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Using strong motion recordings to construct pushover curves

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Abstract

In several recent case studies of buildings with strong motion earthquake recordings, data has been evaluated to estimate the sequential peak cyclic force-displacement relationship of individual modes of building vibration. The process uses some old classical methods of record evaluation to supplement current electronic computer methods. By plotting the results, a pushover curve for one or more modal responses of the building can be estimated. From these graphical plots, global elastic limits and ductility demands of the earthquake on the subject modes of vibration can be approximated. Results can be used to compare analytical evaluations of mathematical models to observed performance of the building.

Introduction

Prior to 1971, there was a limited number of strong motion recordings of buildings responding to earthquakes. At the time of the San Fernando, California, earthquake of February 9, 1971, many buildings were instrumented in the Los Angeles area. Fortunately, most of the instruments worked and a good supply of building response records were produced (Murphy, 1973). Since that time the supply of available strong motion building records has increased, and during the Northridge, California earthquake of January 1994, hundreds of additional records were produced. As instruments and methods of processing improve, there is now an abundance of easily available electronically digitized strong motion data. Because so much information is available, there has been a tendency to evaluate the data using high speed computer-aided techniques. However, these evaluation techniques can be supplemented by recovering additional information in simple ways that may seem "old fashioned" in our computer dominated era. While system identification can expedite the determination of response values, simple hand calculations and visual identification can be used to enhance our understanding of building response. These low-tech methods can be of assistance in understanding observed damage, identifying higher vibrational mode effects, verifying modeling assumptions, and evaluating nonstructural elements (Gilmartin, et al. 1998). This paper emphasizes the use of strong motion recordings to construct pushover curves.

The term "pushover" is used to describe the analytical procedure that helps to identify the sequence of yielding components and redistribution of forces in a structure when subjected to lateral loading that exceeds the elastic capacity of the structure. The results are usually plotted in terms of lateral force (V/W) and lateral roof displacement of the roof (Δ_R). It can also be plotted in terms of spectral acceleration (S_a) and spectral displacement (S_d) for use in the capacity spectrum method (CSM) of evaluating structures (Freeman, 1998). In recent years, the pushover procedure has become accepted as a useful tool for a variety of approximate methods of evaluating inelastic response of buildings to earthquake ground motion. Like most engineering procedures, the pushover analysis is not an exact science. Approximations and engineering assumption must be made for each structural component in terms of strength, stiffness, and deformation characteristics. Give the same structure to four engineers/researchers and you will most likely get four different pushover curves. Therefore, it would be useful to study recorded motion to determine if we can improve pushover techniques and learn a little more how buildings respond to strong motion earthquake motion.

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Three case history examples are being presented to illustrate how pushover data can be obtained from strong motion building recordings. The first example is a full-size, four-story reinforced concrete frame structure built in the Nevada Test Site that had been used for underground nuclear explosions in the 1960's and 70's. Test data was obtained from field testing of the structure as well as from strong motion instrumentation of response to ground motion caused by nuclear explosions. The second example is a Van Nuys hotel structure whose motion was well instrumented during both the 1971 San Fernando and the 1994 Northridge earthquakes. The pushover data obtained from the strong motion records is compared to analytical results obtained from four other studies. The third example includes the results of a study of a tilt-up structure located in Hollister, California that was well-instrumented for several earthquakes.

