

USE OF STRONG-MOTION DATA  
IN EARTHQUAKE RESISTANT DESIGN

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ABSTRACT

The study first reviews some simple methods of calculating maximum displacement, base shear and interstory drift from acceleration response records. A record from a 17-story building in Los Angeles obtained during the Northridge earthquake is used for illustration. More detailed analyses of similar records are suggested as attractive subjects for Master's theses and senior projects. Simple analyses of records from Northridge and other earthquakes convincingly indicate base shears and drifts that are substantially in excess of those used in design. The author urges that earthquake resistant design be based increasingly on measurements of earthquake response and measured properties of materials, and less on empiricism and qualitative assessments of earthquake performance.

INTRODUCTION

The records obtained from the earthquake response of structures are the basic data against which methods of earthquake resistant design and techniques for the analysis of earthquake response must be judged. The records are also useful for quickly informing the technical communities involved in earthquake damage assessment and recovery of the scope of the problems they may face. For the design engineer of a building that has been heavily shaken, the records can be helpful in assessing the likelihood of structural and non-structural damage that is not readily visible. The records also provide a means for educating the media and the public about the nature and extent of earthquake response. This paper reviews some of the simple methods whereby important results can be determined quickly from the measurements of earthquake response and notes the importance of recorded motions in improving the process of earthquake resistant design.

REVIEW OF ELEMENTARY METHODS

Figure 1 shows an example of the measurements which can be obtained from the response of a structure to strong ground motion. In this case the records are the accelerations measured at various points in a 17-story building during the Northridge earthquake of January 17, 1994 (CSMIP, 1995).

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Los Angeles - 17-story Residential Bldg.  
(CSMIP Station 24601)

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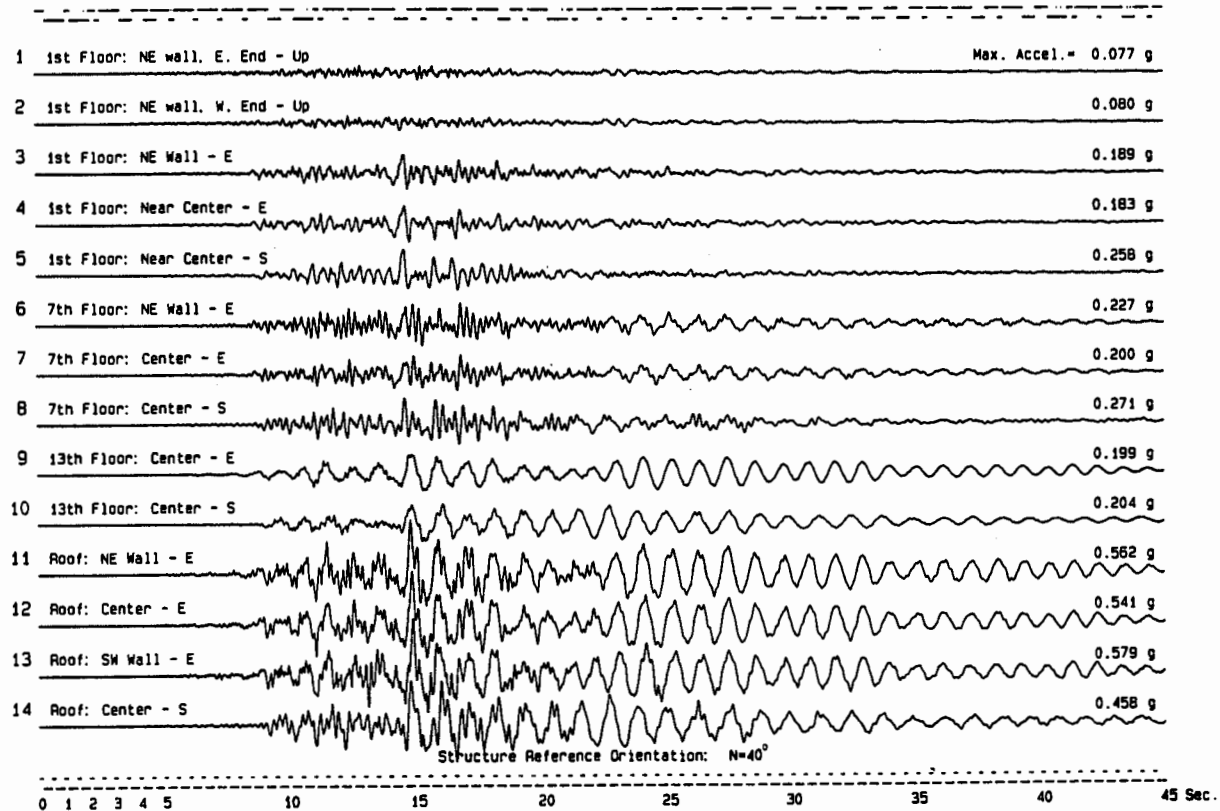


Figure 1. The acceleration measured in a 17-story residential building in Los Angeles during the Northridge earthquake of January 17, 1994 (Station 24601, CSMIP, 1995).

