

Experimental Investigation of P-Delta Effects to Collapse during Earthquakes

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Summary

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 drift limits for the minimization of P- Δ effects in highway bridge piers. The stability factor θ is found to be strongly related to the relative structural performance in this regard. Specimens for which values of θ are less than 0.25 satisfied the criterion proposed by the National Cooperative Highway Research Program (NCHRP) for at least a portion of their respective test schedule. The drift limit is violated prior to the final test, in which complete specimen collapse occurs.

Introduction

Arbitrary lateral drift limits prescribed by modern design codes to limit nonstructural damage also indirectly ensure that structural performance is minimally affected by the effect of gravity loads on the lateral force resisting system of the structure (a.k.a. P- Δ effect). However, as conventional drift limits are being progressively eliminated and replaced by other performance-based limits, inelastic behavior is to a greater extent relied upon the dissipation of seismic input energy. Accurate quantification of the destabilizing effect of gravity loads is therefore becoming more significant for structural design. As a result, it may be desirable to investigate the destabilizing effect of gravity loads in order to enhance our understanding of the condition ultimately leading to structural collapse and to ensure public safety during extreme events.

A recently published report (Vian and Bruneau 2001) discusses a program of shaking table testing of simple frames through collapse. This series of tests provides well-documented data (freely available at http://civil.eng.buffalo.edu/users_ntwk/experimental/case_studies/vian), which may be used to develop or validate algorithms capable of modeling the inelastic behavior of steel frames up to collapse. Fifteen specimens having different properties were tested in an attempt to identify some of the general parameters responsible for trends in the behavior of SDOF structures to collapse under earthquake loading. Peak responses are compared with a drift limit proposed by other researchers to minimize P- Δ effects in bridge piers. Experimental results are also compared with results obtained using a simple analytical model in order to illustrate the use of the generated experimental data for the development and calibration of models of inelastic behavior to collapse.

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. Figure 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 K_o , ignoring P-Δ effect, equal to $12 E I / L^3$. A bilinear SDOF model is shown in Figure 1b. Elastic-perfectly plastic structural response (neglecting P-Δ effect) is shown, as well as the response modified by the influence of P-Δ effect. A summary of additional concepts on P-Δ effects on simple structures during earthquakes can be seen in MacRae et al., (1993).

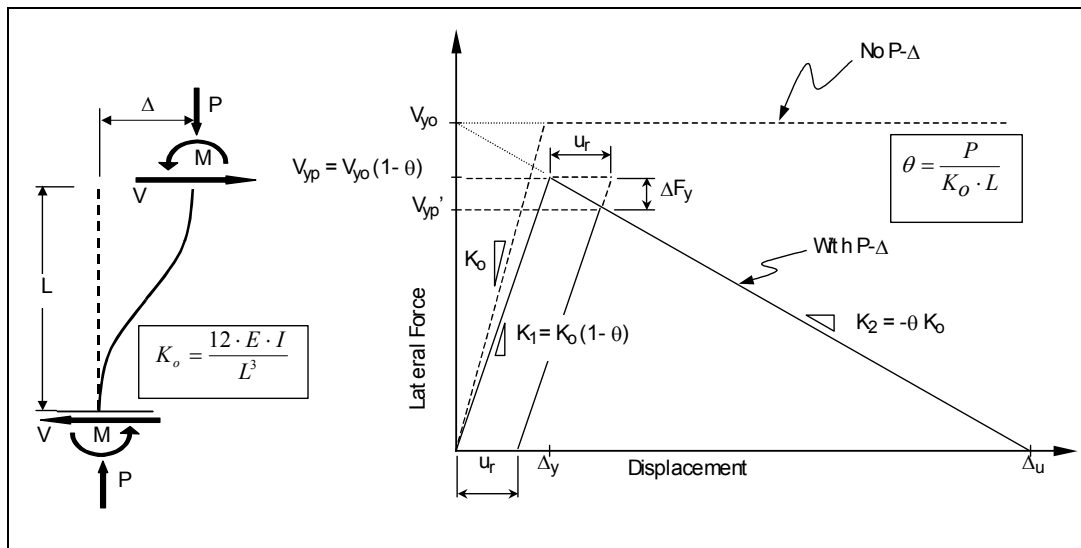


Figure 1. Bilinear lateral-force-versus-displacement model for SDOF structure

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 whose results are analyzed in this paper. The 1940 El Centro S00E ground acceleration time history was used in this study. A displacement record was then generated and used 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 rather than intended to be scaled models of actual structures.

