# SMIP96

## SMIP96 SEMINAR ON SEISMOLOGICAL AND ENGINEERING IMPLICATIONS OF RECENT STRONG-MOTION DATA

Sacramento, California
May 14, 1996

## PROCEEDINGS


Sponsored by

California Strong Motion Instrumentation Program
Division of Mines and Geology
California Department of Conservation

Supported in Part by

California Seismic Safety Commission


DEPARTMENT OF CONSERVATION
Division of Mines and Geology

The California Strong Motion Instrumentation Program (CSMIP) is a program within the Division of Mines and Geology of the California Department of Conservation and is advised by the Strong Motion Instrumentation Advisory Committee (SMIAC), a committee of the California Seismic Safety Commission. Current program funding is provided by an assessment on construction costs for building permits issued by cities and counties in California, with additional funding from the California Department of Transportation, the Office of Statewide Health Planning and Development, and the California Department of Water Resources.

## DISCLAIMER

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Edited by

M.J. Huang

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California Department of Conservation
801 K Street, MS 13-35
Sacramento, California 95814-3531

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# PREFACE

The California Strong Motion Instrumentation Program (CSMIP) in the Division of Mines and Geology of the California Department of Conservation promotes and facilitates the improvement of seismic codes through the Data Interpretation Project. The objective of this project is to increase the understanding of earthquake strong ground shaking and its effects on structures through interpretation and analysis studies of CSMIP and other applicable strong-motion data. The ultimate goal is to accelerate the process by which lessons learned from earthquake data are incorporated into seismic code provisions and seismic design practices.

Since the establishment of CSMIP in the early 1970s, over 600 stations, including 400 ground-response stations, 140 buildings, 20 dams and 40 bridges, have been installed. Significant strong-motion records have been obtained from many of these stations. One of the most important sets of strong-motion records is from the 1994 Northridge earthquake. During this earthquake strong-motion records were obtained from 116 ground-response stations and 77 extensively-instrumented structures. In addition to these records, CSMIP in cooperation with the City of Los Angeles and other agencies, collected and archived accelerograms recorded at over 300 high-rise buildings during the Northridge earthquake. These buildings were instrumented by the building owners as required by the City's Building Code. The strong-motion records from the Northridge earthquake have been and will be the subject of CSMIP data interpretation projects.

The SMIP96 Seminar is the eighth in a series of annual events designed to transfer recent interpretation findings on strong-motion data to practicing seismic design professionals and earth scientists. In the oral presentations, investigators of four CSMIP-funded data interpretation projects will present the results from interpretation studies of CSMIP data during the past year. In addition, CSMIP staff will present the lessons learned from the Northridge code-record recovery and review the records from damaged buildings, and two invited speakers will present topics related to strong-motion data and damage. One paper is on the Seismic Safety Commission's Northridge Buildings Case Studies Project, and the other on post-Northridge evaluation of steel frame buildings. Director Elin Miller of the Department of Conservation will present a luncheon address on the actions the Department is undertaking to reduce seismic hazards in California.

The papers in this Proceedings volume presented by the investigators of four CSMIP-funded data interpretation projects represent interim results. Following this seminar the investigators will be preparing final reports with their final conclusions. These reports will be more detailed and will update the results presented here. CSMIP will make these reports available after the completion of the studies.

Anthony F. Shakal
CSMIP Program Manager

Moh J. Huang
Data Interpretation Project Manager

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**SMIP96 SEMINAR ON**

**SEISMOLOGICAL AND ENGINEERING IMPLICATIONS**

**OF RECENT STRONG-MOTION DATA**

Radisson Hotel, Sacramento, California
May 14, 1996

## PROGRAM

| | |
|---|---|
| 8:30 - 9:20 | **Registration** |
| 9:20 - 9:30 | **Welcoming Remarks**<br>*Jeffrey Johnson*, Seismic Safety Commission, and Chair, Strong Motion Instrumentation Advisory Committee (SMIAC)<br>*James Davis*, State Geologist, Division of Mines and Geology |
| 9:30 - 9:40 | **Introductory Remarks**<br>*Anthony Shakal*, Manager, Strong Motion Instrumentation Program |
| **SESSION I** | **Moderator:** *Chris Poland*, H.J. Degenkolb Associates<br>Chair, SMIAC Buildings Subcommittee |
| 9:40 - 10:10 | **Analysis of Strong Motion Records from a Parking Structure during the January 17th Northridge Earthquake**<br>*S. Hilmy*, *S. Werner*, A. Nisar and J. Masek, Dames and Moore |
| 10:10 - 10:40 | **Performance of 20 Extensively-Instrumented Buildings during the 1994 Northridge Earthquake**<br>*F. Naeim*, John A. Martin and Associates |
| 10:40 - 11:00 | Break |
| **SESSION II** | **Moderator:** *Bruce Bolt*, UC Berkeley<br>Chair, SMIAC Ground-Response Subcommittee |
| 11:00 - 11:30 | **Verification of Response Spectral Shapes and Anchor Points for Different Site Categories for Building Design Codes**<br>*W. Silva* and *G. Toro*, Pacific Engineering Analysis |
| 11:30 - 12:00 | **Recovery of Records from Code-Required Accelerographs after the Northridge Earthquake**<br>*A. Shakal*, C. Petersen, R. Darragh, M. Huang, SMIP, and R. Nigbor, K. Madura, Agbabian Associates |
| 12:00 - 12:20 | **Questions and Answers for Sessions I and II** |

| | |
|---|---|
| 12:20 - 1:50 | **Lunch**<br>Introduction *James Davis*, State Geologist, Division of Mines and Geology<br>Luncheon Speaker *Elin Miller*, Director, Department of Conservation |
| **SESSION III** | **Moderator:** *Vern Persson*, Division of Safety of Dams, DWR<br>Chair, SMIAC Lifelines Subcommittee |
| 1:50 - 2:20 | **Implications of the Strong-Motion Records from a Retrofitted Curved Bridge on Seismic Design and Performance**<br>*D. Liu*, A. Kartoum, S. Dhillon, X. Chen and R. Imbsen,<br>Imbsen and Associates |
| 2:20 - 2:50 | **Case Studies of 25 Buildings Subjected to Strong Shaking in the Northridge Earthquake**<br>*F. Turner*, Seismic Safety Commission staff (Invited) |
| 2:50 - 3:10 | Break |
| **SESSION IV** | **Moderator:** *Jeffrey Johnson*, Seismic Safety Commission<br>Chair, SMIAC |
| 3:10 - 3:40 | **Strong-Motion Records from Buildings Damaged in Earthquakes**<br>*M. Huang*, P. Malhotra and A. Shakal, SMIP |
| 3:40 - 4:10 | **Application of Recorded Motion to Post-Northridge Evaluation of Steel Frame Buildings**<br>*J. Kariotis*, Kariotis and Associates (Invited) |
| 4:10 - 4:30 | **Questions and Answers for Sessions III and IV** |
| 4:30 - 4:35 | **Closing Remarks** |

# ANALYSIS OF STRONG MOTION RECORDS FROM PARKING STRUCTURE DURING THE JANUARY 17TH NORTHRIDGE EARTHQUAKE

**S. Hilmy, S. Werner, A. Nisar and J. Masek**
**Dames & Moore**

## ABSTRACT

The parking structure studied in this investigation is the first parking structure from which significant strong-motion data has been obtained during January 17, 1994 Northridge earthquake. Although the structure did not suffer significant damage, the study of the recorded motions was conducted to evaluate the seismic response of parking structures during strong ground shaking and the adequacy of the current seismic design provisions for such structures.

An important element of this research project was the use of system identification of the recorded motions in the parking structure, in order to estimate normal modes of vibration excited in the structure during the Northridge Earthquake. These normal modes were then used to calibrate a detailed finite element model of the structure which, in turn, was used to carry out detailed seismic analyses of the structure. The analyses indicated that proper modeling of all the elements of the parking structure including soil flexibility led to reasonable prediction of the main dynamic response characteristics of the parking structure. From this, several design recommendations were proposed in this study to improve the current modeling techniques and the code design provisions of parking structures.

## INTRODUCTION

The extensive damage to parking structures during the Northridge earthquake, resulted from several unique characteristics of such structures (Ref. 1). For example, many of the damaged parking structures were constructed from precast concrete components which lacked adequate strength, ductility and redundancy. In addition, the architectural configuration of the sloped ramps in parking structures and the existence of the deep spandrels attached to the perimeter columns results in short effective lengths of the columns. The shear demands for these short columns increase significantly as their length decreases. The concrete ramps in parking structures form a connecting link between floors that is not typically modeled in the seismic design and analysis process. Sloped ramps with a large span-width ratio may experience large floor accelerations and may result in more flexible response and high seismic stresses in certain members. Another unique feature of the parking structure is the existence of long spans and open architecture, both to reduce construction cost and to increase parking space. Parking structures typically lack interior nonstructural elements and are subjected to effectively larger forces and deformations. Finally, separation joints in older parking structures were often insufficient to prevent pounding, which was observed in many instances.

The poor performance of many parking structures during the January 17th, 1994 Northridge Earthquake demonstrates the need for further evaluation of the current seismic analyses and design procedures for such structures. An important vehicle for improving these procedures and for understanding the seismic response characteristics of parking structures is the compilation and analysis of strong motion records from such structures using sound analysis procedures. Unfortunately, there had been no known evaluation of recorded motions in parking structures during the past earthquakes. In fact, the structure investigated in this project is the first parking structure in which earthquake motions had been recorded.

A set of 14 strong motion records was obtained by the Strong Motion Instrumentation Program (SMIP), of the California Department of Mines and Geology, at a 6-story parking structure in Los Angeles during the Northridge earthquake. The recorded horizontal acceleration time histories had peak values of 0.29 g at the base of the structure and 0.84 g near the center of the roof diaphragm. One of the sensors, which was attached to the top of the roof parapet indicated acceleration as high as 1.21 g. However, the structure did not suffer from significant structural damage during the earthquake. The main objective of the study summarized herein was to study these recorded motions using the MODE-ID method and to investigate the adequacy and accuracy of the finite element modeling techniques currently used in the analysis of these structures. Based of the assessment of the behavior of the parking structure and study of the recorded motion, demand/capacity ratios of the structural members have been compared to allowable code values.

## DESCRIPTION OF THE PARKING STRUCTURE UNDER INVESTIGATION

The overall structural configuration and the location of accelerometers are shown in Fig. 1. The parking structure is located near downtown Los Angeles. It is a six story reinforced concrete structure that is rectangular in plan, and has plan dimensions of approximately 307 feet in the east-west direction and 260 feet in the north-south direction. The structure has seven levels of parking with a total usable area of approximately 550,000 square feet. The typical floor height of the structure is 10 feet. The building was constructed in two phases. The first three stories were constructed in 1977 "Phase I" construction project. These stories contain a 5-inch cast-in-place post-tensioned concrete slab spans between adjacent precast concrete beams, which are spaced at 18 ft. o.c. The lateral load resisting system consists of cast-in-place shear walls. Each wall is 32.5 feet wide, and 14" thick. There are two interior walls (72' long and 16" thick) along the east-west direction. The soil at the site consists of alluvium soil on a deep layer of firm sand. All columns are supported on drilled bell caissons.

In 1979, a "Phase II" project resulted in construction of three additional levels above the original parking structure, with a similar architectural layout to the existing structure. However, during this phase, cast-in-place concrete was used for all additional columns and walls. The interface between Phase I and II construction is provided by roughening the existing concrete surface and by providing full strength butt welding of existing and new reinforcements. The exterior spandrels at the south and north sides are separated from the columns. However, the spandrels at the east and west sides are connected to the shear walls with continuous steel dowels to provide flexural continuity at the beam-wall joints.

**Photo 1. View looking northeast**

Photo #1 shows a view of the structure. One important characteristics of the parking structure is that the interior prestressed beams are seated on neoprene bearing pads at the columns corbels, with no positive ties between the beams and columns. The slabs are connected to the columns with $^3/_4$ inch diameter coil inserts that are embedded 6 inches into the columns and are connected to $^3/_4$ inch diameter threaded rods that are embedded three feet into the slab; the typical connection between the precast concrete columns and cast-in-place walls is provided through steel dowels and shear keys.

## STRONG MOTION DATA

The parking structure is located approximately 31 km from the epicenter of the Northridge earthquake which occurred at 4:30 a.m. on the morning of January 17, 1994, and had a moment magnitude (MW) of 6.7. The California Division of Mines and Geology deployed a total of 14 strong motion accelerometers within the structure whose locations along the first floor, fourth floor, and roof are shown in Fig. 1. This instrumentation system has been designed to measure (a) horizontal translations (in two orthogonal directions) and torsional rotations of each instrumented floor, (b) vertical translations of the first floor, together with rocking rotations of the floor about the north-south axis; © in-plane diaphragm deformations in the north-south direction, and (d) out-of-plane bending deformations of the parapet on the north side of the roof. In addition, a single vertical accelerometer is located on the roof.

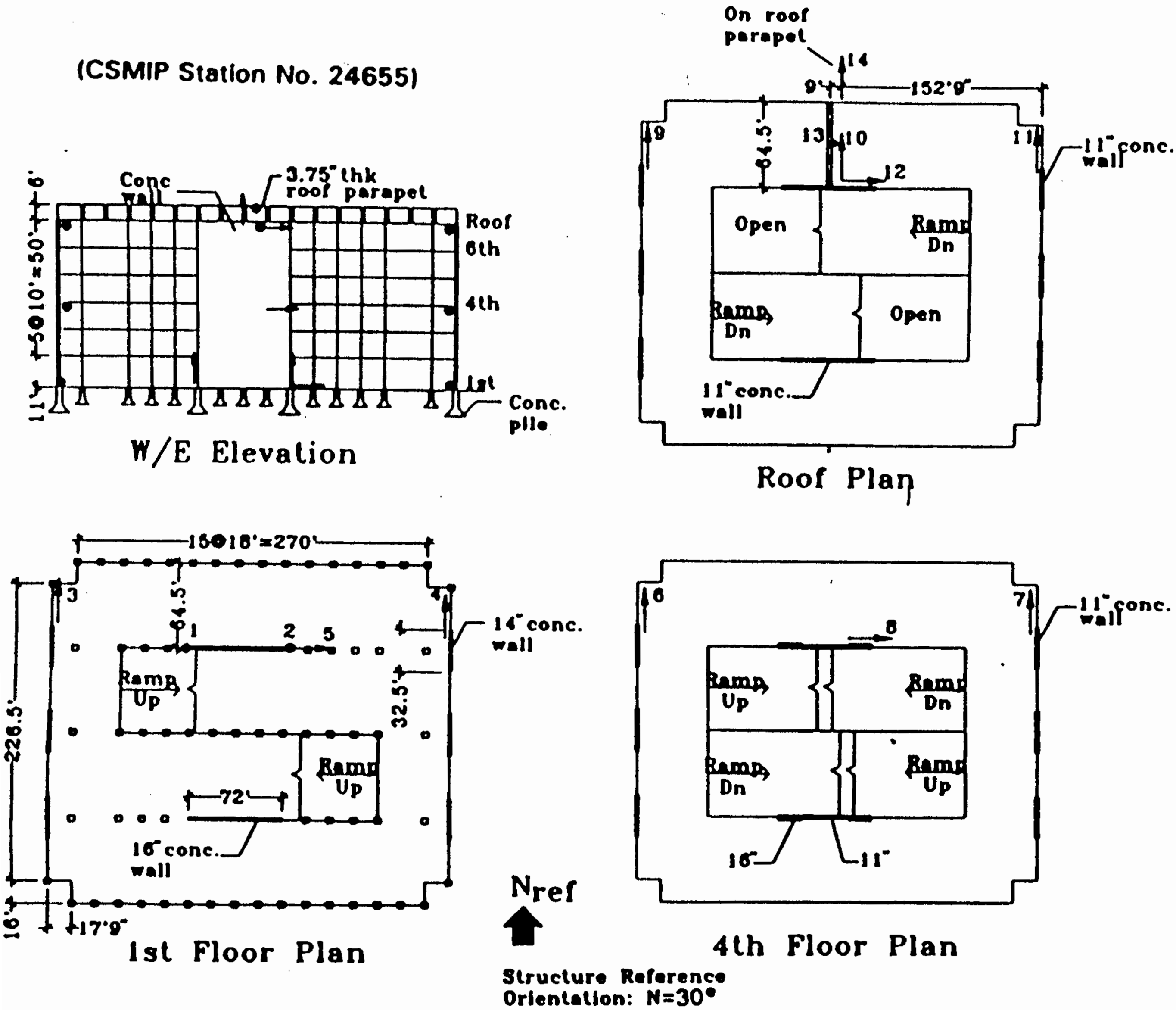



Figure 1. Structural Configuration and the Location of Accelerometers

The motions recorded at the base of structure were moderately strong, with peak horizontal accelerations of 0.29 g and 0.15 g in the north-south and east-west directions respectively and peak accelerations of the two vertical accelerograms of 0.22 g and 0.11 g. These motions were amplified substantially over the height of the structure, attaining peak horizontal roof accelerations of 0.55 g and 0.31 g in the north south and east-west directions. The north-south accelerations at the mid-length of the roof was amplified still further with a peak acceleration of 0.84 g. In addition very strong horizontal motions were recorded on the north parapet (with peak acceleration of 1.21 g), and strong vertical motions of a roof girder were also recorded (peak acceleration = 0.52 g). The duration of the strong shaking segment of the recorded motions was in the order of 12-13 sec. A comparison between the time-history acceleration records at different stations is shown in Fig. 2. The generated response spectra curves of the recorded motion at the base of the structures, along the north-south and east-west directions, are shown in Fig. 3.