In accordance with common practice,  $a$  will denote acceleration,  $v$ , velocity, and  $d$ , displacement. The symbol for interstory drift will be  $\gamma$ . Attention is focused first on the acceleration recorded at the top of the structure; in some cases this and the corresponding acceleration at the base may be the only records available. One can draw a base line through the trace and use the indicated maximum acceleration to find a calibration factor if a scale factor is not given in the report. In the case of the Center-E component of the roof response (trace 12) in Figure 1, for example, a factor of  $0.077g/mm$  was found before reproduction for the PROCEEDINGS (It is a coincidence that the maximum value of trace 1 is  $0.077g$ ). Although the maximum acceleration occurred at about fifteen seconds, the oscillatory response of what appears to be the fundamental mode of the building produces almost equally large accelerations in the 22 to 35 second range. This is the portion of the response that will be examined as the largest displacement, base shear and interstory drift are expected during this time.

Applying the scaling factor to the large peaks in the Roof Center-E (trace 12) record around 25 seconds gives a value of  $0.27g$  for the maximum acceleration of the roof in the fundamental mode. This total response can also be taken as the response of

the structure with respect to its base, as the ground motion at this time (trace 4) appears small and unrelated to the response. A calculation similar to that for the acceleration shows the building to have a fundamental period of vibration of 1.1 sec in this time frame.

#### Acceleration at the Roof

The record of acceleration obtained on the roof is of direct interest, particularly to mechanical engineers because of the large amount and variety of equipment that is often installed there. The measured acceleration can be compared to the design motions for such equipment and the response spectra distributed with the processed records can be used to compare the earthquake response of equipment to values used in the design.

A comparison of the roof acceleration to the acceleration at the base of the building also provides a measure of how much the structure amplified and altered the ground motion. This comparison can help engineers answer the obvious question of how much stronger the shaking may have been for occupants of the upper floors in comparison to those near the ground. In the present example the structure amplified the motions by roughly a factor of three, increasing the pulse of ground motion at about 15 seconds from a value of 0.183g at the first floor (trace 4) to a value of 0.541g on the roof (trace 12).

#### Maximum Displacement

The acceleration in the time range of 22 to 35 seconds is essentially sinusoidal, which allows the maximum displacement at the center of the roof in this direction to be evaluated from

$$d_{\max} = \left( \frac{T}{2\pi} \right)^2 a_{\max} \quad (1)$$

Using the values for acceleration, 0.27g, and period, 1.1 sec, determined above, the maximum displacement of the 17-story building in its fundamental mode is found to be 8.2 cm. If processed records are available, scale factors are included with the plots of individual traces and this approximate calculation of displacement need not be done, as velocity and displacement calculations are part of the package of processed data. The value of the maximum displacement reported in the processed records in this case is 10.5 cm, but a value of 8.2 cm is quite representative of the maximum oscillatory displacement in the selected time period.

#### Base Shear

The base shear can also be approximated from the acceleration records with little detailed knowledge of the building. The records in Figure 1 can be used to estimate the acceleration profile of the structure during the time of maximum response (note that floors 13 and 7 (traces 9 and 7) are responding in phase with the roof in the selected time frame). However, to illustrate the most general approach, only the roof acceleration will be used; in this case one must assume an acceleration profile. In the present example it

appears justified to assume that the structure is vibrating only in its fundamental mode in the 22 to 35 second range.