The Process of Reading Records

Currently, strong motion is recorded electronically in a digitized format, with multiple time synchronized channels. It is relatively easy to add, subtract, and compare channels at various locations on a building and to plot time histories in acceleration, velocity, or displacement units. In earlier days, the technology of recording building motion was not as well advanced. In many cases, only hard copy paper records were available. Those that were recorded were generally on analog; time synchronization between channels was not always accurate; and processing of raw data was time consuming. Therefore, in these earlier days, a considerable amount of time was spent using hand methods on paper recordings. Modern times have allowed us to formalize and speed up the process of evaluating the strong motion recordings; however, some information is being lost by not looking more carefully into the recorded data. After subtracting out the ground motion from the motion recorded at the elevated floors of a building, it becomes easier to identify various vibration modes of the building. For example, at the roof of a building one can generally identify cycles of motion that represent the fundamental mode of vibration. The irregularities in the wave motion can be smoothed by hand to approximate the wave length (i.e., period of vibration) and the peak-to-peak amplitudes of each cycle of motion. Working in the displacement domain, the peak resonance amplitude can be approximated by one-half of peak-to-peak amplitude and the period can be approximated by measuring between peaks along the time scale (Figure 1 and reference Searer, et al. 1998). For periods, it sometimes helps to average the cycle in question with the two adjacent cycles. This process is done for all cycles that can be clearly identified. With the series of displacement (Δ_R) vs period (T) data, the acceleration (a_R) can be approximated by $a_R = \Delta_R\ (2\pi/T)^2$. The next step is to superimpose (by hand) approximate sine waves using amplitude a_R and period T on the corresponding recorded acceleration time history. The recorded acceleration time-history plot is likely to have substantial high frequency motion from higher modes which tend to overshadow the fundamental mode. If the superimposed a_R waves tend to bisect the high frequency wave motion and flow along with the time history, it can be assumed that the values obtained from the displacement time-history are valid. If not, try again. Verification can be obtained by looking at other floor levels for consistency of results.

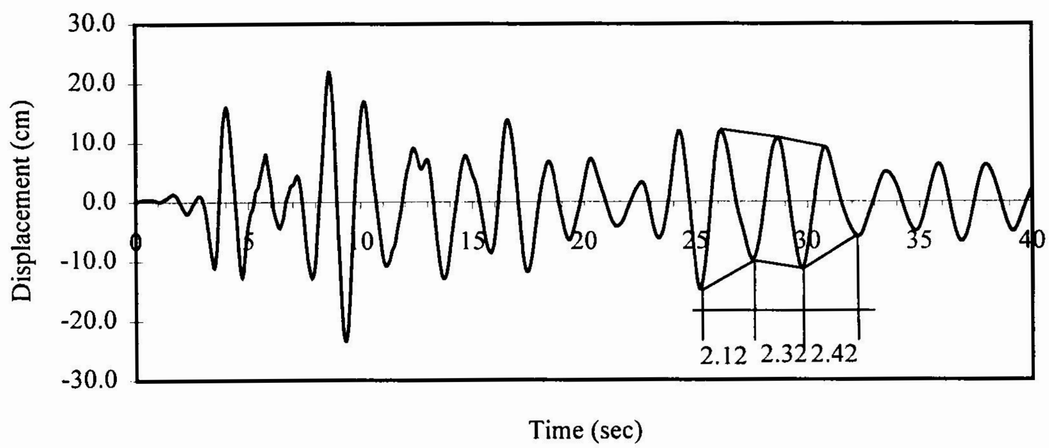

Figure 1. Sample measurements for obtaining displacement and period data points for the pushover curve

A modal response may reach a peak, disappear and then reappear with a lengthened period. This generally indicates some inelastic response, possible damage, and a change in structural response properties. Further verification can be obtained by comparing results at different floor elevations. Once the process is complete, the data points for Δ_R and a_R can be plotted sequentially in time. This plot represents a pushover curve where Δ_R represents the lateral displacement and a_R (acceleration) represents the lateral force when multiplied by an effective mass. The next step is to convert Δ_R and a_R to spectral displacement (S_d) and spectral acceleration (S_a) by use of modal participation factors (PF). The PF is the ratio of roof response to the response of a representative single-degree-of-freedom with the same period of vibration. This ratio, which is a function of mass and deformed shape, is generally between 1.3 and 1.4 for regular multi-story buildings (Freeman, 1998).

Four-Story Test Structure

Two four-story reinforced concrete frame structures were constructed by the U.S. Atomic Energy Commission in Area 1 of the Nevada Test Site (NTS) for the specific purpose of structural response investigations (Freeman, 1971). These structures, completed in March, 1966, have been subjected to ground motion generated by NTS events (i.e., underground nuclear explosions) and by imposed structural vibration field tests. The case-history presented in this paper covers the longitudinal direction of the south structure.