Description of Specimens

Fifteen four-column specimens were fabricated at the University of Ottawa and tested to failure at the UB-SEESL in the course of this research. The specimens were divided into three groups of five specimens each. The slenderness ratio of the specimens of each group was equal to 100, 150, and 200, respectively. A range of values of capacity/demand ratios for axial load in columns was chosen for each slenderness ratio. In order to fully document considered imperfections, each column of each specimen was measured in numerous ways prior to testing (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 for estimation of forces during testing. Displacement transducers (LVDTs) were used to measure table displacements, vertical mass displacements and total horizontal mass displacements. Special modifications to the LVDTs setup were required to allow measurements of structural mass displacements during the entire response history, including much of the collapse. A schematic of the test setup and instrumentation is shown in Figure 2.

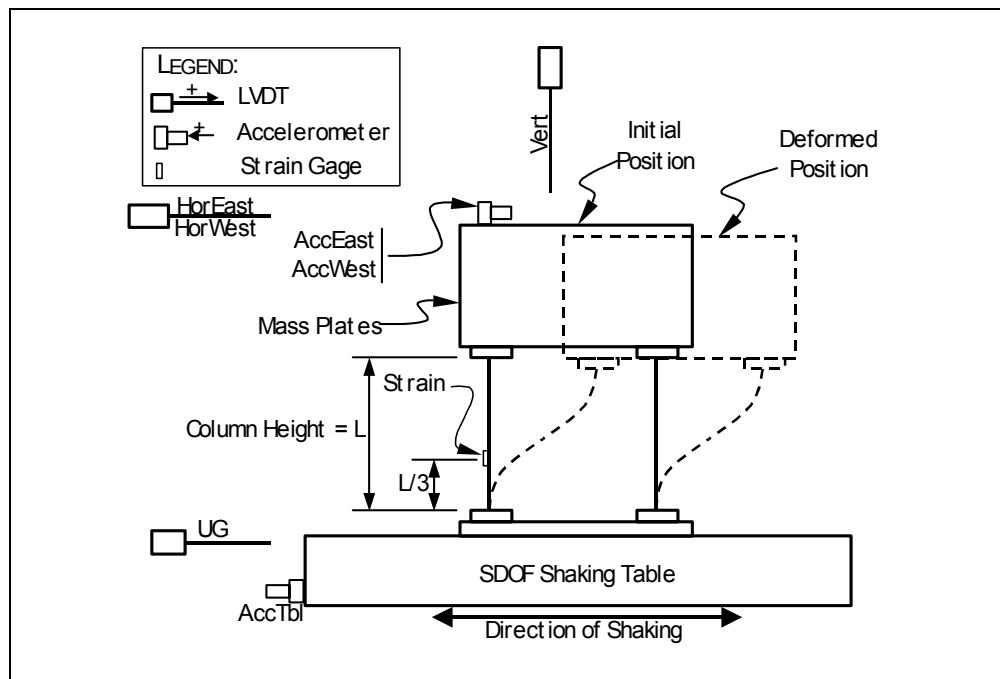


Figure 2. Schematic of test setup and instrumentation

Shaking Table Testing

A free vibration test was performed on each specimen prior to the initiation of its respective schedule of shaking table tests. Shaking table test schedules were established for each specimen, creating a series of progressively more severe shaking table tests until the specimen collapsed. Approximately five levels of ground motion intensity were selected and applied to each specimen. All specimens were 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 actual bridge and building structures, the value of θ is unlikely to be greater than 0.10 and is generally less than 0.06 (MacRae et al., 1993). Specimen 1 was found to be the only one whose value of θ ($= 0.065$) was close to the abovementioned range. Specimens 2, 6 and 11 have stability factors

whose values are slightly larger than the likely upper limit ($= 0.123, 0.101$ and 0.138 , respectively). For all other specimens, $\theta \geq 0.155$.

Results of a graphical study of peak response parameters are summarized below. Values of three response parameters (spectral acceleration $S_{a-final}$, displacement ductility μ_{final} and drift γ_{final}) obtained from the penultimate shaking table test were compared with the stability factor θ . These response quantities were also compared with the static stability limit μ_s , which is defined as the structure's ductility at ultimate displacement Δ_u . From the information given in Figure 1, μ_s can be shown to be the inverse of the stability factor. The following general observations can be made:

1. 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 Figure 3 for the penultimate test (subscript "final"). The plot 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 had the lowest values of θ , were the only specimens able to withstand spectral accelerations greater than $0.75g$.
2. Specimen 1 was the only specimen that underwent both a value of ductility greater than five ($= 20.35$) and a value of drift larger than 20% ($= 64\%$) prior to collapse. It is recalled that specimen 1 is the only specimen whose value of θ is less than 0.10.