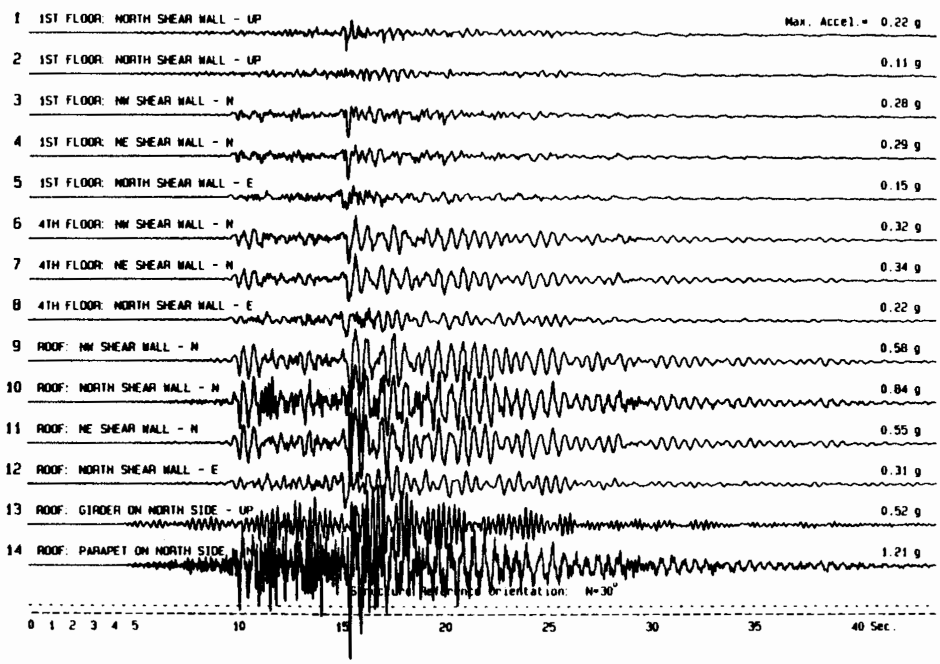



**Figure 2. A Comparison Between the Time-History Acceleration Records at Different Stations**

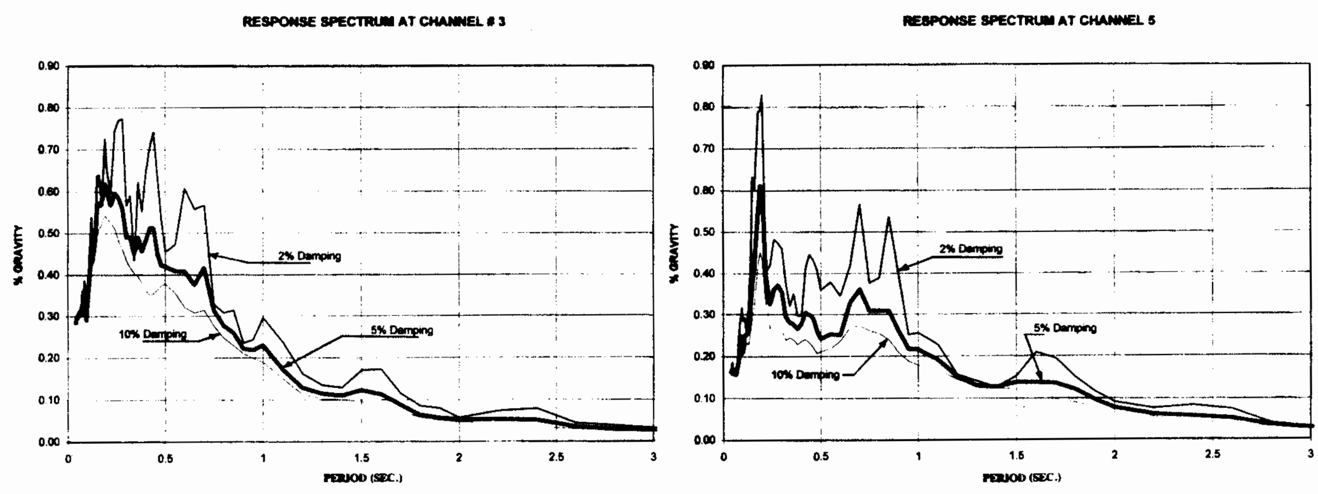



**Figure 3. Generated Response Spectra Curves of Recorded Motions**

## SYSTEM IDENTIFICATION OF RECORDED MOTIONS

In order to estimate the modal parameters for the modes of vibration of the structure during the Northridge earthquake the MODE-ID system identification procedure was applied to the strong motion records. This procedure involved three main steps: (1) seismic response analysis was carried out by examining the accelerogram records and by computing transfer functions. These transfer functions were computed as ratios of the Fourier Amplitude Spectra (FAS) of the recorded motions at the upper floors to the FAS of the motions of the ground floor; (2) model parameters that were identified by MODE-ID were the natural period, mode shape, damping ratio, and participation factors for each significant mode of vibration; and (3) assessment of the adequacy of the identified model, by comparing the computed model motions to the recorded motions; assessment of the relative contributions of the structure's pseudostatic response and its response in each mode of vibration to its total response; and interpretation of the relative translational, torsional, and rocking contributions to the response of the parking structure in each mode.

The structural response to the input motions is assumed to consist of two components; the pseudo-static component and the dynamic component. The pseudostatic component represents the "quasi-static" contributions of the individual support motions to the building's total response (neglecting inertial and damping effects). It can be visualized as a time-dependent "reference" position of the structure whose deformed shape at each instant of time depends on the instantaneous position of the structure's supports. This pseudostatic response is represented as the product of a pseudostatic matrix and the vector of input motions. The dynamic response component represents the contributions of the structure's modal vibrations about its pseudostatic reference position. The model parameters that are used to compute the dynamic component are the natural period, damping ratio, input participation factors, and mode shape amplitude for each significant mode excited by the earthquake. The pseudostatic and normal mode parameters are estimated by a least-squares output-error method, in which MODE-ID uses an optimization algorithm to compute the "best" matching of the measured response (Ref. 2). Within a Bayesian probability framework, the estimated parameters can be viewed as most probable values based on the given data (Ref. 2).

For the parking structure under investigation, it was not necessary to identify the pseudostatic matrix using MODE-ID; rather, the matrix was calculated directly based on the assumption that the base of the structure was rigid. The pseudostatic matrix and then one mode at a time were successively incorporated into the model, and the modal parameters identified from each MODE-ID run were used as input to the next run with one additional mode included. This process led to the identification of the modal parameters for each significant mode, such that the resulting building model (which also includes the pseudostatic matrix) minimized a measure-of-fit parameter J(θ). This parameter is defined as the ratio of the sum of the output errors to the sum of the squares of the measured accelerations, i.e.,

$$J(\theta) = \frac{1}{V} \sum_{i=1}^{NR} \sum_{n=0}^{NT} [a_i(n\Delta t) - \ddot{y}_i(n\Delta t, \theta)]^2 \tag{1}$$

where

$$V = \sum_{i=1}^{NR} \sum_{n=0}^{NT} [a_i^2(n\Delta t)] \tag{2}$$

and

| | | |
|---|---|---|
| $a_i$, $\ddot{y}_i$ | = | Measured acceleration and computed model acceleration for the ith output degree of freedom (where I= 1,2,- - - - - NR, which is the total number of output channels). |
| θ | = | Pseudostatic matrix elements and identified modal parameters. |
| Δt | = | Time step at which the recorded motions in the structure have been digitized (= 0.01 sec.). |
| n | = | Time step number ranging from 0 to NT, which corresponds to a total duration of NT x Δt sec. |
| V | = | Sum of the squares of the recorded accelerations. |

## Time-Invariant and Time-Varying Models

Both time-invariant and time-varying models of the parking structure were used to show how the modal parameters vary over time (as the intensity of the ground shaking varies), and to assess the degree to which nonlinear behavior may have played a roll in the structure's seismic response. Four time segments over which the strength of the shaking appeared to be clearly different were considered: (a) 10-15 sec., which corresponds to the initial buildup of the strength of the shaking; (b) 15-30 sec. which corresponds to the duration of the strongest shaking of the structure during the earthquake; © 30-40 sec., during which the shaking of the structure decayed to very low levels; and (d) 40-60 sec., when the structure was undergoing essentially free vibration under very low intensities of shaking. In addition to the time-invariant models, time-varying models were identified using overlapping sliding time windows with a duration of 5 sec. and an overlap of 2.5 sec.; i.e.10-15 sec., 12.5-17.5 sec., 15-20 sec., 17.5-22.5 sec., etc.

For both the time-invariant and time-varying models, the input motions to MODE-ID consisted of the horizontal motions recorded at the base of the structure, as well as the average (i.e., translational component only) of the vertical base motions. The output motions were considered to be the horizontal motions measured at all of the instrument locations above the base of the structure, as well as both sets of vertical motion records measured at the base.

The significant modes of vibration estimated for the parking structure include the effects of horizontal translation in the north-south and east-west directions, torsional rotation (about a vertical axis), and rocking of the structure about its north-south axis. The effects of rocking on the structural response in the east-west directions were estimated by computing an equivalent rigid body translational component of the mode shape amplitude at each instrumented floor due to rocking, i.e.

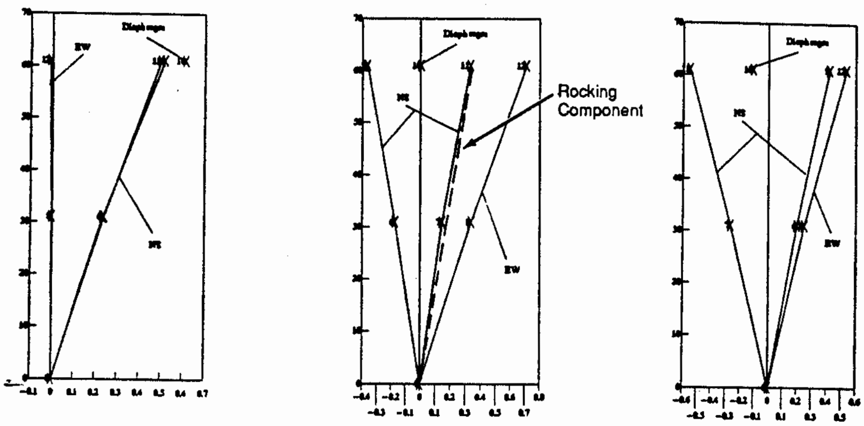
$$\phi_{R,i,n} = (\phi_{v2,n} - \phi_{v5,n}) \times H_i / D_{2-5} \tag{3}$$

where, for the $n^{th}$ mode, $\phi_{R,i,n}$ is the mode-shape's east-west component of translation at the $i^{th}$ floor due to rocking of the base, $\phi_{v2,n}$ and $\phi_{v5,n}$ are the mode shape's vertical component of translation at the locations of Channels 2 and 5 along the base of the structure, $D_{2-5}$ is the distance between Channels 2 and 5, and $H_I$ is the height of the $i^{th}$ floor above the base.

## Model Assessments

An important element of the MODE-ID process is an evaluation of how well the various models of the parking structure that were identified from each set of recorded earthquake motions represent the structure's seismic response during the Northridge Earthquake. This assessment was based on (a) the use of past experience (Ref. 4, 5) to evaluate whether the minimum value of $J(\theta)$ obtained for each model was sufficiently small to represent a good overall fit between the measured response of the structure and the computed model response; and (b) visual comparison of recorded and computed model acceleration time histories and their Fourier amplitude spectra, at selected locations in the structure. As part of this model assessment, we also evaluated the relative contribution of the pseudostatic response and each identified mode of vibration to the structure's seismic response. To accomplish this, we examined how much $J(\theta)$ decreased as the pseudostatic matrix and each identified mode were successively incorporated into the model.

Once a theoretical pseudostatic matrix for parking structure was developed, this matrix was incorporated into all time-invariant and time-varying models that were identified for the structure. For each time segment, a total of six modes of vibration were identified (Fig. 4). The first identified mode corresponds to the first translational mode of vibration in the north-south direction. At each instrumented floor, the north-south translational components of the mode shape amplitudes along the east and west faces of the parking structure are comparable to each other, increase nearly linearly with increasing height above the ground floor, and are much larger that the east-west translational components (which are essentially negligible). In-plane deformations of the roof diaphragm are relatively large for the 10-15 sec. time segment, and are somewhat smaller for the other time segments.




a) Mode 1: First North-South (T=0.53 sec, $\xi$ = 4.8 %)

b) Mode 2: First East-West (T=0.42 sec, $\xi$ = 4.6 %)

c) Mode 3: Torsional (T=0.38 sec, $\xi$ = 4.2 %)

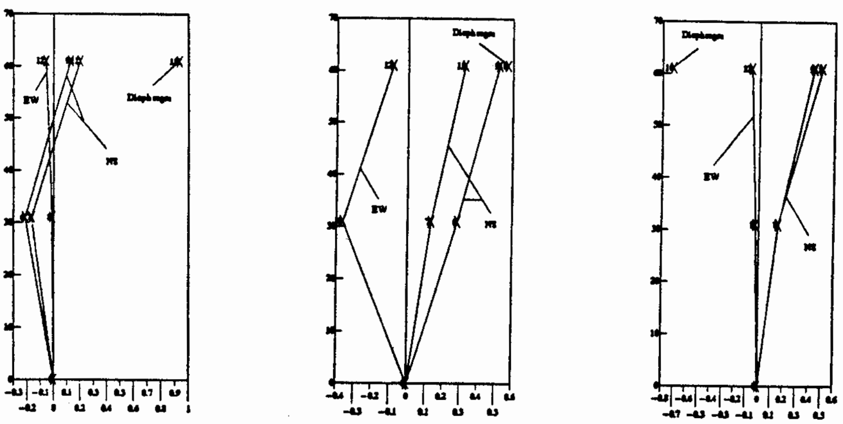
| Output \ Input | 3 | 4 | 5 | $\nabla$ |
|---|---|---|---|---|
| 6 | 1 | 0 | 0 | 0 |
| 9 | 1 | 0 | 0 | 0 |
| 7 | 0 | 1 | 0 | 0 |
| 11 | 0 | 1 | 0 | 0 |
| 10 | 0.5 | 0.5 | 0 | 0 |
| 8 | 0 | 0 | 1 | 0 |
| 12 | 0 | 0 | 1 | 0 |
| 1 | 0 | 0 | 0 | 1 |
| 2 | 0 | 0 | 0 | 1 |

Input Channels: 3, 4, 5, $\nabla$

where $\nabla$ = (Ch1 + Ch2)/2

Output Channels: 6, 9, 7, 11, 10, 8, 12, 1, 2

**Pseudostatic Matrix**



d) Mode 4: Second North-South (T=0.18 sec, $\xi$ = 5.0 %)

e) Mode 5: Higher Mode (T=0.15 sec, $\xi$ = 4.4 %)

f) Mode 6: Higher Mode (T=0.12 sec, $\xi$ = 1.7 %)

**Figure 4. Identified Mode Shapes for Time Segment = 15-30 Sec., and the used Pseudostatic Matrix for the Rocking Base Model of the Parking Structure**

Mode #2 is dominated by the east-west translational components of motion, which increase nearly linearly with increasing height above the ground floor, and contains some torsional rotations of the diaphragms, and only small translations in the north-south direction. The in-plane diaphragm deformation in this mode is small. Mode #3 features significant torsional rotations accompanied by only very small north-south translations. Mode #4 contains north-south translational components at the fourth floor that are of comparable magnitude but opposite sign to those at the roof. Modes #5, and #6 are higher modes.