The test structures were designed and built to conform to the 1964 Uniform Building Code (UBC) at a lateral force level twice that required for a building located in a zone of high seismicity (i.e., Zone 3 in the 1964 UBC). The resulting design lateral force was 9.1 percent of the weight of the building (i.e., 9.1 percent of gravity). Although ductile detailing of concrete frame structures were not yet adopted by the UBC, some of the details being proposed for future codes were incorporated into the design. The structures were 12 feet by 20 feet in plan with four 9-foot stories. The four columns were 14 inches by 16 inches. Perimeter beams, 15-inches deep longitudinal and 12-inches deep transverse, supported the six-inch thick concrete slab. A post design evaluation indicated that allowable stresses would be reached at lateral forces of about 18 percent of gravity and yielding could be expected at about 27 percent gravity. This can be translated to spectral accelerations (S_a) of roughly 0.11g (design), 0.22g (allowable), and 0.33g (yielding). Note that the effective mass of the first mode is 0.83 of the total mass (e.g., 0.83 x 0.11g = 0.091g).

Within two weeks of completion, the structure was subjected to a ground motion from an underground explosion which resulted in peak spectral accelerations of about 0.05g and a fundamental response period of 0.372 seconds. About 10 days later field tests were performed by pulling on the structure, and then quickly releasing the load to measure free vibration. The equivalent fundamental mode response spectral acceleration was about 0.009g with a period of 0.366 seconds. This was the first indication of a period dependency on amplitude. In May 1966, the structure was again subjected to another underground event, this time spectral accelerations were up to 0.31g and periods measured 0.45 seconds. Over the next few years, the structure continued to be recorded for motion caused by field tests and underground events (Freeman, et al. 1976). Data covering the period from March 1966 to April 1969 is summarized in Figure 2. The points mark representative spectral accelerations and displacements for each event in chronological order. Note that after points c and d, there is a slight reduction in stiffness which may be attributed to the structure approaching yield limits and minor hairline cracking in beam-column joints. The

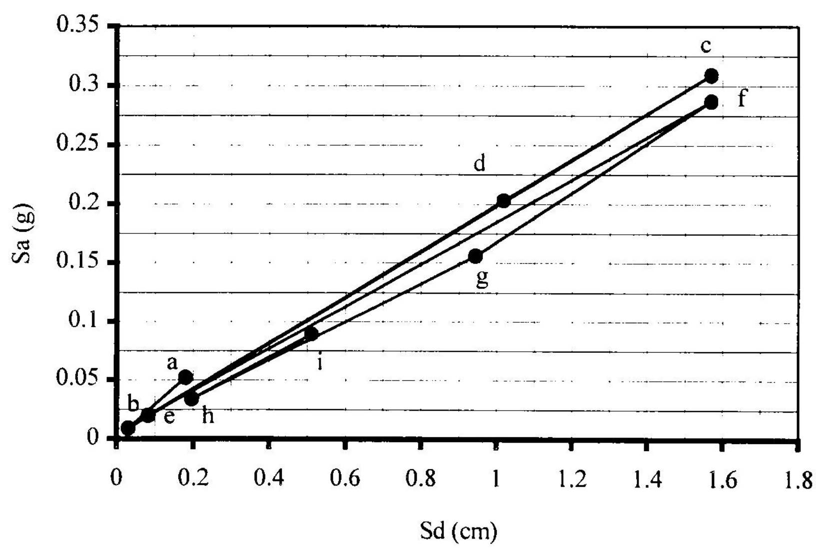

Figure 2. 4-story structure: Field Tests and measured events 1966-1969

periods of vibration relate to the slope of the S_a vs. S_d plot (i.e., $T = 2\pi (S_d/S_a)^{1/2}$) and elongated from an initial period of 0.37 seconds to a period of 0.49 seconds at point g.