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

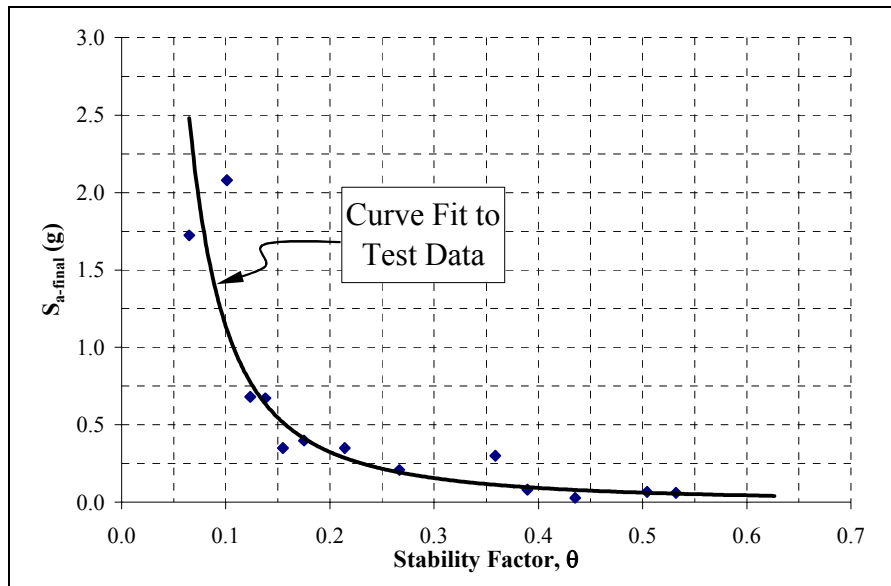


Figure 3. Spectral acceleration vs. stability factor

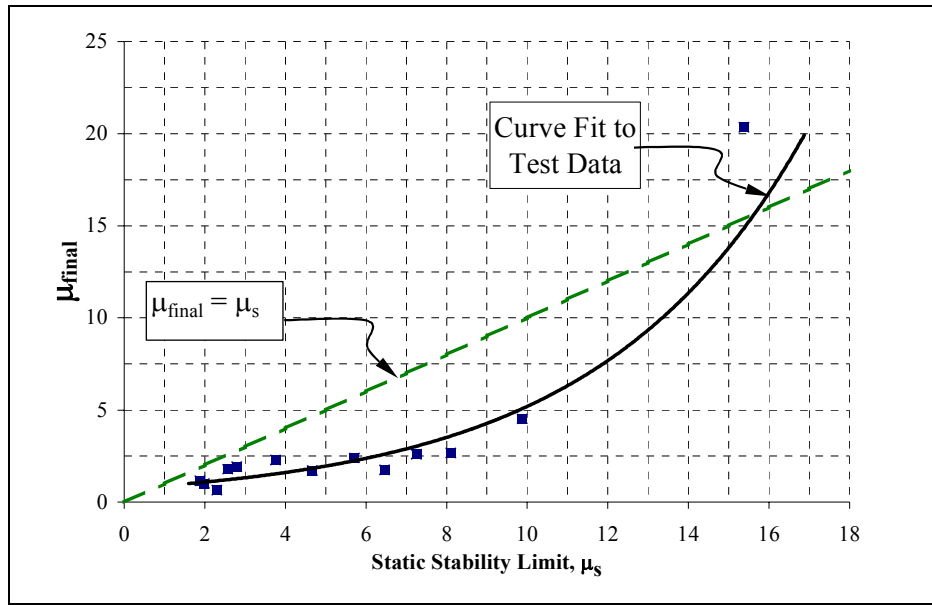


Figure 4. Displacement ductility vs. static stability limit

Overall, ultimate inelastic behavior is shown in 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 θ greater than 0.10, a relatively low level of inelastic behavior was exhibited before collapse. Specimens for which the value of θ is less than 0.10 were able to withstand ground motions with higher spectral accelerations, experience larger values of ductility and accumulate larger drifts than specimens having a value of θ greater than 0.10. Very slender structures, characterized by large values of θ , will undergo relatively small inelastic excursions prior to collapse.

Comparison with 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 the 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. Studies on additional demands on the structure arising from P- Δ effect are included in the proposed revisions.