## Comparisons between Computed Model Motions and Recorded Motions

For each time segment, the time-invariant models comprised of the above six normal modes plus the pseudostatic matrix shown in Fig. 4 led to an excellent fit between the computed model motions and the recorded motions. This excellent fit is evidenced by: (a) the very low values of the measure-of-fit parameter, $J(\theta)$, which range from about 0.021-0.035 (where, from past experience, values of $J(\theta)$ of about 0.15 or less generally represent an excellent fit); and (b) very close visual comparisons of the time histories and Fourier amplitude spectra of the computed model motions and the recorded motions, as typified by the comparisons shown in Fig. 5.

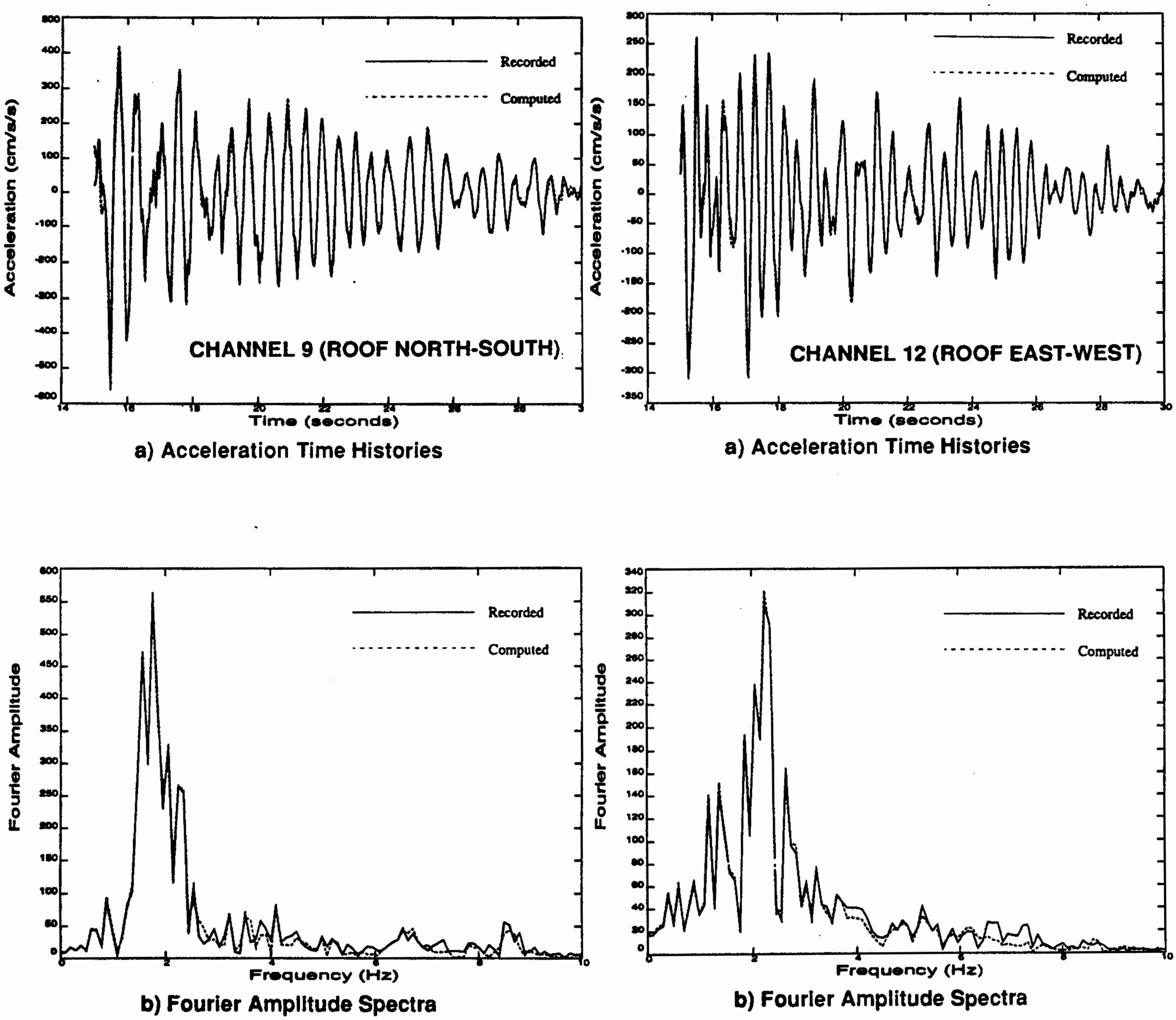
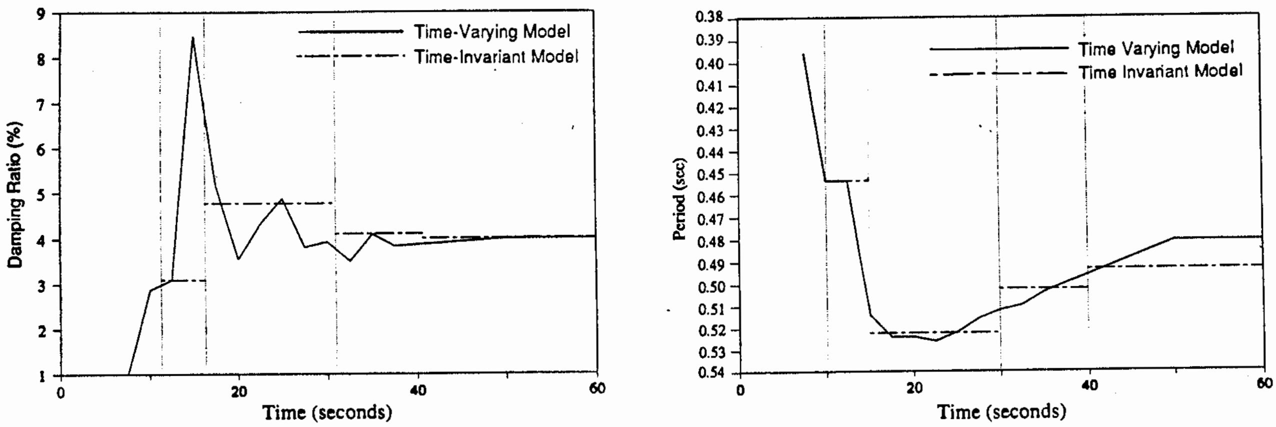


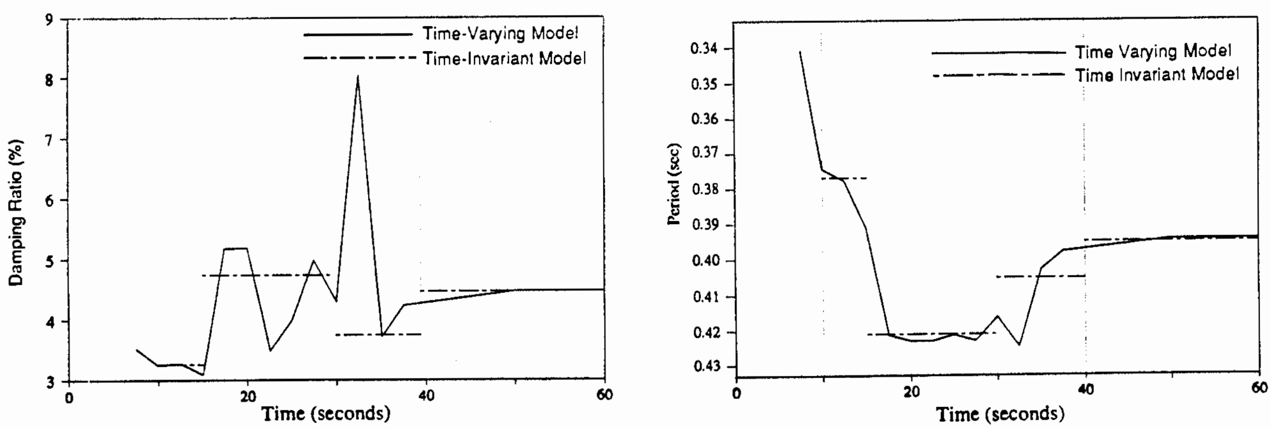

**Figure 5. Comparison between Recorded Motion of Parking Structure and Computed Model Motions for the 15-30 Sec. Time Segment.**

## Variations in Modal Parameters between Different Time Segments

Fig. 6 shows how the period of vibration and the damping ratio for mode 1 and 2 vary over time. at different time windows. The rather small differences between these natural period and damping ratio values among all of the various time windows suggests that the parking structure did not undergo significant nonlinear response during the Northridge Earthquake.




a) Mode 1: First North-South




b) Mode 2: First East-West

**Figure 6. Variation of Natural Periods and Damping Ratios of the Parking Structure Over Time**

The variations in natural period and damping ratio between the various time windows exhibit similar trends for the two predominant modes. For both modes, the natural periods are consistently longer for the 15-30 sec. time window than for the 10-15 sec. window, and the damping ratios are consistently smaller. As the time proceeds from the 15-30 sec. window of strongest shaking to the 30-40 sec. window of decreased shaking, the natural periods for the two modes are shortened somewhat and the damping ratios are decreased. However, for both modes, the periods are still longer than those for the initial 10-15 sec. time segment, and the damping ratios are still slightly larger. A few significant peaks and irregularities are observed for the damping estimates from the time varying models which may possibly be due to the short time window duration used for these modes.

## FINITE ELEMENTS COMPUTER MODELS

A detailed finite elements computer model was developed to study the dynamic behavior of the parking structure under consideration. Since the Northridge earthquake did not result in noticeable damage to the structure, only a linear model was considered using the SAP90 general finite element computer program. The model was calibrated against the structure's recorded motions, through comparison of the computed model motions and the recorded motions and comparison of the modal parameters of the finite element models against those identified by MODE-ID from the strong motion records. Once the finite element models was calibrated and checked in this way, it was used to carry out detailed analysis of the structure's dynamic response to the recorded base motions.

Fig. 7 shows the three-dimensional (3-D) plot of the finite element model that was developed for the parking structure. The general characteristics of the model are as follows: the model included 2309 nodes resulting in 13,399 equations of motion; the shear walls and ramps were modeled with shell elements, These walls were supported on soil springs with a coefficient of subgrade reaction of 300 1b/in/in, in order to incorporate soil-structure interaction. Coupling beams between the east and west walls were modeled using cracked and uncracked section properties and columns were attached to the sloped diaphragm. Hinged conditions were used at the base of the columns; the first computed 25 modes of vibration were considered producing over 99% mass participation.

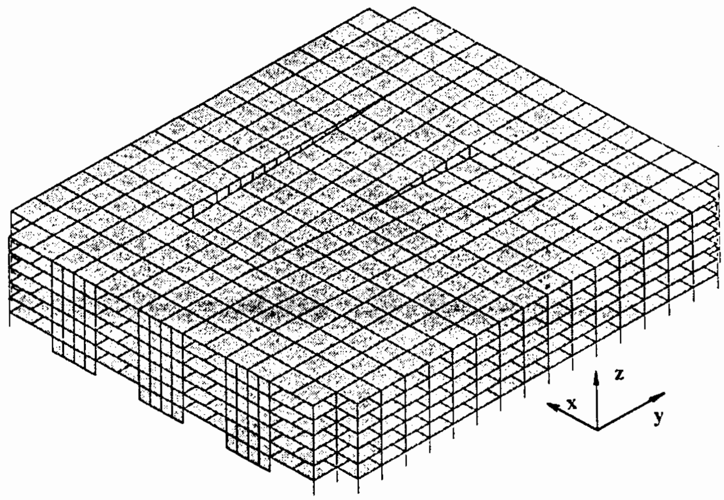



**Figure 7. 3-D Finite Element Model of Parking Structure**

The computer models were subjected to horizontal input motions in the north-south and east-west directions in the form of 5-percent damped response spectra. Along the north-south direction, these input motions corresponded to the average of the spectra of the recorded base motions of Channels # 3 and Channel # 4. Along the east-west direction, a response spectrum curve recorded at Channel # 5 was considered (see Fig. 3). Due to the absence of free-field vertical acceleration records, no ground shaking was considered in the vertical direction. In addition to the response spectrum analyses, transient analyses were performed using the recorded time-history motions in both directions. The standard mode superposition method and the Ritz vectors algorithm are used in SAP90 program to solve the dynamic equilibrium equations of motions for the complete structure model.

### Normal Modes of Vibration

Fig. 8 shows the resulting first and third fundamental modes of the structure. Mode #1 is a lateral mode along the north-south direction (75% mass participation); the second fundamental mode is a lateral mode along the east-west direction (77% mass participation); and the third mode is a pure torsional mode. 2-D views of the significant modes along the north-south and east-west directions are shown in Fig. 9. Table 1 compares between the period of the vibrations of the first six fundamental modes obtained from Mode-ID, and computer analysis using cracked and uncracked sections. It is clear from this table that the cracked diaphragm model provides better correlation with the results obtained from Mode-ID method, particularly for the modes sensitive to in-plane diaphragm motion (i.e. Mode 1 and Mode 4).

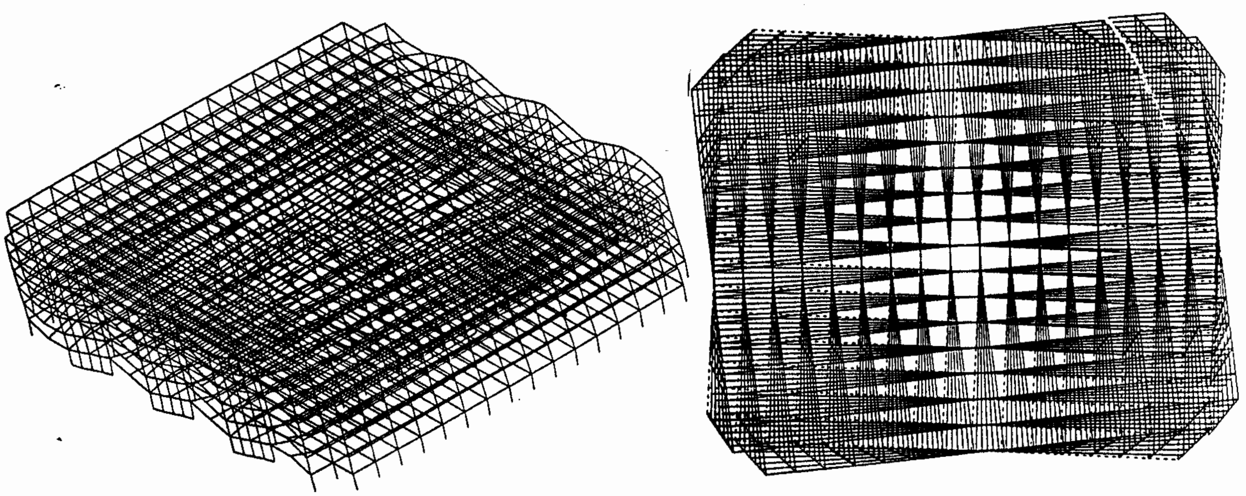


**Figure 9. First Fundamental Modes acting along north-south direction, and the Torsional Mode**

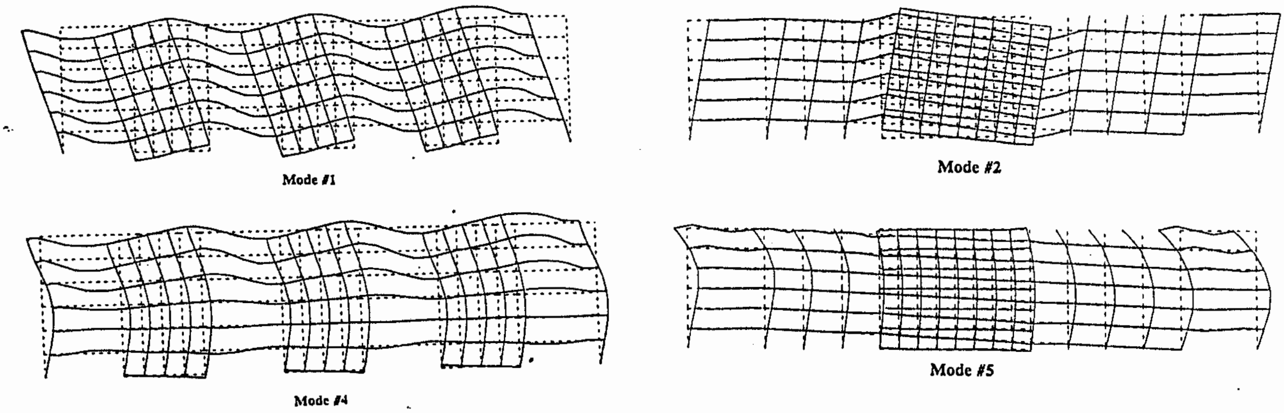



**Figure 9. Two-Dimensional View of the Fundamental Modes of the Parking Structure.**

**Table 1**
**Fundamental Periods (Sec.)**

| Case | Fundamental Periods (Sec.) | | | | | |
|---|---|---|---|---|---|---|
| | Mode 1 | Mode 2 | Mode 3 | Mode 4 | Mode 5 | Mode 6 |
| Mode ID (Time Window 15-30 Secs.) | 0.530 | 0.420 | 0.380 | 0.180 | 0.150 | 0.120 |
| Finite Element (F. E. Model) | 0.517 | 0.430 | 0.389 | 0.157 | 0.128 | 0.120 |
| F. E. Model with Cracked Diaphragm | 0.528 | 0.447 | 0.393 | 0.183 | 0.150 | 0.137 |

A comparison between the maximum recorded acceleration, at the locations of Channel #6 through Channel #12, is shown in Table 2. In general good results were obtained from the finite elements models.