In 1974, a field test program was developed to perform a destructive test on the south test structure (Czarnecki, et al. 1975). A reciprocating mass vibration generator was designed and built for this test that had a maximum displacement of 20 cm, peak-to-peak. The planned procedure for the destructive test was to tune the vibration generator to the structure's natural frequency. The input force would then be gradually increased until significant structural damage was observed. However, because the design of the automated vibration generator controls did not anticipate the complexities of nonlinear response, a gradual increase of force was not achieved. By use of hand controls, maximum displacements and moderate localized spalling was finally achieved. After the structure had sustained damage and its natural period had nearly doubled, the condition of the structure seemed to stabilize. Because of the limited displacement capacity of the vibration generator, the force necessary to cause additional damage could not be produced. The results of the "destructive" test is summarized in Figure 3 by points #1 through #12. Note that the data from Figure 2 is also included in Figure 3. As S_a exceeded 0.3g and S_d exceeded 2 cm, there was a loss in stiffness due to cracking and possible yielding of reinforcing steel. After reaching S_a of 0.45g and S_d of 5 cm, a dramatic reduction of stiffness occurred resulting in a period lengthening from 0.7 seconds to 0.9 seconds. However, the structure was still stable and was able to reach S_a equal to .47g.

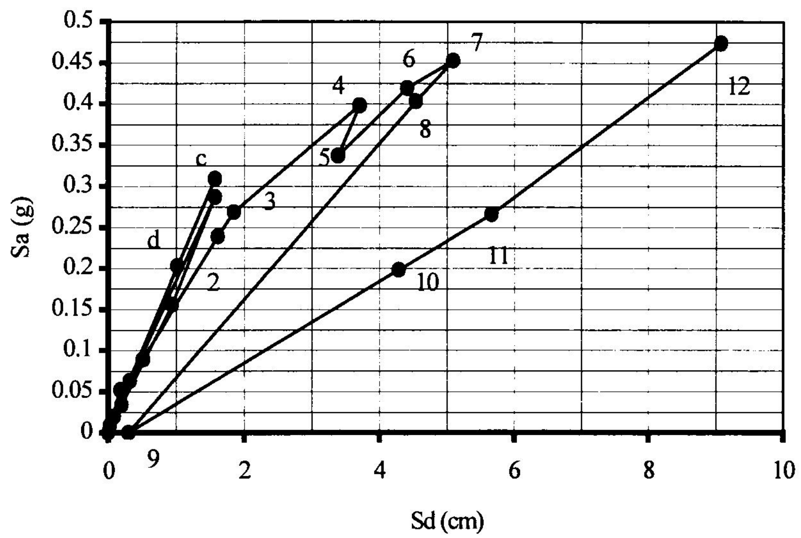

Figure 3. 4-story structure: Strong motion vibration generator test, 1974

Seven-Story Hotel

The seven-story hotel, a reinforced concrete frame structure designed in 1965, is located at 8244 Orion Avenue in Van Nuys, California. It is 62 feet by 160 feet in plan with column spacing at about 20 feet in each direction. The floor framing system consists of 8 to 10 inch thick flat slabs with perimeter beams. A more complete building description can be obtained from Freeman and Honda, 1973 and Murphy, 1973. For the 1994 Northridge earthquake sixteen channels of strong motion data is available and for the San Fernando earthquake nine channels of data were obtained. This building has been the subject of many studies and has been evaluated by many researchers.

The building was damaged by the 1971 earthquake (Murphy 1973). Although the damage was reported to be primarily nonstructural, results of analytical evaluations indicated some degree of inelastic response (Freeman 1978). During the 1994 Northridge earthquake the structure was severely damaged (Gilmartin, et al. 1998). Analysis of the 1971 strong motion records in the longitudinal direction indicated that the structure had an initial fundamental period of about 0.7 seconds, went into the inelastic range of response, and responded at a fundamental period of 1.5 seconds during its maximum excursions. An aftershock recorded a period of 1.2 seconds for a moderate response in the elastic range. Analysis of the 1994 longitudinal strong motion records indicate an initial period of about 1.2 seconds, approaching a period of 1.5 seconds until there was a significant loss of stiffness and the period elongated toward the 2.0 second range.

