the simple freeware program NONLIN (Charney 1998).

Specimen data necessary for model verification was presented in the available report. Yield stress determined from tension testing was used to calculate the plastic moment capacity, M_p, of the average column section for each specimen. From this, the plastic base shear (neglecting P-Δ), V_{yo}, and the yield displacement, Δ_y, are subsequently calculated. Column and material data are also used to calculate the stability factor, θ, initial stiffness, K_1, plastic base shear considering P-Δ, V_{yp}, and yield displacement, Δ_y, for Specimen 11, equal to 0.138, 8.05 N/mm, 255 N, and 31.67 mm, respectively. Average dimensions listed in the report were used to determine these properties prior to analysis. Analysis of a test specimen was carried out using these bilinear properties as model input, with results presented below. Note that the analyses are carried out on one column of the four making up the specimen, with one quarter of the total specimen mass acting on it.

Force-Displacement Comparison

Analytically and experimentally obtained lateral force versus displacement results were compared for the specimen for its entire series of tests, by performing non-linear dynamic analyses using a bilinear elastic-perfectly plastic model. The average damping ratio estimate of 1.804% was used for each analysis. The bilinear parameters of the virgin specimen specified in the NONLIN model were obtained from data previously presented and described above. However, when the specimen experienced a residual displacement at the conclusion of a test, the model was modified for the subsequent test to account for the lower yield base shear upon reloading due to P-Δ effects and bias in the cumulative drifts.

Fig. 1 shows, for the assumed bilinear force-displacement model, the reduced specimen yield level, V_{yp}', following a residual displacement, u_r, is given by:

$$V_{yp}' = V_{yp} - \Delta F_y = K_1 \cdot \left(\Delta_y - \theta \cdot u_r\right) \quad (2)$$

where ΔF_y is the specimen yield base shear reduction, and all other terms have been defined previously.

Two series of bilinear dynamic analyses were performed: First, the experimentally obtained residual displacement from each test was used to calculate the reduced yield force for the subsequent analysis (referred to as Method 1 hereafter). Second, the residual displacement obtained analytically was used to calculate the reduced yield force (Method 2). Note that Method 2 is a purely analytical approach, whereas Method 1 is a hybrid in that the experimental results are used to "adjust" each successive analysis. Residual displacement and reduced yield force values obtained using each of these methods is summarized in Table 1. Experimentally estimated base shear, V_p*, and Method 2 analytical base shear, f_s, are plotted versus relative displacement, u_{rel}, in Fig. 6 for each shake table test in the series for Specimen 11. Note that neither method provides a good match with experimental data, and that the second method, shown below, predicted collapse before the final test.

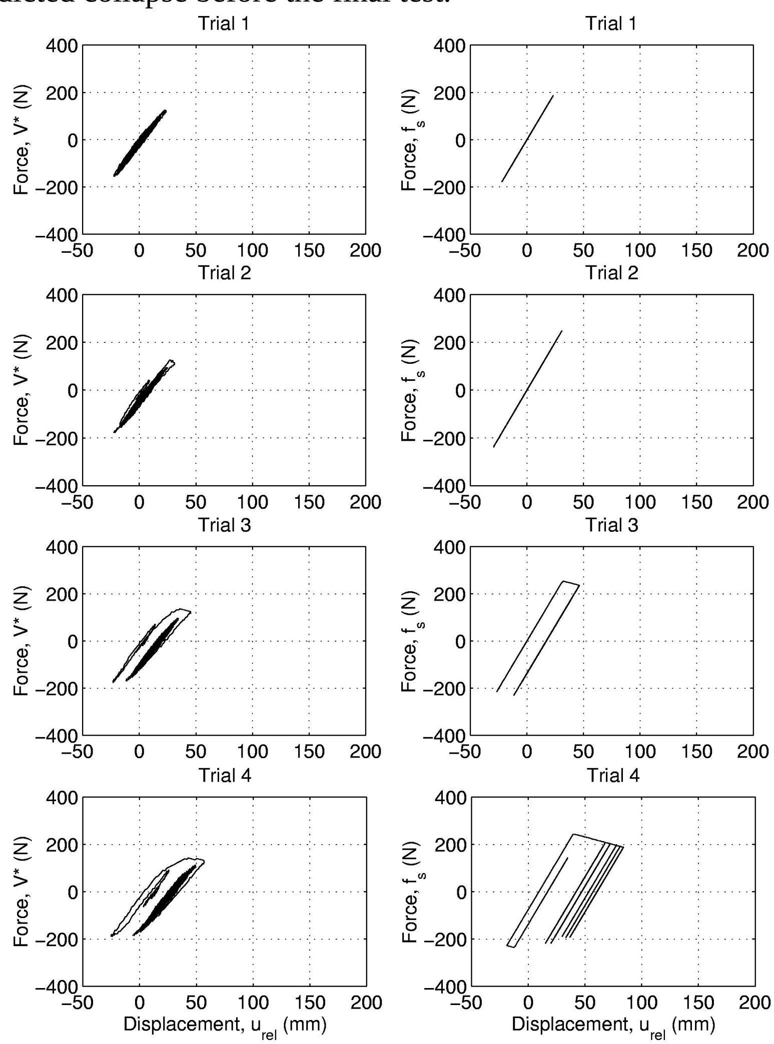

Figure 6a. Force-Displacement – Specimen 11 – Experimental vs. Method 2 (Tests 1 – 4)