**Table 2**
**Comparison Between Maximum Recorded Acceleration and Computed Acceleration**

| Channel # | Ch. #6 | Ch. #7 | Ch. #8 | Ch. #9 | Ch. #10 | Ch. #11 | Ch. #12 |
|---|---|---|---|---|---|---|---|
| Max. Recorded Acceleration (g) | 0.32 | 0.34 | 0.22 | 0.58 | 0.84 | 0.55 | 0.31 |
| Max. Computed Acceleration(g) (uncracked) | 0.334 | 0.34 | 0.196 | 0.583 | 0.79 | 0.598 | 0.389 |
| Max. Computed Acceleration(g) (Cracked) | 0.327 | 0.34 | 0.20 | 0.60 | 0.90 | 0.61 | 0.368 |

The computed time history records (signatures) are plotted vs. the recorded time-history results (Fig. 10). The comparison is given for the 15-30 sec. time window of the strongest shaking. Generally, a reasonable fit between the computed and the recorded spectra curves was obtained. An excellent agreement is obtained for the building's frequencies, but there is indication of overshooting of the amplitudes at some cycles. This suggests that larger damping ratio than those obtained from the MODE-ID method would give better fit. It is also indicated that the computed response along the east-west direction is more accurate than the response obtained along the north-south direction.

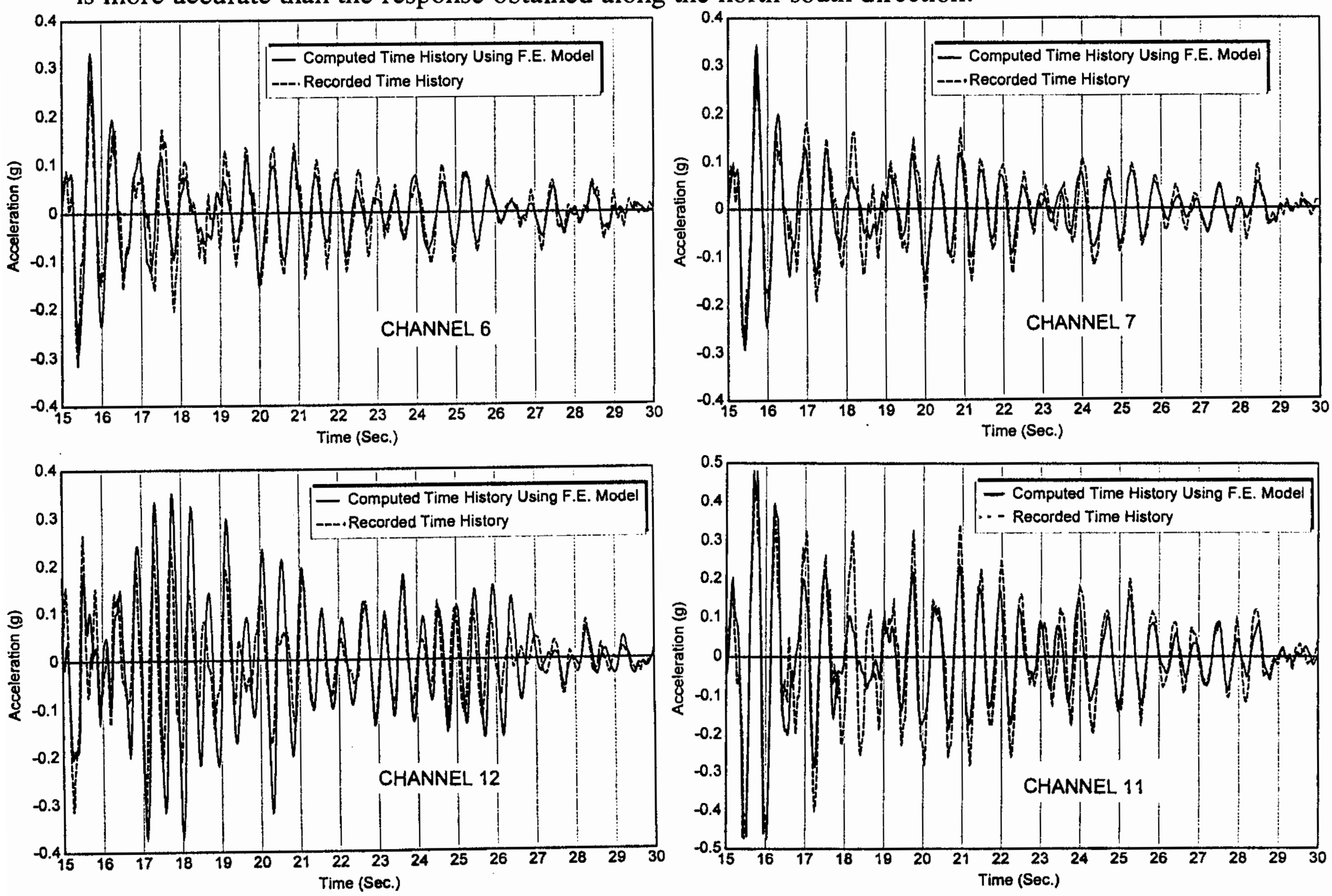



**Figure 10. Comparison Between the Recorded Time History and the Computed Time History Using F.E. Model for Selected Channels.**

In order to assess the structural behavior and to compare between the predicted and recorded response and the code provisions, four computer runs were performed as follows: Run #1: The finite element model was subjected to the recorded ground response curves corresponding to 5% damping. Uncracked section properties were used in this run; Run #2: This run is similar to Run #1, except that cracked sections (60% E) were used for the diaphragms and the coupling beams of the shear walls; Run #3: The model was also subjected to the UBC94 response spectrum curve for Zone 4; Run #4: The computer results were also compared with the results obtained from the UBC94 Code equivalent static lateral loads. In this regard, $R_w$ of 8 was considered.

Table 6 shows the results of the seismic base shear obtained for the 4 cases defined above. The following observation were made: The code seismic shears are larger along the east-west direction than the north-south direction (13.2% g compared to 11.7% g). However, the finite element results using UBC code spectrum indicate different distribution (99% g along the North-South direction, and 77% along the East-West direction). It is shown that the base shear obtained from the code response spectrum curve, and the recorded ground motion are 8.46 and 5.88 times the code base shear, respectively. These factors can be compared to the reduction factor $R_w$ = 8, which is used in UBC as a measure of the ductility of the structural system of this parking structure. As shown in Table 6 the results using the recorded ground motion as input are much smaller than the results using the UBC94 response spectrum.

**Table 6**
**Seismic Base Shear in the Parking Structure**

| Case | N-S Direction E-W Direction | | E-W Direction | |
|---|---|---|---|---|
| | V (Kips) | % G | V (Kips) | % G |
| F. E. Analysis Using Recorded Motions (uncracked Diaphragm) | 18,567 | 43% | 9,940 | 23% |
| F. E. Analysis Using Recorded Motions (cracked Diaphragm) | 19,187 | 44% | 10.356 | 24% |
| Finite Element Analysis UBC Spectra at Zone 4 | 44,084 | 99% | 33,100 | 77% |
| Code Equivalent Static (UBC 94) with Rw = 8 | 5,052 | 11.7% | 5,705 | 13.2% |

Table 7 provides a comparison between the seismic lateral displacement at different levels of the structure. The maximum diaphragm displacements at the middle of the span of the diaphragm at the sixth level, along the north-south direction, is shown in Table 8. The following observations were made: (1) The inter-story drift based on the finite element results is approximately 0.2%, and did not result in noticeable damage to the nonstructural elements; (2) Although the ground motion did not result in noticeable damage, the maximum deflection obtained from Run #1 is approximately 2.85 times the code deflection; (3) The finite element model indicates that the north-south deflection at the mid-span of the diaphragm is approximately 16% higher than the deflection at the end shear walls.

Table 9 provides the computed seismic shear and moment demands at typical north-south and east-west reinforced concrete shear walls. This table shows that the recorded motions produced shear and flexural seismic demands that less than the capacity of the walls. This explains the absence of hair line shear cracks in these walls. In addition, there was no indication of any overstressing at the location of the construction joints at the third level (where Phase II construction started).

**Table 7**
**Seismic Lateral Displacement in the Parking Structures**

| Case | F.E. with recorded motion | | F.E.with cracked diaphragm | | F.E.with UBC Spectrum | | Code Static Rw=8 | |
|---|---|---|---|---|---|---|---|---|
| Level | N-S wall | E-W wall | N-S wall | E-W wall | N-S wall | E-W wall | N-S wall | E-W wall |
| Roof | 1.43" | 0.74" | 1.45" | 0.84" | 3.40" | 2.63" | 0.50" | 0.39" |
| 6th | 1.21" | 0.62" | 1.23" | 0.70" | 2.90" | 2.23" | 0.43" | 0.31" |
| 5th | 0.97" | 0.50" | 0.98" | 0.56" | 2.31" | 1.81" | 0.34" | 0.24" |
| 4th | 0.71" | 0.37" | 0.72" | 0.42 | 1.71" | 1.36" | 0.25" | 0.18" |
| 3rd | 0.46" | 0.24" | 0.46" | 0.28" | 1.10" | 0.90" | 0.17" | 0.11" |
| 2nd | 0.21" | 0.12" | 0.21" | 0.14" | 0.52" | 0.45" | 0.08" | 0.04" |

**Table 8**
**Maximum Diaphragm Displacements (N-S)**

| Case | F.E. with recorded motion | F.E.with cracked model | F.E. with UBC Spectra | Code Static Rw=8 |
|---|---|---|---|---|
| Roof | 1.581" | 1.663" | 3.78" | 0.56" |

**Table 9**
**Maximum Seismic Shear and Moment Demands on Typical N-S and E-W Walls**

| Wall | N-S wall | | | | E-W wall | | | |
|---|---|---|---|---|---|---|---|---|
| Case | Seismic Shear | | Seismic Moment | | Seismic Shear | | Seismic Moment | |
| | V (kips) | D/C | M (k.ft) | D/C | V (kips) | D/C | M (k.ft) | D/C |
| F. E. Analysis Using recorded Motion | 2244 | 0.96 * | 59,160 | 0.69 | 5110 | 0.73 | 208,334 | 0.56 |
| F. E. Analysis Using UBC Spectra | 5328 | 2.28 * | 142,604 | 1.65 | 16586 | 2.36 | 676,211 | 1.80 |
| Code Equivalent Static Rw = 8 | 842 | 0.84 ** | 36,585 | 0.595 | 2852 | 0.95 | 123,940 | 0.33 |

* indicates using $\phi = 1$, ** indicates using $\phi = 0.6$ and factored load.

Table 10 indicates that the shear walls experienced noticeable uplift. Maximum uplift forces of 2225 kips and 1763 kips in the east-west walls and north-south walls, exceeded the estimated 1210 kips uplift resistance forces. It is noted that the code uplift forces are less than the uplift resistance forces, indicating that code stress checks will not predict this uplift behavior.

**Table 10**
**Maximum Seismic Uplift Forces and Displacements**

| **Wall** | **N-S middle wall** | | **E-W wall** | |
|---|---|---|---|---|
| **Case** | **Uplift force** | **Uplift disp.** | **Uplift force** | **Uplift disp.** |
| Finite Element Analysis Using recorded Motion | 1763 kips | 0.29" | 2225 kips | 0.31" |
| Finite Element Analysis Using UBC Spectra | 4230 kips | 0.68" | 7444 kips | 1.05" |
| Code Equivalent Static Rw = 8 | 627 kips | 0.10" | 1143 kips | 0.16" |

Note: Uplift forces in this table do not include reduction due to gravity loads

A stress check was performed to assess the seismic behavior of the short columns created due to the sloped ramps configuration. The result of this investigation for some critical short columns, indicated that the column's seismic shear demand forces due to the recorded input motions did not exceed their ultimate strength capacity, i.e., the demand/capacity ratio less than 1.0. However, higher levels of seismic shaking, due to the UBC response spectrum, will result in overstressing of these columns with demand/capacity ratio of approximately 2.5. This may be critical and could result in shear failure of these nonductile columns (Ref. 6). Typical column details, show limited tie reinforcements of #3 @ 12" o.c. New code provisions for gravity columns require minimum ties of #4 @ 6" o.c.

Another stress check was performed to examine the upper coupling beams connecting the N-S shear walls. It is shown that these beams were overstressed using the response spectrum analysis using the recorded response spectrum curves, with shear D/C of 2.84 and the flexural D/C of 1.94. The proposed ATC-33 (Ref. 7) recommends the use of limiting D/C ratios for shear and flexure as 1.0 and 2.0, respectively, for immediate occupancy performance. Therefore, higher D/C ratios may be justifiable in this case, based on the adequate performance of these long coupling beams during the Northridge earthquake .

One important assessment for this particular parking structure, is to study the integrity of the connections between the precast concrete columns and the floor slab. Table 11 provides the results of the stress check of some of the critical connections. D/C ratios up to 2.18 were obtained using the finite element model with recorded ground motion. However, no indication of overstressing was found during the site review. It is also indicated in Table 11 that D/C ratios up to 5.25 can be obtained at some of these connections, when using the Code response spectrum. It is highly questionable that these connections will sustain such large demands without experiencing excessive damage. Failure of these connections may lead to the separation of the columns from the slab, which may result in columns' instability (Ref. 8)

**Table 11**
**Maximum Pull Forces At Selected Column-Slab Connections**

| Forces | Conn. # 1 | | Conn. # 2 | | Conn. # 3 | | Conn. # 4 | |
|---|---|---|---|---|---|---|---|---|
| Case | P (kips) | D/C | P (kips) | D/C | P (kips) | D/C | P (kips) | D/C |
| F. E. using Recorded Motion. | 116 | 2.18 | 91 | 1.72 | 76 | 1.43 | 73 | 1.37 |
| F. E. Analysis Using UBC Spectra | 278 | 5.25 | 219 | 4.13 | 227 | 4.28 | 217 | 4.09 |
| Code Equivalent Static Rw = 8 | 35 | 1.03 | 31 | 0.91 | 40 | 1.18 | 38 | 1.12 |

Connection # 1: For Column at Grid (7) and ( C) @ 2nd Level South Ramp.
Connection # 2: For Column at Grid (8) and ( C) @ 2nd Level South Ramp.
Connection # 3: For Column at Grid (6) and ( B) @ 5th Level., Connection # 4: For Column at Grid (6) and ( B) @ 4th Level .

## RESPONSE OF VERTICAL VIBRATION OF THE ROOF GIRDER

In order to measure the vertical acceleration at the top level of the parking structure, one of the sensors was installed at the middle of the 65-foot long upper girder at the roof (Channel #13). A two-dimensional computer model was prepared to model the vertical response of the girders. Fig. 11 shows the first fundamental mode shape (estimated as 0.29 seconds). The following observations were made: (1) For large span girders (65 ft long), effective floor vertical acceleration in the middle of the girder can be significant (up to 3.5 times the peak ground vertical acceleration). This can lead to shear overstressing at the end of the beams (Ref. 7); (2) The acceleration of the nonstructural elements that may be attached at the middle of the long span girder (e.g. piping, etc) with fundamental periods between 0.20 to 0.40 seconds, can be significant; and (3) The current building codes do not provide simplified formulas to consider the effect of the vertical acceleration on the floor girders or beams. It appears that guidelines to consider such an effect are required. One design approach would be to increase or decrease the gravity loads acting on the beam with 2.5 times the peak ground vertical acceleration (in %g) times the gravity loads.