In general, the base shear,  $V$ , can be written from Newton's law as

$$V = m_n a_n + m_{n-1} a_{n-1} + \dots + m_2 a_2 + m_1 a_1, \quad (2)$$

in which the subscripts refer to one of the  $n$  floor levels. Defining  $m$  as the average floor mass, Eqn. 2 can be written

$$V = m a_n \left( \frac{m_n}{m} + \frac{a_{n-1} m_{n-1}}{a_n m} + \dots + \frac{a_2 m_2}{a_n m} + \frac{a_1 m_1}{a_n m} \right). \quad (3)$$

Using the fact that the total mass,  $M$ , is  $n$  times the average mass,  $m$ , and assuming for the moment that the floor masses are all equal, the base shear as a fraction of the weight of the building becomes

$$\frac{V}{Mg} = \frac{a_n}{g} \frac{1}{n} \left( 1 + \frac{a_{n-1}}{a_n} + \dots + \frac{a_2}{a_n} + \frac{a_1}{a_n} \right). \quad (4)$$

If the structure is vibrating in the fundamental mode, the term in parentheses is just the sum of the mode shape ordinates, with the mode shape normalized so the ordinate at the roof is equal to unity. This quantity times  $1/n$  is therefore simply the average value of the fundamental mode, a quantity that does not vary much. The fundamental mode of most buildings lies somewhere between the half sine wave of a uniform shear beam, typical of short frame buildings, and a straight line, shown by tall structures. If the fundamental mode is a straight line normalized to unity at the top of the structure, the average modal ordinate is 0.5; if the mode shape is the half sine wave of a uniform shear beam, the value increases to  $2/\pi = 0.64$ . Applying this to Eqn. 4, and using the middle of the range,

$$\frac{V}{Mg} = 0.6 \frac{a_n}{g} \pm 15\%. \quad (5)$$

In Eqn. 5, the factor 0.6 should be increased if the fundamental mode is believed to be close to that of a uniform shear beam, and decreased if the mode is thought to be closer to a straight line.

For a structure with unequal floor masses, the effect of such differences is seen from Eqns. 3 and 4 to be the same as having a different mode shape. For most structures, only a few floors will have masses much different from the average, and the final result, the average value of the modal ordinates, will not be changed very much.

In the 17-story building being used as an example,  $a_n/g = 0.27$  and the mode shape is thought to be closer to a straight line than indicated by the factor of 0.6 in Eqn. 5 because shear walls provide most of the resistance for this direction of response. The floors of the structure have the same area, so the assumption of regular masses seems justified. Putting this together, the base shear experienced in the earthquake in the 22 to 35 sec time frame was about 15 percent of its weight. The base shear might have been as high as 17 percent of its weight, or as low as 13 percent, but it very unlikely to be outside this range, because the values come so directly from the measured response and from a reasonable range of relative accelerations (or displacements) of the structure at the time of maximum response.

### Interstory Drift

A similar approach can be used to estimate the maximum interstory drift indicated by the recorded motion. The additional information needed is the height of the building,  $h$ , and the interstory height,  $l_j$ . The average interstory height,  $l = h/n$ , is also introduced. Using Eqn. 1, and assuming that the floors are vibrating in phase in the fundamental mode, the maximum drift at the  $j^{\text{th}}$  level can be expressed as

$$\gamma_j = \frac{d_{j+1} - d_j}{l_j} = \frac{T^2 a_n}{4\pi^2 l_j} (\phi_{j+1} - \phi_j), \quad (6)$$

in which  $\phi_j$  is the value of the fundamental mode at the  $j^{\text{th}}$  level. In estimating the drift it is useful to look again at the cases of a straight line mode and a half sine wave mode. For a straight line mode normalized to unity at the top of the structure, the difference between modal ordinates for any two adjacent floors is  $l_j/h$ , and Eqn. 6 reduces to

$$\gamma = \frac{T^2 a_n}{4\pi^2 h} = \frac{d_{\max}}{h}, \quad (7)$$

taking the same value for all stories. In the case of a half sine wave mode,  $(\phi_{j+1} - \phi_j)/l_j$  is well-approximated by the slope of the mode shape at the height,  $x_j$ , of interest.

Applying this to Eqn. 6 gives, in comparison to Eqn. 7

$$\gamma_j = \frac{T^2 a_n}{4\pi^2 h} \frac{\pi}{2} \cos \frac{\pi x_j}{2h}. \quad (8)$$

If the fundamental mode resembles a half sine wave, the maximum drift is at the first floor level and even for a building of four or five stories, the cosine function in Eqn. 8 is near unity. Using unity for the cosine and  $\pi/2 = 1.57$ , Eqns. 7 and 8 can be combined to estimate the range of the maximum first story drift,

$$1.3 \frac{d_{\max}}{h} \pm 25\%. \quad (9)$$

For the example structure,  $d_{\max}$  was been found to be 8.2 cm and the height of the building is given (CSMIP, 1995) as 149 ft. 8 in. Using these values in Eqn. 9 gives a maximum drift between 0.2 and 0.3 percent.