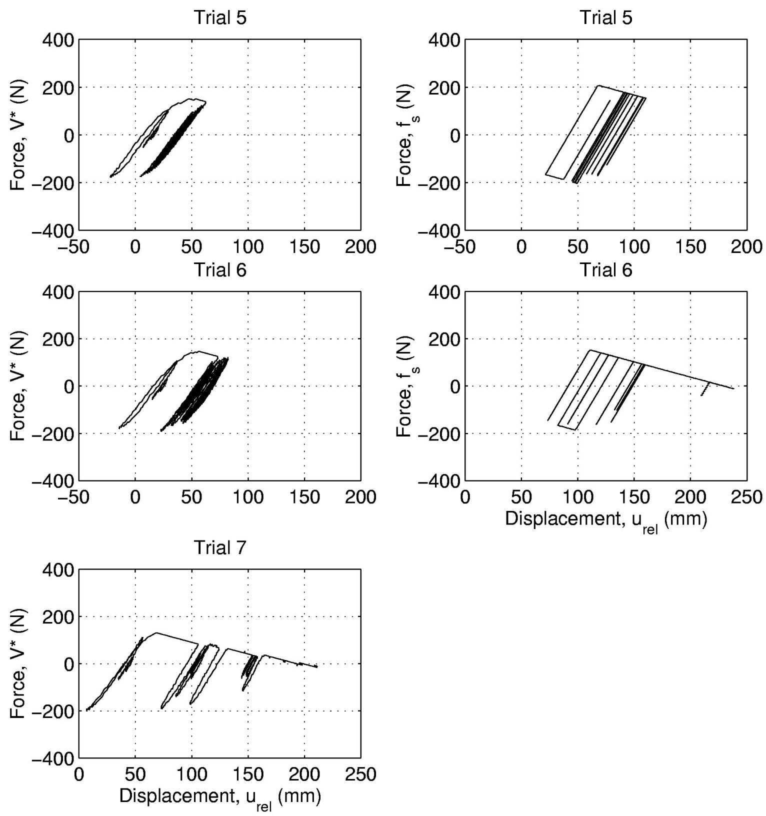

Figure 6b. Force-Displacement – Specimen 11 – Experimental vs. Method 2 (Tests 5 – 7)

Note that the specimens tested exhibit the dynamically unstable behavior characteristic of a negative hysteresis center curve (HCC), a concept of analyzing the stability of hysteresis loops proposed by MacRae and Kawashima (1993). As a result, the system has a tendency to drift in a given direction once yielding has started, resulting in large cumulative residual displacements and lower cyclic energy absorption capability prior to failure.

TABLE 1. Yield Force Reductions – Specimen 11

Test	**V_{yp}' (N)**	**u_r (mm)**	**ΔF_y (N)**	**V_{yp}' (N)**	**u_r (mm)**	**ΔF_y (N)**
	Method 1			**Method 2**		
1	255.0	0.7	-0.8	255.0	0.0	0.0
2	254.2	3.1	-3.4	255.0	0.0	0.0
3	251.6	12.8	-14.2	255.0	17.0	-18.9
4	240.8	24.3	-27.0	236.1	60.7	-67.4
5	228.0	35.7	-39.7	187.6	91.5	-101.6
6	215.3	63.2	-70.2	153.4	∞	-
7	184.8	∞	-	-	-	-

Damping is underestimated for part of the analyses, as discussed elsewhere (Vian and Bruneau 2001), resulting in larger displacements and premature collapse obtained analytically.

Note that these analyses are used for illustration purposes only. It is expected that more accurate and refined analytical models will be used, calibrated and developed to match, until collapse, the data from these benchmark experiments. Some work in this direction has already been conducted (Sivaselvan and Reinhorn 2001).

Conclusions

Data was gathered in a previously described experimental shake table test program of specimens subjected to earthquake ground motions up to collapse (Vian and Bruneau 2001).

The stability factor, θ, was observed to have the most significant effect on the structure propensity to collapse. As θ increases, the maximum attainable ductility, sustainable drift, and spectral acceleration, which can be resisted before collapse, all decrease. When this factor was larger than 0.1, the ultimate values of the maximum spectral acceleration, displacement ductility, and drift reached before collapse were all grouped below values of 0.75 g, 5, and 20%, respectively. Stability factor values less than 0.1 tended to increase each of those response values significantly. More tests should be performed to more accurately quantify the impact of this factor over various ranges, and in combination with other parameters.

Additional studies on the effect of nonlinear damping on the specimens, as well as on the observed response compared to that predicted using design equations, beyond the scope of this paper, have been presented elsewhere (Vian and Bruneau 2001)

Acknowledgments

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References

Charney, F. A., (1998), "NONLIN: Nonlinear Dynamic Time History Analysis of Single Degree of Freedom Systems," Federal Emergency Management Agency.

MacRae, G. A., Kawashima, K., (1993) "The seismic response of bilinear oscillators using Japanese earthquake records," Journal of Research, 30, Public Works Research Institute, Tsukuba, Japan.

MacRae, G. A., Priestley, M.J.N., Tao, J., (1993) "P-Δ Effects in Seismic Design," Report No. SSRP-93/05, Department of Applied Mechanics and Engineering Sciences, University of California, San Diego.

Sivaselvan, M. V. and Reinhorn, A.M. (2001) "Large Inelastic Deformations Analysis of Planar Frames," *J. Struct. Engrg.*, ASCE, *Submitted for Review and Possible Publication*.

Vian, D. and Bruneau (2001), "Experimental Investigation Of P-Delta Effects To Collapse During Earthquakes," *Technical Report MCEER 01-0001*, Multidisciplinary Center For Earthquake Engineering Research, University at Buffalo, Buffalo, N.Y. http://civil.eng.buffalo.edu/users_ntwk/experimental/case_studies/vian