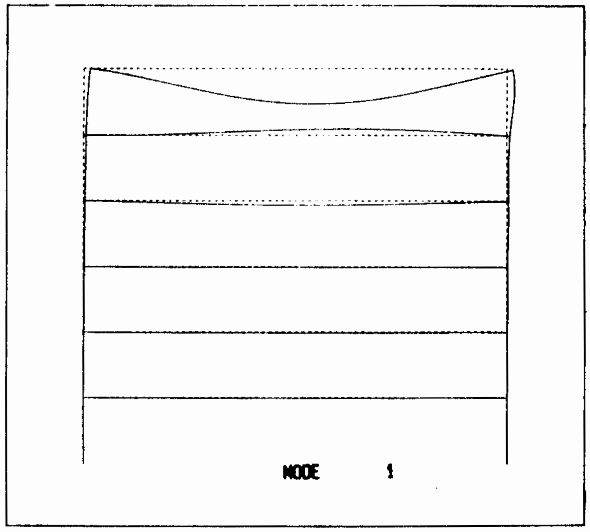



**Figure 11. 1st Fundamental Mode Shape of Roof Girder**

## RESPONSE OF THE ROOF PARAPET

In order to measure the out-of-plane acceleration at the roof parapet of the parking structure an accelerometer (channel # 14) had been installed at the top of the north roof parapet, and recorded very strong shaking (with a peak acceleration of 1.21 g). A three-dimensional computer model was developed to model the interaction between the parapet and the parking structure. Ten line elements were used to model the parapet. Other line elements were used to model the vertical beam and edge beams. Lumped masses were used at the nodes of the parapet elements. Uncracked sections were used for section properties. A transient analysis using Channel # 10 record as an input motion; 5% damping was considered. The result of the finite element analysis indicated that the first and second mode shapes provide for over 99% of the mass participation (78.6% and 21.3%, respectively). The first fundamental period is 0.079 seconds, and the second mode is 0.016 seconds.

Fig. 12 shows Mode # 1 and a comparison between the recorded and generated linear time history for the out-of-plane top acceleration. It is shown that an excellent prediction of the behavior can be obtained using the analytical model. A comparison between the stresses required by UBC94 Code provisions, and the recorded stresses scaled to an Upper Level event indicated that in this building the design of the parapet, based on the code formula, is conservative and should produce satisfactory results.

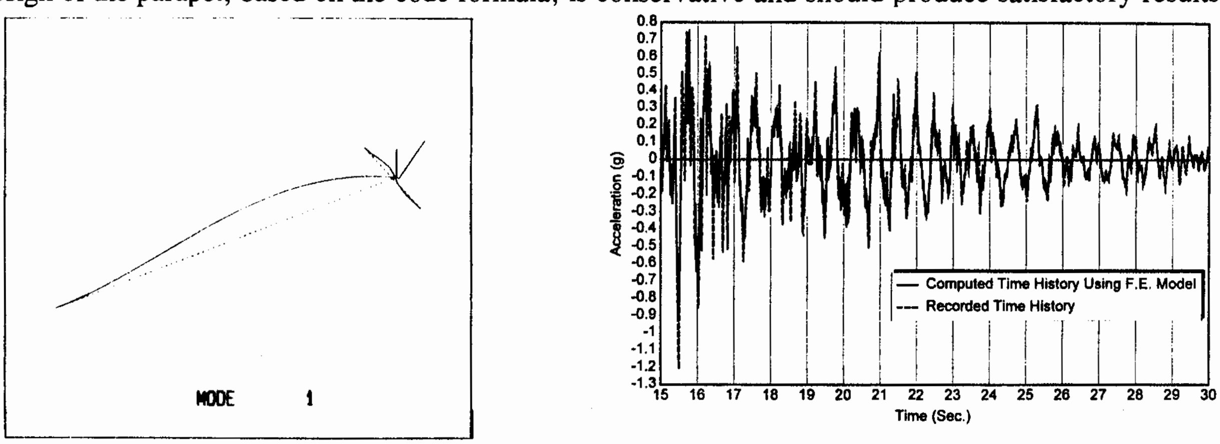



**Figure 12. Mode Shape and Comparison Between Recorded and Generated Time History for Roof Parapet**

## CONCLUSIONS

The following are the main conclusion of the study presented in this paper:

(1) The system identification resulted in models whose computed motions compared very closely to the parking structure's recorded earthquake motions. The model parameters that were estimated showed that: (a) the structure's response was dominated by its first north-south and first east-west translational modes of vibration in its two principal directions, together with its pseudostatic response component; (b) rocking of the structure about its base was an important contributor to its east-west translational response (and possibly to its north-south response as well, although rocking in the north-south direction was not explicitly measured by the current array of strong motion instruments at the structure); (c) the variations in the structure's estimated modal properties over the duration of the shaking suggests that nonlinear behavior was not a strong contributor to this parking structure's seismic response during the Northridge earthquake; and (d) Damping ratios varying from 4% to 5% of critical were estimated for the structure's significant modes of vibrations during the time window of strongest ground shaking.

(2) The finite element computer model described in this report was able to reasonably predict the main dynamic characteristics of the structure. It is clear that the cracked diaphragm model provides better correlation with the results obtained from MODE-ID method, particularly for the modes sensitive to in-plane diaphragm motion (i.e. Mode 1 and Mode 4 which correspond to the modes in the north-south direction).

(3) The finite elements results indicated that possibly, at a higher level of ground shaking, the drift ratio may exceed the value recommended by the current provisions of the code which is based on the $3(R_w/8)$ factor. Therefore, an increase of this factor to reflect the nonlinear response of the structure at higher levels of ground shaking is recommended. The results also indicated that the maximum uplift forces at the end of the shear walls during the Northridge earthquake exceeded the estimated uplift resistance forces. It is noted that the code stress checks will not predict this uplift behavior.

(4) The finite element results indicate that the roof probably experienced flexural cracking, and provided more flexible response, with a slight increase in the seismic forces. The finite element model indicates that diaphragm deflection is approximately 10% higher than the deflection at the end shear walls. Finite element mode with cracked diaphragms, indicate that this ratio increases to 16%.

(5) The study of the coil inserts connecting the precast columns and the concrete floors indicated that D/C ratios up to 5.25 can be obtained at some of these connections, when using the Code response spectrum. It is highly questionable that these connections will sustain such large demands without experiencing excessive damage. Failure of these connections may lead to the separation of the columns from the slab, which may result in columns' instability.

(6) For large span girders (65 ft long), effective floor vertical acceleration in the middle of the girder can be significant (up to 3.5 times the peak ground vertical acceleration). This large acceleration produces significant vertical loading that should be included in the design. Both the increase and the decrease of the total loads action on the girder should be considered. One design approach would be to increase or decrease the gravity loads acting on the beam with 2.5 times the peak ground vertical acceleration (in %g) times the gravity loads.

(7) A computer model was able to predicted the large acceleration recorded at the top of the roof parapet at the north side of the parking structure. It was shown that in this building the design of the parapet, based on the code formula, is conservative and should produce satisfactory results.

## ACKNOWLEDGMENT

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# PERFORMANCE OF 20 EXTENSIVELY-INSTRUMENTED BUILDINGS DURING THE 1994 NORTHRIDGE EARTHQUAKE

Farzad Naeim

Director of Research and Development
John A. Martin and Associates, Inc.

## ABSTRACT

In the aftermath of the January 17, 1994 Northridge earthquake hundreds of strong ground motion and building response accelerograms were retrieved from stations throughout the greater Los Angeles basin. Particularly important among the building response records were the data obtained from instrumented buildings which experienced relatively large ground motions. This paper provides a summary of the results obtained from an elaborate two-year project which included inspection of the buildings, damage assessment, performance evaluations. The forces, displacements, and dynamic characteristics interpreted from recorded data are contrasted with those suggested by building codes. Key response parameters and characteristics of each building are studied and where necessary observations are provided which may be used to improve future editions of the building codes.

## INTRODUCTION

According to Nehru every theory must be tempered with reality. The January 17, 1994 Northridge earthquake ($M_L = 6.4$, $M_W = 6.7$, $M_S = 6.8$) provided ample opportunity for earthquake engineers to test their theories and practices of structural design and seismic performance against the realities of strong ground shakings. Hundreds of strong ground motion and building response accelerograms were recorded by and retrieved from instruments installed by California Division of Mines and Geology, Strong Motion Instrumentation Program (Shakal and others, 1994; CSMIP 1994-95), United States Geological Survey (USGS, 1994) and other agencies throughout the greater Los Angeles basin.

Particularly important among the building response records were the data obtained from 17 instrumented buildings distributed throughout the Los Angeles area which experienced peak base accelerations greater than 0.25 g, two instrumented downtown skyscrapers which experienced ground level accelerations of about 0.18g, and a two-story base isolated Fire Command Control building which experienced a peak base acceleration of about 0.22g.

As a part of this investigation, the above buildings were inspected to the extent possible and their performance were evaluated relative to various aspects of recorded ground motion and building configuration. Building superintends and structural engineers who had examined the buildings were consulted and their observations were summarized. Detailed information on building structural systems, nonstructural systems, contents, construction history, extent and location of damage, and loss estimates were gathered.

For each building the code specified values for natural periods design base shears and drift indices were calculated. Two sets of code values were developed: one corresponding to the edition of the building code used in the actual design of each building , and the other based on the 1994 edition of the Uniform Building Code (ICBO, 1994). These values were compared with natural periods and maximum base shears interpreted from the earthquake records.

A unique feature of this project is development of a CD-ROM based interactive information system which contains all text, photos, sketches, earthquake records and most importantly all of the analytical tools which were developed and utilized for this study (Naeim, 1996). The companion SMIP Information System is a Microsoft Windows based system and is built around an open-architecture relational database system which can be modified and expanded by the users.

## DESCRIPTION OF THE BUILDINGS

The following buildings were studied as a part of this investigation. The acronyms used for identification of these buildings in the rest of the paper are given in parenthesis:

<u>Burbank 10-story residential building with 16 sensors (BURBANK 10)</u> This building was designed and constructed in 1974. Its vertical load carrying system consists of precast and poured-in-place concrete floor slabs supported by precast concrete bearing walls. The lateral load resisting system consists of precast concrete shear walls in both direction. The foundation system includes concrete caissons which are 25 to 35 feet deep. The largest peak horizontal accelerations recorded at the base and at the roof are 0.34g and 0.77g, respectively. The peak velocity at the roof is about 63 cm/sec.

<u>Burbank, 6-story commercial building with 13 sensors (BURBANK 6)</u> This steel moment frame building was designed in 1976 and constructed in 1977. The vertical load carrying system consists of 3" concrete slab over metal deck supported by steel frames. The lateral load resisting moment frames are located at the perimeter of the building. The foundation system includes concrete caissons approximately 32 feet deep. The largest peak horizontal acceleration recorded at the base and at the roof are 0.36g and 0.47g, respectively. The peak velocity at the roof is about 48 cm/sec.

<u>Los Angeles, 17-story residential building with 14 sensors (LARES 17)</u> This building was designed in 1980 and constructed in 1982. Its vertical load carrying system consists of 4" or 8" precast, pretensioned concrete slabs supported by precast concrete walls. The lateral load resisting system consists of distributed precast concrete shear walls in both direction. The foundation system includes 44" diameter and 54 feet long drilled piles. The largest peak horizontal acceleration recorded at the base and at the roof are 0.26g and 0.58g, respectively. The peak velocity at the roof is about 48 cm/sec.

Los Angeles, 19-story office building with 15 sensors (LAOFFI 19) This office building has 19 stories above the ground and 4 stories of parking structure below the ground. It was designed in 1966-67 and constructed in 1967. The vertical load carrying system consists of 4.5" reinforced concrete slabs supported on steel frames. The lateral load resisting system consists of moment resisting steel frames in the longitudinal and X-braced steel frames in the transverse direction. The foundation system consists of 72 feet long, driven, steel I-beam piles. The largest peak horizontal acceleration recorded at the base, ground floor and roof are 0.32g, 0.53g and 0.65g, respectively. The peak velocity at the roof is about 65 cm/sec.

Los Angeles, 2-story Fire Command Control building with 16 sensors (LACC 2) This is a 2 story seismic isolated building. The isolation system is composed of elastomeric bearings. The vertical load carrying system is steel vented roof decking and steel decking with 3 to 4 inches of concrete fill at the first and second floors. The floor system is supported by steel frames and rubber bearings. The lateral load resisting system is perimeter chevron braced frames above the isolation interface. The foundation system is composed of spread footings. The building was designed in 1988 and constructed in 1989-90. In the E-W direction, the largest peak horizontal accelerations recorded below the isolation plane, at the floor directly above the isolation plane, and the roof are 0.22g, 0.35g and 0.77g, respectively. In the N-S direction, the largest peak horizontal accelerations vary from 0.18g at the base to 0.07g directly above the isolation system and 0.09g at the roof.

Los Angeles, 3-story commercial building with 15 sensors (LACOMM 3) This department store building has three stories above and two parking levels below the ground. The building was designed in 1974 and constructed in 1975-76. The vertical load carrying system consist of 3.25 inches of light-weight concrete slab over metal deck in upper three floors and 18 inches thick waffle slabs in the basement floors. The lateral load resisting system is steel braced frames in the upper three stories and concrete shear walls in parking floors. The foundation system consists of spread footings and drilled bell caissons. The largest peak horizontal accelerations recorded at the base is 0.33g. At the roof peak horizontal acceleration of 0.97g and peak velocity of 57 cm/sec were recorded.

Los Angeles, 5-story Warehouse with13 sensors (LAWH 5) This is a 5-story reinforced concrete building was constructed in 1970 with perimeter ductile concrete frames acting as its lateral system. The largest peak horizontal accelerations recorded at the base and at the roof are 0.25g and 0.28g, respectively. The peak velocity at the roof is about 34 cm/sec.

Los Angeles, 52-story office building with 20 sensors (LAOFFI 52) This office building has 52 stories above and 5 levels below the ground. It was designed in 1988 and constructed in 1988-90. The vertical load carrying system consists of 3 to 7 inches of concrete slabs on steel deck supported by steel frames. The lateral force resisting system consists of concentrically braced steel frames at the core with moment resisting connections and outrigger moment frames in both directions. The foundation is composed of spread footings of 9 to 11 feet thickness. The largest peak horizontal accelerations recorded at the basement and at the roof are 0.15g and 0.41g, respectively. The peak velocity at the roof is about 40 cm/sec.

Los Angeles, 54-story office building with 20 sensors (LAOFFI 54) This office building has 54 stories above and 4 levels below the ground. It was designed in 1988 and constructed in 1988-90. The vertical load carrying system consists of 2.5 inches of concrete slabs on a 2inche metal deck supported by steel frames. The lateral force resisting system consists of perimeter tubular moment resisting frames which step in twice in elevation.. The foundation system consists of a 9 feet deep mat foundation. The largest peak horizontal accelerations recorded at the basement and at the roof are 0.14g and 0.19g, respectively. The peak velocity at the roof is about 34 cm/sec.

Los Angeles, 6-story office building with 14 sensors (LAOFFI 6) This office building has five stories above and one level below the ground. It was designed in 1988 and constructed in 1989. The vertical load carrying system consists of composite construction of concrete slabs over metal decks supported by steel frames. The lateral load resisting system is a combination of Chevron braced and moment resisting steel frames. Mat foundations are utilized beneath the four towers and spread footings are used elsewhere. The largest peak horizontal accelerations recorded at the base and at the roof are 0.24g and 0.48g, respectively. The peak velocity at the roof is about 70 cm/sec.