Although the value of maximum acceleration experienced by the structure was quite high, 0.541g, the values of base shear, displacement and drift associated with the maximum response of the fundamental mode are sufficiently moderate that significant damage is deemed unlikely in this case.

### ADDITIONAL METHODS OF ANALYSIS

The previous section shows that much can be learned very quickly from a single acceleration record and rudimentary knowledge of the structure. Naturally, if more is known about the structure, or if additional records are available in or near the structure, more detailed analyses can be done. Two case studies of reading and interpreting strong-motion records are presented, for example, in Housner and Jennings (1982). Although more involved than what was presented above, these studies are still sufficiently simple that they can be easily done and readily explained.

Advances in the capabilities of computers and developments in software make it possible to perform easily even more involved calculations, calculations that not so long ago were feasible only as research studies. With this in mind, the author notes that studies of the measured earthquake response of selected buildings would make excellent topics for Master's theses and senior projects and suggests to his academic colleagues that they take advantage of these opportunities. Not only would such studies provide valuable experiences for the students, but they would also help structural engineers assimilate the important information in the strong-motion data base.

### IMPLICATIONS FOR EARTHQUAKE ENGINEERING PRACTICE

It is critical to realize that the earthquake records are telling us some very important things in a very direct way about what is actually happening to structures, independent of how the structures are designed, where the strength comes from, and whether the response is linear or nonlinear. The maximum base shear, for example, is dependent only on the acceleration profile in the structure at the time of maximum response. Similarly, the maximum displacement of the structure is determined by the acceleration record, even when the response is not sinusoidal. Even more information will be available in the future, when dense instrumental arrays in structures that are now in the planning stages produce records. Data from these arrays will greatly increase the

precision of calculations of structural response including interstory shears, interstory drifts and, in some cases, strains in structural members.

The 17-story building used as an example exhibited quite significant response, even though it was rather distant from the area of strongest shaking during the Northridge earthquake. Using the technique described above, the drifts and base shears in two buildings in the San Fernando Valley, one in Sherman Oaks and one in Tarzana, were calculated. These buildings were closer to the region of strongest shaking and showed periods of 3.1 and 2.5 seconds, drifts of 0.8 percent and 1.6 percent, and base shears of 8 percent and 17 percent, respectively. The drifts and base shears substantially exceed the design limits recommended, for example, by the latest SEAOC Blue Book (SEAOC, 1996). Nor were these structures in the area of strongest shaking, which is generally understood to be further to the north, where the motions may have been roughly twice as strong as they were in the southern part of the San Fernando Valley. Furthermore, earthquakes significantly larger than the Northridge earthquake have obviously occurred in California and are entirely possible in most of the urbanized regions of the state.

Such considerations and other recent studies (Hall et. al., 1996, Jennings, 1997) lead the author to conclude that structures that respond successfully to nearby major earthquakes ( $M=7$  and above), will exhibit interstory drifts on the order of 3 percent, or more, and/or base shears of 30 to 40 percent of their weight, or more. Although one may disagree with the specifics of this assessment, it does seem that the records of earthquake response are showing us convincingly that the values of excitation and response formally considered in the building codes are distinctly less than what actually happens during major earthquakes. For comparison to the numbers discussed above, SEAOC (1966) gives a design base shear for a two second period building (zone 4,  $I=1$ ,  $S=1.2$ ) of between 3 and 9 percent, depending on the reduction factor,  $R_w$ , and an allowable drift of 0.4 percent (or  $0.03/R_w$ ).

A difference between code design values and measured earthquake response of this magnitude--approaching a factor of ten--is not a tenable situation. As more records become available and understood, it seems inevitable that the process of earthquake resistant design will be increasingly, and quite appropriately, based more and more upon records and measured properties of materials, and less and less upon empiricism and qualitative assessments of earthquake performance. This process is well along now in the design of special structures.

We should work hard to speed this process of change and improvement of the building codes; not only is the appropriate level of public safety a concern, but pushing for betterment of governing codes is part of our professional responsibility. In addition, if we are proven slow in responding to the challenges of incorporating new information into earthquake resistant design practices, it will become increasingly difficult for structural engineering to attract its share of the brightest young people, given the opportunities in competing fields of technology.

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