The strong motion records were analyzed by the process described earlier. The 1971 analysis used data from earlier studies (Freeman and Honda 1973 and Freeman 1978). The analysis of the 1994 data is described in Gilmartin, et al. 1998. The results were converted to S_a and S_d coordinates and are summarized in Figure 4. The first four points

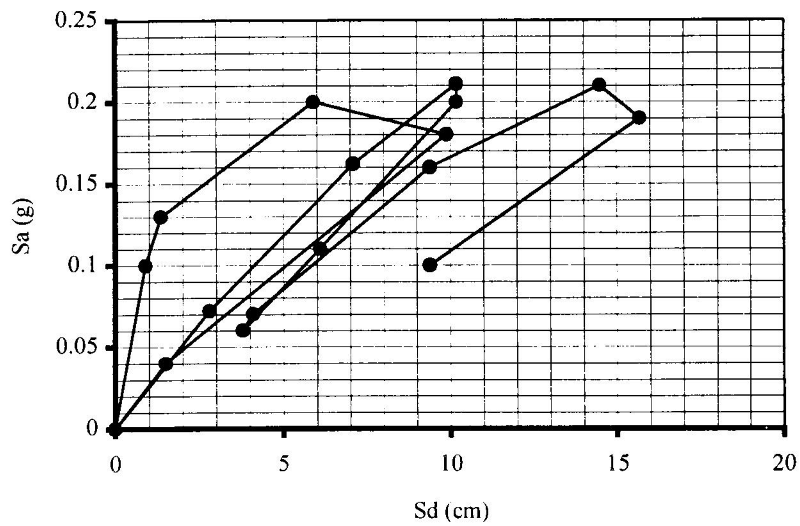

Figure 4. 7-story hotel: Pushover curve from strong motion recordings

on the curve represent the measured pushover data from the February 1971 earthquake. The fifth point (S_a = 0.04, S_d = 1.5) represents the aftershock in March 1971. The balance of points represent the first ten seconds of the 1994 earthquake. An upper bound envelope of the plot would represent an equivalent pushover curve of the capacity of the building.

For comparison, analytically developed pushover data from other researchers were converted into S_a and S_d coordinates. A summary of these results are shown in Figure 5. Although there is a large degree of variance, the results do fit into a broad band that can be compared to the measured pushover in Figure 4.

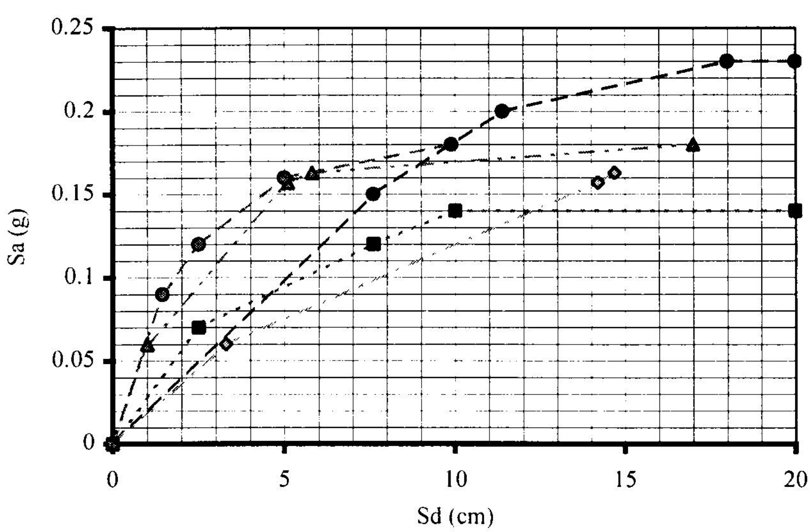

Figure 5: 7-story hotel: Pushover curves from independent analyses

Tilt-Up Building with Wood Diaphragm

This case history is based on an evaluation of a tilt-up building located in Hollister, California, using strong motion data from the 1984 Morgan Hill, 1986 Hollister, and 1989 Loma Prieta earthquakes. The project was funded by the National Science Foundation (Freeman, et al. 1997). The building is a typical single-story concrete panel tilt-up-wood roof diaphragm system with plan dimensions of 100 feet by 300 feet. In order to better understand the characteristics of the diaphragm response under the three earthquakes, the diaphragm periods, T_d, and the corresponding amplitudes were plotted for all clearly recognizable cycles for each of the three events. The amplitudes and periods were used to calculate spectral accelerations and displacements. By plotting the corresponding spectral acceleration (S_a) and displacements (S_d), as shown in Figure 6, it is possible to develop the force-displacement relationship for the diaphragm, with force as a function of S_a and displacement as a function of S_d.