Los Angeles, 6-story parking structure with 14 sensors (LAPARK 6) The first three stories of this concrete parking structure were constructed in 1977. The upper three floors were added in 1979 based on designs dated 1975 and 1978. The vertical load carrying system consists of 5.75 in. concrete slabs and 5 in. post-tensioned concrete slabs supported by precast concrete beams and columns (see the Information System photos). The lateral force resisting system consists of six cast-in-place reinforced concrete shear walls in the North-South direction and two in the E-W direction. The foundation system is made of drilled concrete caissons. The largest peak horizontal acceleration recorded at the base, near the north-east shear is 0.29g. Channel 1 at the base of the North shear wall recorded a peak vertical acceleration of 0.22g. At the roof, the sensor placed on the mid-span of a girder recorded a peak vertical acceleration of 0.52g. Elsewhere on the roof, a sensor attached to a parapet on the North side recorded peak horizontal and vertical accelerations of 1.21g and 0.52g, respectively.

Los Angeles, UCLA Math-Science building with 18 sensors (UCLA7) This Math-Science addition to the engineering school building at UCLA is a 7 story building with no basement. It is separated by seismic joints from adjacent wings of the building. The vertical load carrying system for the upper floors (third and higher) consists of 2.5 inches of concrete slab over metal deck supported by steel frames. At the third floor a thick concrete slab supported by concrete walls make up the gravity system. The lateral load resisting system consists of concrete shear walls between the base and the third floor and moment resisting steel frames from the third floor to the roof. The largest peak horizontal accelerations recorded at the base and roof are 0.29g and 0.76g, respectively. The maximum velocity recorded at the roof is about 73 cm/sec.

Los Angeles, 7-story hospital building in with 24 sensors (LAHOSP 7) This structure is the first base isolated hospital building in the United States. It was designed in 1988 and constructed between 1989 to 1991. The vertical load carrying system consists of concrete slabs on metal decks supported by steel frames and rubber isolators. The lateral force resisting system consists of diagonally braced perimeter steel frames isolated by lead-rubber and elastomeric isolator units.

Foundation system consists of continuous and isolated spread footings.. The largest free-field peak horizontal acceleration recorded adjacent to the building is 0.49g in the N-S direction. The largest horizontal peak acceleration recorded at the foundation, immediately above the isolation plane, and at the roof of the building are 0.37g (N-S), 0.14g (E-W), and 0.21g (N-S).

<u>Los Angeles, 9-story office building with 18 sensors (LAOFFI 9)</u> This office building was designed and constructed in 1923. It consists of concrete frames with unreinforced masonry infill walls. It consists of one level of basement and 9 floors above the ground. The largest peak horizontal accelerations recorded at the basement and roof are 0.18g and 0.34g , respectively. The maximum velocity recorded at the roof is about 45 cm/sec.

<u>Los Angeles, Hollywood Storage Building with 12 sensors (HWSTOR)</u> This building has 14 stories above and one level below the ground. It was designed in 1925. The vertical load carrying system consists of 8 in. thick concrete slabs supported by concrete frames. The lateral load resisting system, consists of reinforced concrete frames in both directions. The deep foundation system consists of concrete piles. The "free-field" station adjacent to the building recorded peak accelerations of 0.41g in the N-S direction, 0.19g in the E-W direction, and 0.19g in the vertical direction. The maximum peak horizontal accelerations recorded at the base and at the roof are 0.28g and 0.49g, respectively. The uncorrected trace of Channel 11 at the roof shows a peak acceleration of 1.61g. However, at the time of publishing this paper the corrected version of this record was not available. It is possible that this sensor was not properly calibrated at the time of the earthquake since it has high frequency content which is not corroborated by other instruments. The peak velocity at the roof is about 38 cm/sec.

<u>North Hollywood, 20-story hotel with 16 sensors (NHHOTEL 20)</u> This hotel has 20 stories above and one level below the ground. It was designed in 1967 and constructed in 1968. The vertical load carrying system consists of 4.5 to 6 inches thick concrete slabs supported by concrete beams and columns. The lateral load resisting system consists of ductile moment resisting concrete frames in the upper stories and concrete shear walls in the basement. The exterior frames in the transverse direction are infilled between the second and the $19^{th}$ floors. The building rests on spread footings. The largest peak horizontal accelerations recorded at the basement and at the roof are 0.33g and 0.66g, respectively. The largest velocity recorded at the roof is about 77 cm/sec.

<u>Sherman Oaks, 13-story commercial building with 15 sensors (SHERMAN 13)</u> This office building has 13 stories above and two floors below the ground. It was designed in 1964. The vertical load carrying system consists of 4.5 inches thick one-way concrete slabs supported by concrete beams, girders and columns. The lateral load resisting system consists of moment resisting concrete frames in the upper stories and concrete shear walls in the basements. The foundation system consists of concrete piles. The first floor spandrel girders were modified by post-tensioning after the 1971 San Fernando earthquake. The largest peak horizontal accelerations recorded at the basement and at the roof are 0.46g and 0.65g, respectively. The middle floors experienced large acceleration in the neighborhood of 0.6g. The largest velocity recorded at the roof is about 68 cm/sec.

Sylmar, 6-story hospital building with 13 sensors (SYLMAR) The Sylmar County Hospital Building is a unique building built on the site of the old Olive View hospital building which suffered major and irreparable damage during the 1979 San Fernando earthquake. This six story cruciform shaped building has no basements. It was designed in 1976 and was constructed during the period of 1977 to 1986. Its vertical load carrying system consists of concrete slabs over metal deck supported by steel frames. The lateral load resisting system consists of concrete shear walls in lower two floors and steel shear walls encased in concrete at the perimeter of the upper four floors. The building rests on spread footings. The "free-field" station located at the parking lot adjacent to the building recorded 0.91g, 0.61g, and 0.60g in the N-S, E-W, and vertical directions, respectively. The largest peak horizontal accelerations recorded at the ground floor and at the roof of the building are unprecedented at 0.80g and 1.71g, respectively. The largest velocity recorded at the roof was as large as 140 cm/sec.

Van Nuys, 7-story hotel with 16 sensors (VAN NUYS 7) This reinforced concrete structure with no basements was designed in 1965 and constructed in 1966. Its vertical load carrying system consists of 8 in. and 10 in. concrete slabs supported by concrete columns, and spandrel beams at the perimeter. The lateral load resisting system consists of interior column-slab frames and exterior column-spandrel beam frames. The foundations consist of 38 inch deep pile caps, supported by groups of two to four poured-in-place 24 inch diameter reinforced concrete friction piles. The largest peak horizontal accelerations recorded at the basement and at the roof are 0.45g and 0.58g, respectively. The largest velocity recorded at the roof is about 77 cm/sec.

## RESULTS OBTAINED

The 20 instrumented buildings exhibited structural and nonstructural damages ranging from *None* to *High* based on the ATC-38 post-earthquake evaluation procedure. Hundreds of photos exhibiting various types of damage to these buildings and a wide variety of analytical tools developed as a part of this project are available on the CD-ROM based information system which was developed as a part of this investigation (Naeim, 1996). A few sample representatives are reproduced in Figures 1 to 9.

Interpreted maximum direct (N-S or E-W) and differential (torsional) base shears and drift indices are presented in Table 1 where interpreted base shears are compared with recommended code strength design values. Overall levels of structural and nonstructural damage are also indicated on this table.

Interpreted vibration periods of these buildings are compared to code recommended period estimates in Table 2 where predominant periods --if significantly different from fundamental periods-- are identified. This table also shows the shifts in building periods during and after the earthquake.

In light of the results of this investigation the following observations are made:

1. Building code estimates of building periods are consistently less than both the initial and final fundamental periods obtained from interpretation of recorded data. UBC-94 estimates,

however, are much better than the estimates provided by the older editions of the code. It may be necessary to further calibrate code period estimation formulas to reduce this inconsistency.

2. Except for the two base isolated buildings and the two downtown skyscrapers, the building base shears obtained from interpretation of recorded data are larger, sometimes substantially, than the base shears they have been apparently designed for. With the exception of the Van Nuys 7-story hotel, however, these buildings behaved remarkably well given the magnitude of forces they were subjected to. One could suggest that all these buildings performed much better than what would have been expected by routine design analysis techniques. Design procedures need to be modified to take advantage of this excess capacity which is not ordinarily addressed in design analysis schemes.

3. The ratio of the base shears experienced to design code base shears does not correlate very well with the extent of damage observed. The overall drift ratio, however, does correlate rather well. This statement, however, needs further clarification through system identification studies since it is not clear at this time whether the large drifts are contributing to damage or are caused by it.

4. Given the level of forces the building experienced, the overall drift ratios experienced are significantly less than what would have been expected from ordinary design analysis techniques.

5. While structural damage was generally less than what would have been expected, the content damage was generally extensive and usually the dollar value of the content damage and loss of occupancy far out-weighed the cost of structural repair.

6. In seismic response of majority of the buildings, different modes became predominant at different times during the response. In many cases, particularly for taller buildings such as the downtown skyscrapers, the Sherman Oaks 13-story office building, and the North Hollywood 20-story hotel, $2^{nd}$ and or $3^{rd}$ modes had more contribution to the overall response than the fundamental mode. In such cases application of the lateral story force profiles as suggested by static lateral force procedures may grossly underestimate the demand on the middle floors of the building. This can be further illustrated by examining the story force diagrams at the time of maximum base shear which indicate that except for the shorter buildings, the story force profile at the instant of maximum base shear is radically different from that recommended by static lateral force procedures. Lateral force distribution over the height of the building as suggested by static lateral force procedures is generally based on the static deflected shape of the building. Evaluation of the deformed shape at the time of maximum lateral displacement shows that the lateral deformation at this instant almost always follows a shape similar to the first mode of vibration. Our studies indicate, however, that in most cases maximum forces and maximum displacements are not concurrent. In most cases the maximum force response occurs first and the maximum displacement response occurs many seconds later. Current edition of the UBC code requires dynamic (i.e., response spectrum) distribution of forces for irregular structures. In light of observations presented

here it might be prudent to require dynamic distribution of forces for buildings exceeding a certain height (65 feet for example) and limiting the application of static lateral force distribution to the regular buildings of less height.

7. Except for the case of the 6-story Sylmar County hospital, behavior of mounted mechanical equipment was not a strong function of the severity of the ground motions but rather the quality of design and construction. (see for example photos of equipment mounted at the roof of the 3-story commercial building or the Van Nuys 7-story hotel in the Information System developed as a part of this project).

8. Except for buildings with observed structural damage, the period of the building as interpreted from the recorded data did not elongate significantly and when elongation occurred the period came back to the vicinity of the initial value towards the end of the ground motion. The period of damaged buildings however did decidedly elongate.

9. For several buildings, torsion contributed significantly to the seismic response. In one of these cases ( Van Nuys 7-story hotel) the building experienced major damage.

## CONCLUSIONS

An interactive information system was developed which contains all of the gathered information, inspection results, recorded data, performance analysis results, and analytical tools utilized for evaluation of twenty extensively-instrumented buildings which were subjected to significant ground shaking during the January 17, 1994 Northridge earthquake (see Naeim, 1996). This CD-ROM based interactive information system can be a very valuable tool in teaching and learning earthquake engineering and seismic response principles as well as a tool for further research on response of buildings to strong earthquake ground motions. A brief overview of the information generated regarding the seismic performance of these buildings were presented. The interested reader is referred to Naeim (1996) report to California Strong Motion Instrumentation Program and its companion CD-ROM based information system for more details.

## ACKNOWLEDGMENTS

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The opinions expressed in this paper are those of the author and do not necessarily reflect the views of the California Strong Motion Instrumentation Program or John A. Martin and Associates, Inc.

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**TABLE 1. Base Shear, Drift and Overall Damage Summary**

| Building Acronym | Maximum Direct Base Shear Interpreted (% Total Weight)[a] | Maximum Differential Base Shear Interpreted (% Total Weight)[a] | Design Code Strength Level Base Shear (% Total Weight)[b] | UBC-94 Strength Level Base Shear (% Total Weight)[c] | Maximum Overall Drift Index Interpreted (in./in.)[a] | Overall Structural Damage[d] | Overall Non-strucutral and/or Equipment Damage[d] |
|---|---|---|---|---|---|---|---|
| BURBANK 10 | 34 | 14 | 14 | 20 | 0.0023 | Insignificant | Insignificant |
| BURBANK 6 | 22 | 7 | 10 | 7 | 0.0039 | Insignificant | Moderate |
| LARES 17 | 17 | 16 | 18 | 18 | 0.0022 | Insignificant | Insignificant |
| LAOFFI 19 | 34 | 22 | 8 | 12 | 0.0039 | Moderate | Insignificant |
| LACC 2 | 7 | 11 | --[e] | --[e] | -- | None | Moderate |
| LACOMM 3 | 49 | 27 | 18 | 27 | 0.0111 | None | Insignificant |
| LAWH 5 | 17 | 10 | 8 | 17 | 0.0020 | Insignificant | Insignificant |
| LAOFFI 52 | 9 | 4 | --[e] | --[e] | 0.0011 | None | None |
| LAOFFI 54 | 4 | 4 | --[e] | --[e] | 0.0008 | None | None |
| LAOFFI 6 | --[f] | --[f] | --[f] | --[f] | -- | None | None |
| LAPARK 6 | 27 | 13 | 12.5 | 18 | 0.0016 | Insignificant | Insignificant |
| UCLA 7 | 27 | 21 | --[f] | --[f] | 0.0047 | Moderate | Insignificant |
| LAHOSP 7 | --[g] | --[g] | --[e] | --[e] | -- | None | None |

TABLE 1. (Continued)

| Building Acronym | Maximum Direct Base Shear Interpreted (% Total Weight)[1] | Maximum Differential Base Shear Interpreted (% Total Weight)[1] | Design Code Strength Level Base Shear (% Total Weight)[2] | UBC-94 Strength Level Base Shear (% Total Weight)[3] | Maximum Overall Drift Index Interpreted (in./in.)[a] | Overall Structural Damage | Overall Non-strucutral and/or Equipment Damage |
|---|---|---|---|---|---|---|---|
| LAOFFI 9 | 20 | 18 | --[f] | --[f] | 0.0018 | Insignificant | Insignificant |
| HWSTOR | 10 | 10 | --[f] | --[f] | 0.0013 | Insignificant | Insignificant |
| NHHOTEL 20 | 12 | 9 | 6 | 6 | 0.0036 | Insignificant | Moderate |
| SHERMAN 13 | 13 | 3 | 6 | 6 | 0.0067 | Moderate | Insignificant |
| SYLMAR | 97 | 63 | --[e] | --[e] | 0.0022 | Insignificant | High |
| VAN NUYS 7 | 33 | 41 | 7 | 21 | 0.0117 | High | Moderate |

NOTES:

a) For analytical assumptions see (Naeim, 1996).
b) Estimate of the code WSD value times 1.4 at the time of building design.
c) Estimate of the UBC-94 code WSD value times 1.4.
d) ATC-38 Definitions of overall damage states are used.
e) Design was not based on code static lateral force procedures.
f) Sufficient information not available for compiling this value.
g) Approximate methods used are not applicable to this case.

**TABLE 2. Vibration Periods (seconds)**

| Building Acronym | Interpreted N-S & E-W Fundamental Periods [a] | Interpreted Predominant Response Periods[a] | Significant Period Elongation?[a] | Design Code Period Estimates[a] | UBC-94 Period Estimates[a] |
|---|---|---|---|---|---|
| BURBANK 10 | 0.57 - 0.62 | Same | No | 0.30 | 0.58 |
| BURBANK 6 | 1.28 - 1.28 | Same | No | 0.60 | 0.95 |
| LARES 17 | 0.80 - 1.20 | Same | Moderate | --[b] | --[b] |
| LAOFFI 19 | 2.60 - 3.41 | Same | No | 0.76 - 1.90 | 1.24 - 2.33 |
| LACC 2 | 1.28 - 0.2 to 1.14 | Same | Yes in E-W Dir. | --[b] | --[b] |
| LACOMM 3 | 0.55 - 0.51 | Same | No | 0.16 | 0.40 |
| LAWH 5 | 1.46 - 1.37 | Same | No | 0.60 | 0.73 |
| LAOFFI 52 | 6.0 - 6.0 | 1.6 to 2.0 | No | --[b] | --[b] |
| LAOFFI 54 | 6.0 - 6.0 | 1.0 to 2.0 | No | --[b] | --[b] |
| LAOFFI 6 | 0.85 - 0.85 | Same | No | 0.56 | 0.56 |
| LAPARK 6 | 0.35 - 0.40 | Same | No | 0.18 | 0.44 |
| UCLA 7 | 0.66 - 1.02 | Same | No | --[b] | --[b] |
| LAHOSP 7 | 0.64 to 1.5 - 0.79 to 1.5 | Same | Yes | --[b] | --[b] |
| LAOFFI 9 | 1.21 - 1.71 | Same | No | --[f] | --[f] |
| HWSTOR | --[b] | --[b] | --[b] | --[b] | --[b] |
| NHHOTEL 20 | 2.20 - 2.50 | Same & 0.70 | Moderate | 1.20 | 1.60 |
| SHERMAN 13 | 2.6 - 2.9 | Same & 1.0 | Yes | 1.27 | 1.60 |
| SYLMAR | 0.46 - 0.46 | Same | No | --[b] | --[b] |
| VAN NUYS 7 | 1.1 to 1.8 - to 2.2 | Same | Yes | 0.70 | 0.70 |

a) for analytical assumptions, methods and procedures see (Naeim, 1996)

b) Either sufficient information not available for compiling this value or value not applicable.