Figure 5 shows a comparison between the proposed drift limit and peak experimental responses. The estimated base shear coefficient C_s^* is plotted as a function of the maximum drift γ ($= \Delta/H$). Specimens for which the value of θ is less than 0.25 (specimens 1, 2, 4, 6, 7, 11 and 12) are shown in Figure 5a. None of these specimens failed during the initial tests, where the proposed drift limit was not exceeded. However, in the subsequent tests of the schedule, cumulative drifts of the specimens increased due to repeated inelastic action, which eventually caused progressive collapse and violation of the proposed drift limit. Collapse always occurred only after the drift limit was exceeded in a prior test, thus validating the proposed criterion.

As shown in Figure 5b, the remaining specimens, for which the value of θ is greater than 0.25, never satisfied the drift criterion, even for those tests that remained in the elastic range. However, values of θ for these specimens were well above the practical range discussed previously, hence violation of the drift limit is of no consequence.

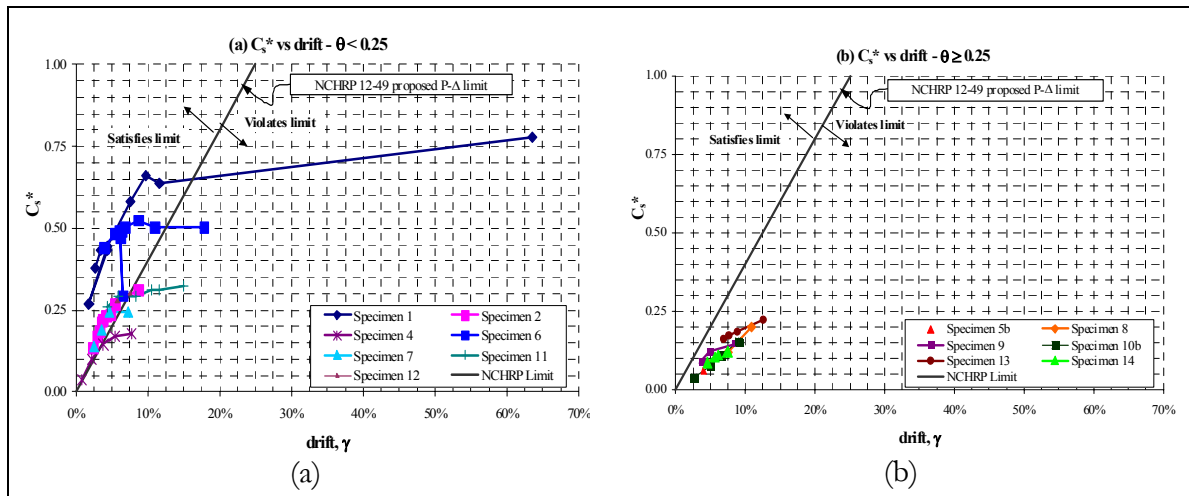


Figure 5. Comparison between test results and NCHRP 12-49 drift limits

Concluding Remarks

Data was gathered through an experimental shaking table test program of specimens subjected to earthquake ground motions up to collapse (Vian and Bruneau 2001). This type of experiment has previously not been carried out.

The stability factor θ was observed to have the most significant effect on the structure's propensity to collapse. As θ increases, the level of maximum attainable ductility, sustainable drift and spectral acceleration that can be withstood before collapse decrease. For values of θ larger than 0.10, ultimate values of 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 0.10 tended to significantly increase each of the aforementioned response values. More tests should be performed to more accurately quantify the impact of the stability factor on the response quantities considered in this study and other possibly relevant response quantities.

Additional studies on the effect of nonlinear damping on the specimens, as well as on the comparison between the observed response and the response predicted by design equations, are beyond the scope of this paper but have been presented elsewhere (Vian and Bruneau 2001).

Acknowledgements

This research was carried out under the supervision of Professor Michel Bruneau and was supported in part by the Earthquake Engineering Research Centers Program of the National Science Foundation under Award Number ECC-9701471 granted to the Multidisciplinary Center for Earthquake Engineering Research.

References

MacRae GA, Priestley MJN, Tao J (1993): P- Δ effects in seismic design. *Report SSRP-93/05*, Department of Applied Mechanics and Engineering Sciences, University of California at San Diego, La Jolla, CA.

Vian D, Bruneau M (2001): Experimental investigation of P-Delta effects to collapse during earthquakes. *Technical Report MCEER-01-0001*, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, NY.