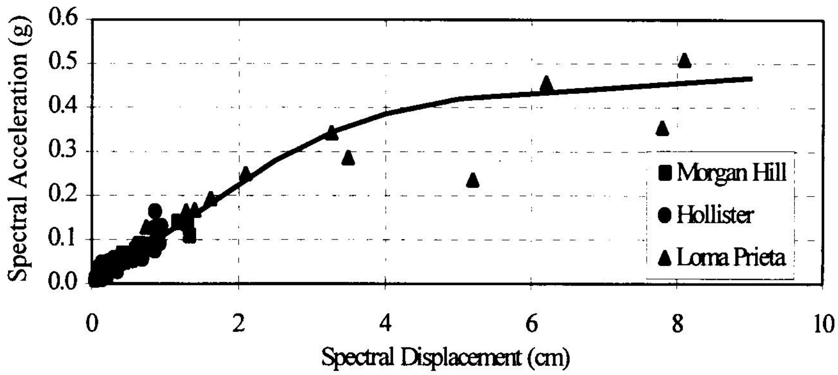

Figure 6. Force-displacement diaphragm response

Conclusions

We have presented three case studies illustrating how pushover data can be obtained from strong motion building recordings. It is our opinion that this is a useful tool for evaluating buildings and provides a learning process for engineers to better understand how buildings respond during earthquakes.

References

Czarnecki, R.M., S.A. Freeman and R.E. Scholl, 1975, *Destructive Test of a 4-Story Concrete Structure*, Fifth European Conference on Earthquake Engineering, Istanbul, Turkey, September 1975.

Freeman, S.A., 1971, *Concrete Test Structures: Second Progress Report on Structural Response*, John A. Blume & Associates Research Division, JAB-99-50, San Francisco, California, July 1971.

Freeman, S.A., 1978, *Prediction Response of Concrete Buildings to Severe Earthquake Motion*, Douglas McHenry International Symposium on Concrete and Concrete Structures, Publication SP-55, American Concrete Institute, Detroit, Michigan.

Freeman, S.A., 1998, *Development and Use of Capacity Spectrum Method*, Paper No. 269, the 6th US NCEE Conference on Earthquake Engineering/EERI, Seattle, Washington, May 31-June 4, 1998.

Freeman, S.A., C.K. Chen and R.M. Czarnecki, 1976, *Dynamic Response Characteristics of Reinforced Concrete Structures*, Proceedings of ASCE-EMD Specialty Conference "Dynamic Response of Structures: Instrumentation, Testing Methods, and System Identification", pp. 210-219, University of California, Los Angeles, California, March 1976.

Freeman, S.A., U.M. Gilmartin and G.R. Searer, 1997, *Tilt-Up Construction: Proposed Seismic Design Procedures Developed from Strong Motion Research*, 1997 CUREe Northridge Earthquake Research Conference, Los Angeles, California, August 20-22, 1997.

Freeman, S.A. and K.K. Honda, 1973, *Response of Two Identical Seven-Story Structures to the San Fernando Earthquake of February 9, 1971*, John A. Blume & Associates Research Division, JAB-99-98, San Francisco, California, October 1973.

Gilmartin, U.M., S.A. Freeman and S.S. Rihal, 1998, *Using Earthquake Strong Motion Records to Assess the Structural and Nonstructural Response of the 7-Story Van Nuys Hotel to the Northridge Earthquake of January 17, 1994,* Paper No. 268, the 6th US NCEE Conference on Earthquake Engineering/EERI, Seattle, Washington, May 31-June 4, 1998.

Murphy, L.M., 1973, *San Fernando, California, Earthquake of February 9, 1971*, Volume I-Effects on Building Structures, Part A, U.S. Department of Commerce, Washington, D.C., 1973.

Searer, G.R., S.A. Freeman and U.M. Gilmartin, 1998, *Use of Building Motion Records to Evaluate Building Response and Damage*, Paper No. 267, the 6th US NCEE Conference on Earthquake Engineering/EERI, Seattle, Washington, May 31-June 4, 1998.