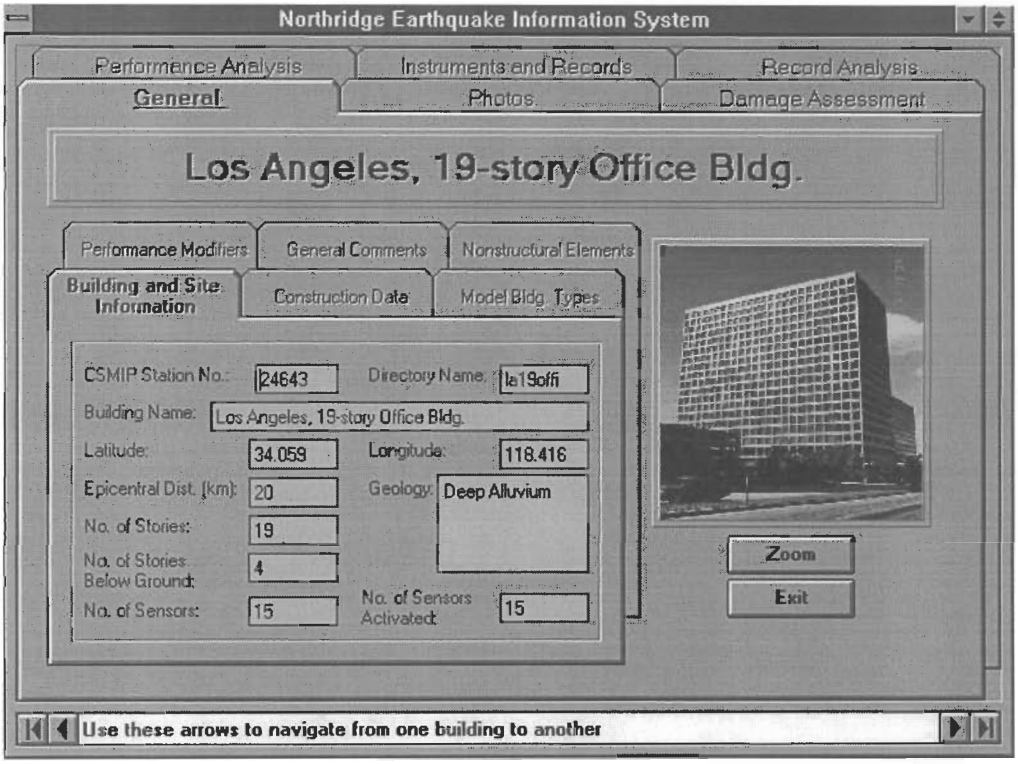



Figure 1. The Information System main folder LAOFFI 19 building.

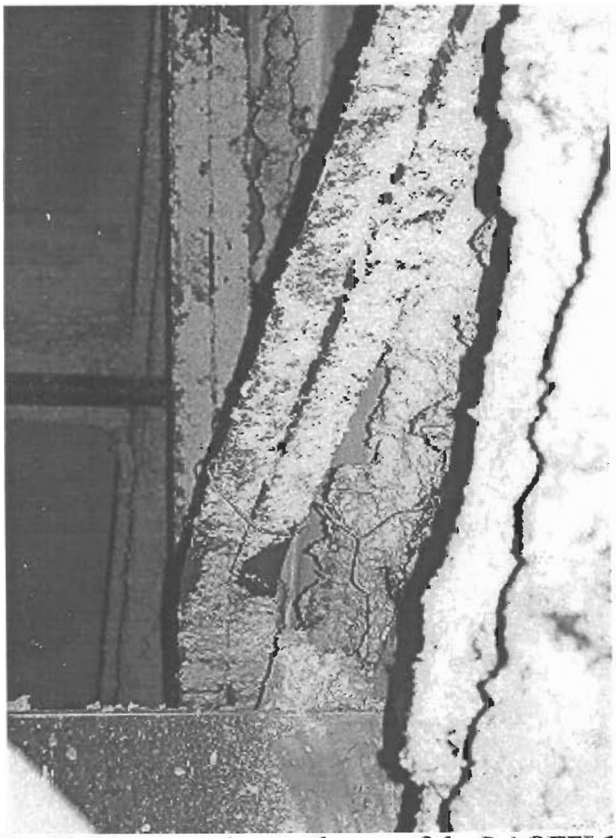


Figure 2. Buckled brace at the penthouse of the LAOFFI 19 building.

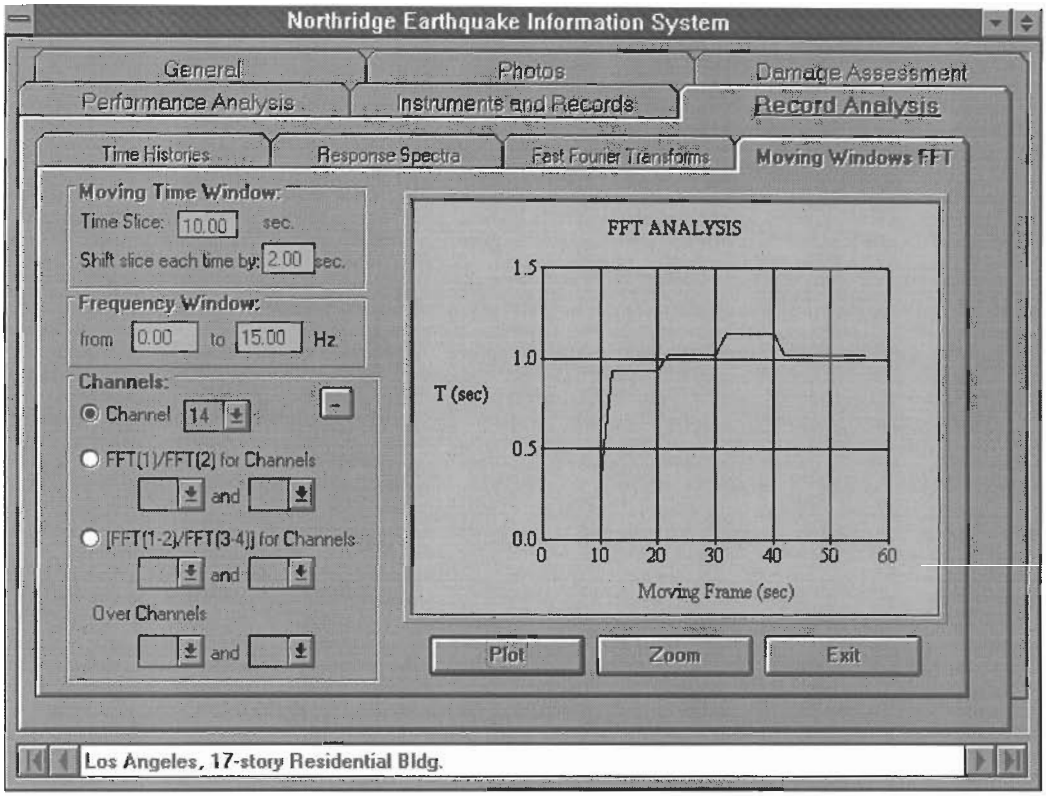



Figure 3. Moving windows FFT analysis for the N-S response of the LARES 17 bldg.

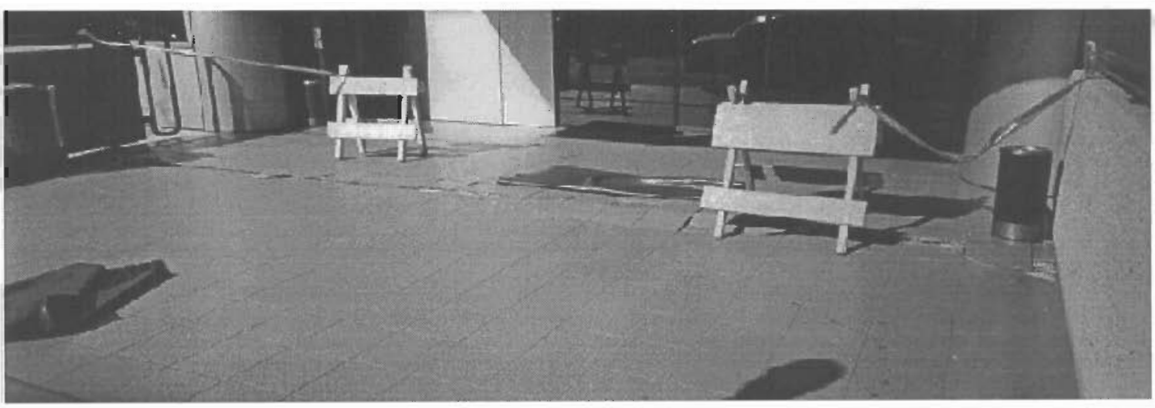
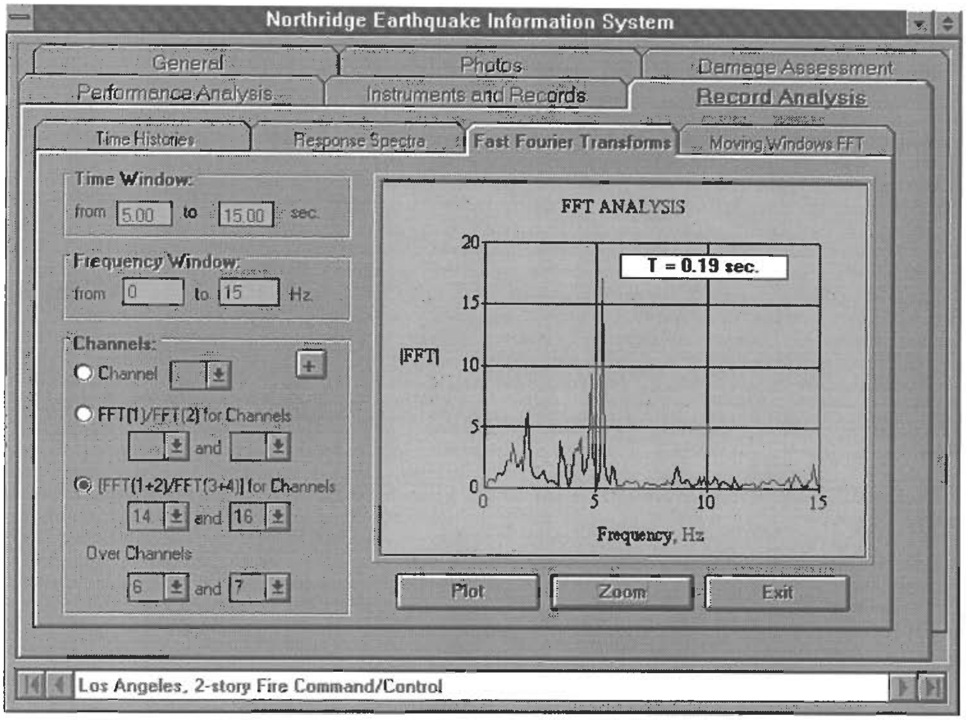
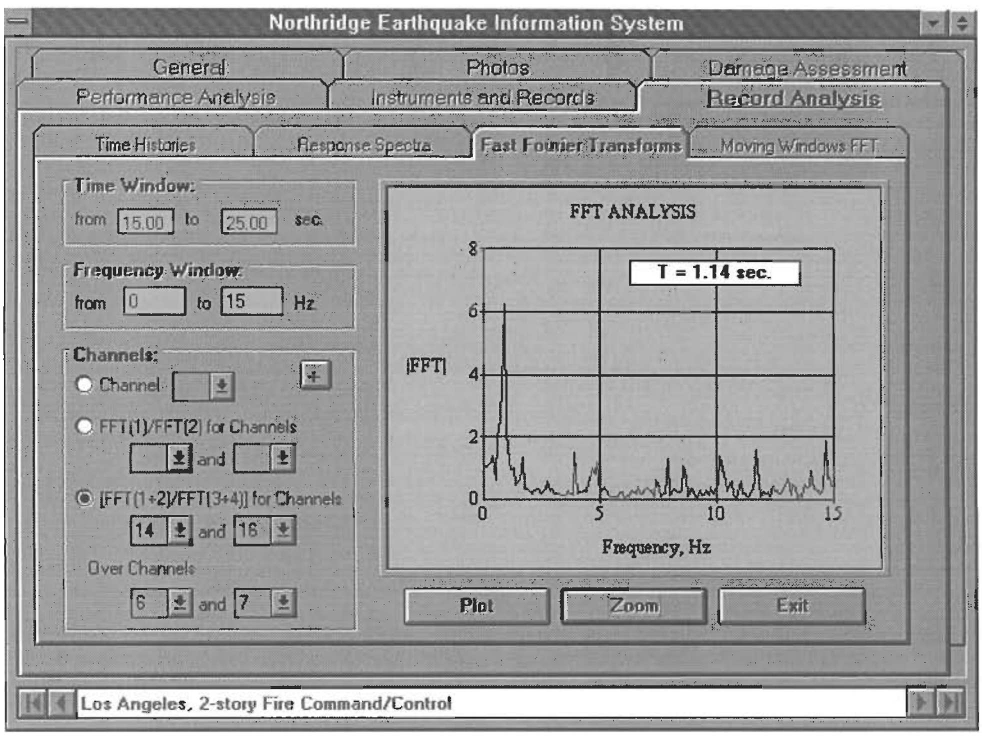


Figure 4. Tiles over isolation pit of the LACC 2 bldg. after the earthquake (photo courtesy of Robert Bachman).




(a) time = 0 to 15 seconds




(b) time = 15 to 25 seconds

Figure 5. Fast Fourier Transform of E-W response of LACC 2 building shows that the pit separation shown in Figure 4 has permitted the building to behave as an isolated system after 15 seconds into the ground motion.

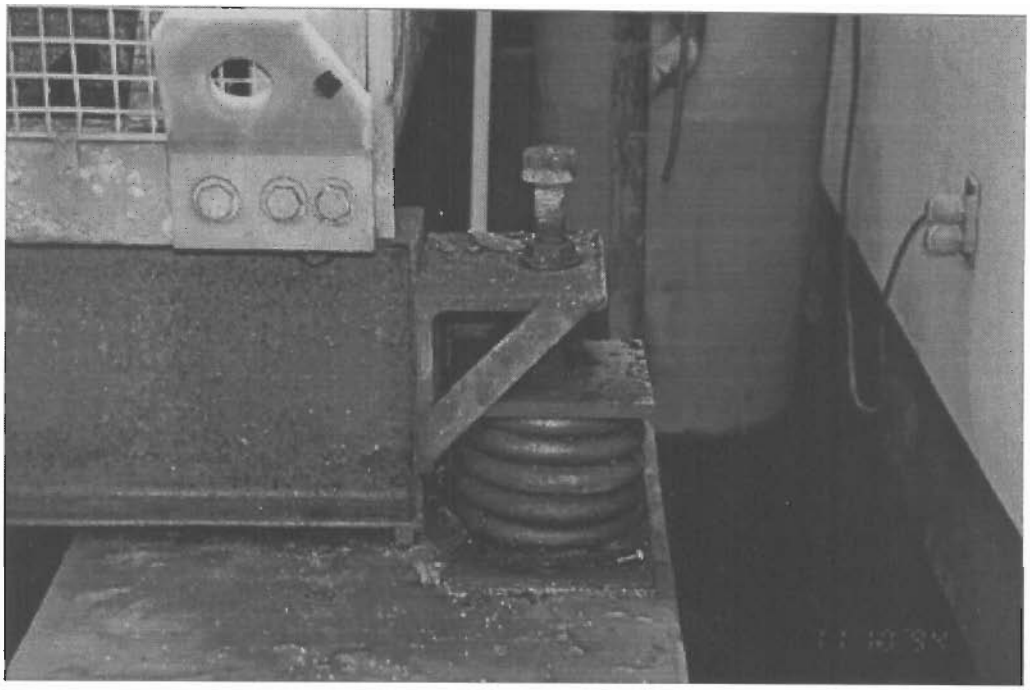


Figure 6. Mechanical equipment damage at the roof of BURBANK 6 building.

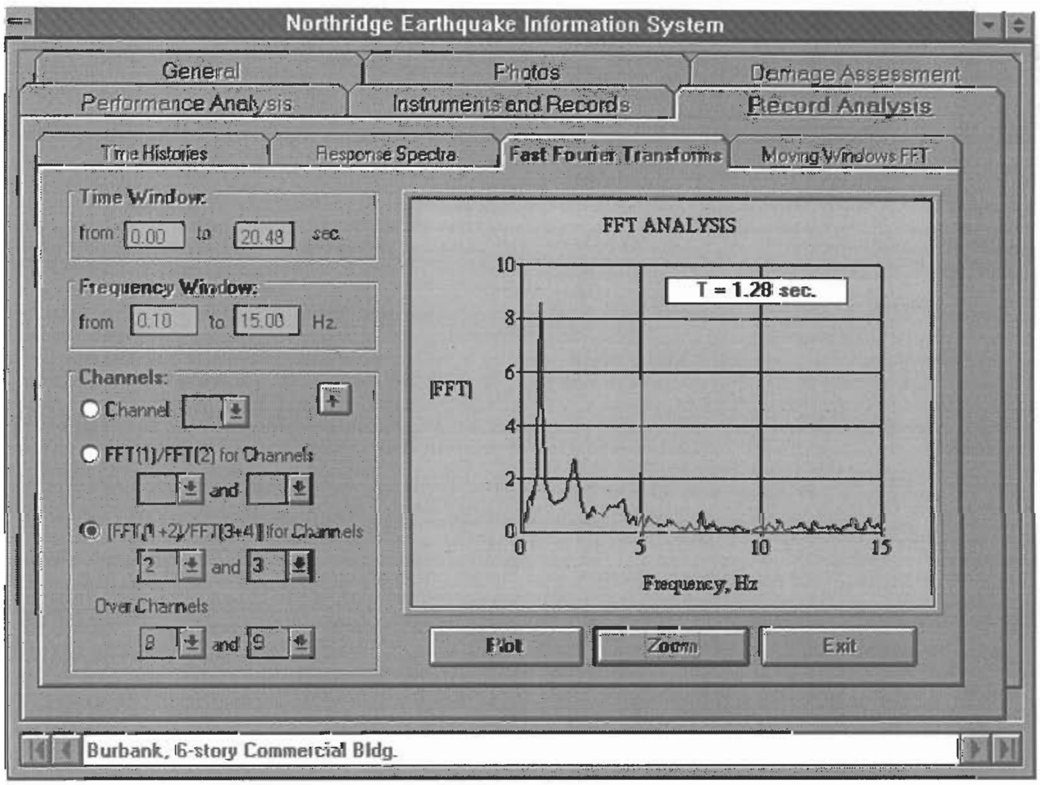



Figure 7. FFT analysis depicting fundamental N-S period of BURBANK 6 building.

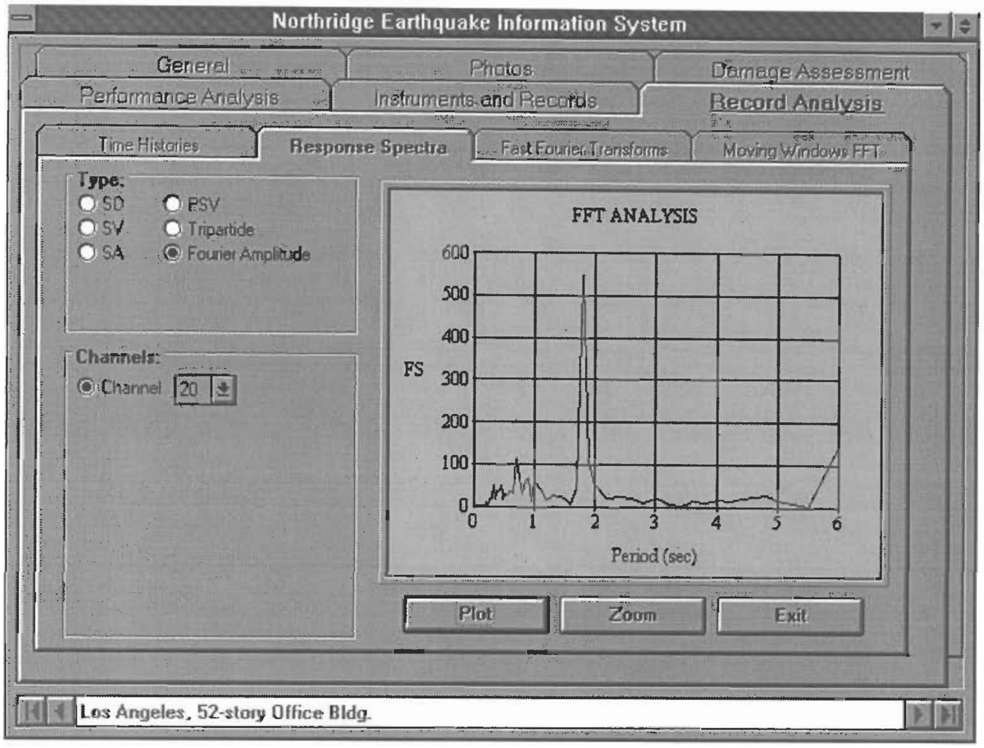



Figure 8. Fourier spectrum indicating a fundamental period of 6.0 sec. For LAOFFI 52 building. Notice that the predominant period however is slightly below 2 seconds.

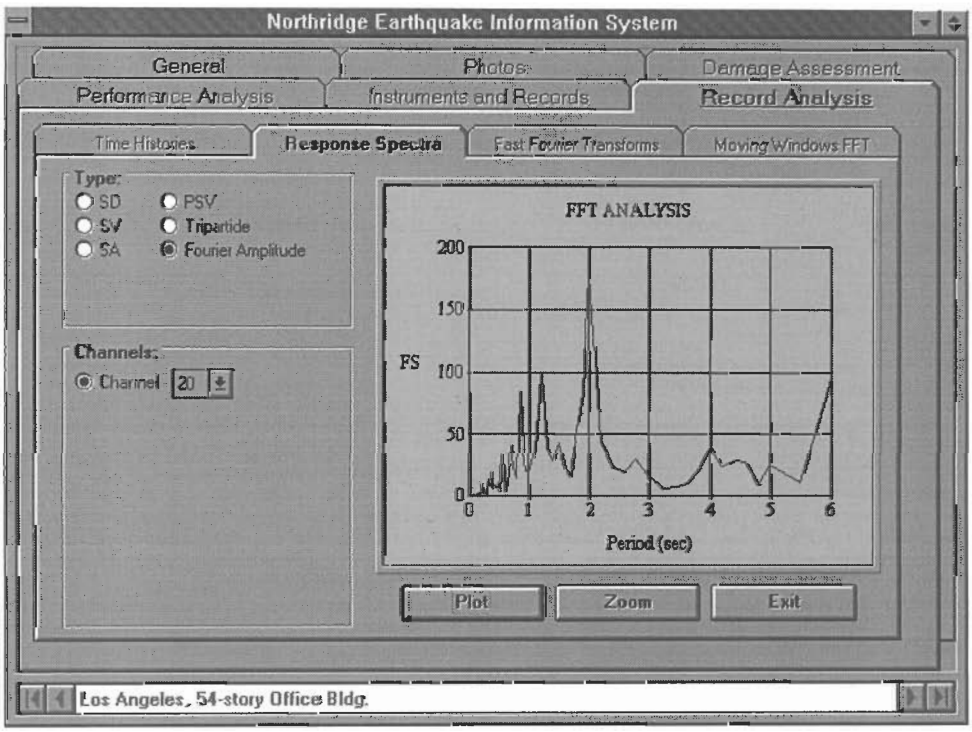



Figure 9. Fourier spectrum indicating a fundamental period of about 6.0 sec. For LAOFFI 54 building. Notice that the predominant period however is at about 2 seconds.

# VERIFICATION OF RESPONSE SPECTRAL SHAPES AND ANCHOR POINTS FOR DIFFERENT SITE CATEGORIES FOR BUILDING DESIGN CODES

Walt Silva[1] and Gabriel Toro[2]
Pacific Engineering and Analysis, El Cerrito, California[1]
Risk Engineering Inc., Boulder, Colorado[2]

## ABSTRACT

The dramatic increase in strong ground motion recordings over the last several years has provided both the impetus and opportunity to empirically examine the seismic design criteria in both the UBC and NEHRP code provisions. In this project both spectral design shapes and the usefulness of two spectral anchors are investigated using a comprehensive strong motion database and updated empirical attenuation relations. For the shapes, the results suggest that both the UBC and NERHRP design spectra provide enveloping criteria (except for site D at short periods) including cases for sites within 10 km of the fault rupture surface. For the NEHRP design spectra, comparison of the Fa and Fv factors to those implied by a recently developed empirical attenuation relation suggest that the NEHRP Fa factors may reflect too little nonlinearity while the Fv factors may show too much nonlinearity. Comparisons of the code shapes to the results from probabilistic seismic hazard analyses indicate that the fixed UBC shape has a moderate tendency to under-predict amplitudes for $T > 1$ sec in places like San Francisco and Sacramento where large ($M > 7$) earthquakes dominate the hazard at these periods. The more flexible NEHRP shape avoids this problem, but requires the specification of two anchoring points.

## INTRODUCTION

Local geologic conditions have long been recognized to have a large effect upon strong ground motions. Del Barrio, in the 1855 Proceedings of the University of Chile states[1] "...a movement... must be modified while passing through media of different constitutions. Therefore, the earthquake effects will arrive to the surface with higher or lesser violence according to the state of aggregation of the terrain which conducted the movement. This seems to be, in fact, what we have observed in the Colchagua Province (of Chile) as well as in many other cases". The stable variations in spectral content for different site conditions give rise to site dependent ground motion characteristics which result from vertical variations in soil properties (Mohraz, 1976; Seed et al., 1976). These effects have been incorporated into building codes in the United States (UBC; NEHRP, 1991) as well as elsewhere (IAEE, 1992) as site category dependent response spectral shapes. The current UBC site categories (Table 1) and shape coefficients were developed primarily during the ATC-3 effort in the early to mid 1970's and reflect the state of knowledge at that time. Site categories S1 to S3 were developed by Seed et al. (1976) and category S4 was added subsequent to the 1982 Mexico City earthquake to accommodate deep soft (clay) profiles. The site coefficients corresponding to the S1 to S4 site categories affect only the intermediate-to-long period portion of the spectrum (range of approximately constant response spectral velocity) as insufficient data were available to resolve stable features of short period site dependent response. It is important to point out that site dependent spectral shapes reflected in the code provision not only represent ground response spectra, but also accommodate judgmental factors for structural engineering considerations, such

[1]Translated from the old Spanish by Professor Ricardo Dobry.

as a factor of 1.25 along with a $T^{2/3}$ fall-off at long periods.

Subsequent to the 1989 Loma Prieta earthquake and the accompanying dramatic increase of data on site effects, the need was recognized for an update of the code provision site factors. This effort was undertaken by a number of practitioners and culminated in a consensus (1992 NCEER/SEAOC/BSSC/Workshop) set of revised site categories (A to F, Table 1) and a set of both short period (Fa) and intermediate-to-long period (Fv) site factors applied to the soft rock (Category B) shape. The new site factors accommodate nonlinear site effects through a dependency on soft rock acceleration and velocity based effective accelerations ($A_a$ and $A_V$, Table 2). For these NEHRP proposed revisions, the factor of 1.25 has been changed to 1.2 and the $T^{2/3}$ spectral decay has been retained. In addition, for categories D, E, and F, at periods shorter than 0.3 sec, the design coeffient is reduced with the multiplicative factor 1 + 5 T. For very long periods (> 4 sec), the design coeffient is increased over the $T^{2/3}$ decay with a $T^{4/3}$ decay.

Since the UBC spectra were developed in the early to mid 1970's and the NEHRP recommended revisions are based primarily on site response analyses (categories D and E at rock PGA'S ≤ 0.1g reflect limited empirical analyses) the increase in strong motion data over the last several years provides an important empirical check on these design requirements. The purpose of this project is to provide such an empirical check on the shapes as well as to investigate the benefits of using spectra anchored at two points as in the NEHRP recommended provisions (1994).

## VERIFICATION OF RESPONSE SPECTRAL SHAPES

### Strong Motion Database

The strong motion database available for development of statistical 5% damped response spectral shapes was compiled by Pacific Engineering and Analysis and contains of 98 earthquakes in the magnitude range of about **M** 5 to **M** 7.4. The database consists of over 2,000 recordings and most of the available volume 1 records have been reprocessed to extend both the short and long period portions of the motions. Recording sites have been assigned available USGS site codes (Boore et al., 1994). Sites which did not have assigned codes but which had available shear-wave velocity profiles were classified according to the criterion listed in Table 1. Records were assigned UBC site categories based on the following assumption: S1 = NEHRP B+C, S2 = NEHRP C+D, and S3 = NEHRP E.

For applications to code shapes, the magnitude selection was limited to **M** 6.3 and above. The breakdown in mean magnitudes, distances, and PGA's for each site category is shown in Table 3. Also shown in Table 3 are the number of records (spectra) and earthquakes contributing to each site category. Excluding the near-fault 0 to 10 km shapes, the average magnitude is about 6.75 at a distance of about 35 to 40 km and with a mean PGA value of about 0.15g.

### Soil Profile Database

To illustrate the type of shear-ware velocity profiles which are implied by the code categories, median and ± 1 sigma profiles were developed for the UBC and NEHRP generic sites (Figure 1). For UBC sites, as with the strong motion data, it was assumed that S1 is comprised of NEHRP B+C, S2 is comprised of NEHRP C+D, and S3 is equivalent to NEHRP E (Bill Joyner, personal communication). The generic profiles were produced from the Pacific Engineering and Analysis profile database of over 700 shear-wave velocity profiles using the criteria listed in Table 1. NEHRP Site A, which represents very hard rock, is not currently represented in the profile database. The decrease in overall stiffness as well as variability is quite apparent in going from NEHRP Sites B to E and from UBC Sites S1 to S2.

## UBC Shapes

Figure 2 shows the statistical shapes compared to the UBC code provisions for UBC sites S1, S2, and S3 (two few data were available for site S4) and Table 3 shows the data distributions in **M**, distance, and number of records. The UBC spectral shape requirements are shown for both the static (lateral force) and dynamic analyses. The unbiased static provisions, which use a spectral decay of $T^{-1}$ instead of $T^{-2/3}$ are also shown. The statistical shapes for S1 and S2 are quite similar, possibly due to the assumed definition of S1 and S2 being overlapping combinations of NEHRP sites B, C, and D. This points out a possible disadvantage of the NEHRP classification scheme in not considering profile depth as part of the criterion. Since profile information rapidly decreases at depths beyond about 70-100 ft, it is very difficult to accurately segregate profiles into depth bins. In general, the UBC shapes are consistent with the ground motion spectra and exceed the median statistical spectra from 0.1 to 10.0 sec.

To assess near fault effects, statistical spectra for site categories S1 and S2 were developed for sites at fault rupture distances of 0 to 10 km (no S3 or S4 sites were available). These results are shown in Figure 2 and suggest that the code shapes (particularly the lateral force requirements) do reasonably well on average for **M** in the range of 6.6 to 7.0 (Table 3). However, the S1 dynamic analysis shape is exceeded by the median ground motion at very short periods (< 0.08 sec).

## NEHRP Shapes

The comparisons of the statistical spectra for NEHRP sites B, C, D, and E to the code design shapes are shown in Figure 3. For the code shapes, both the provision requirements, which accommodate structural considerations, and the unbiased spectra are shown. Site A, very hard rock, is not shown as there are currently no sites in the database assigned this category. The spectrum in the NEHRP recommended provisions differs from the UBC spectrum in three major ways. First, the NEHRP reference Category (B) is narrower than the UBC reference category (S1). Second, the spectrum for Category B is anchored at two points instead of one. The short-period portion (periods shorter than approximately 0.5 sec) is anchored to parameter $A_a$ which is equivalent to the UBC zone factor Z and represents PGA. The intermediate and long-period portion is anchored to parameter $A_V$, which is related to peak ground velocity and is roughly equivalent to spectral acceleration at 1 sec for 5% damping. This concept dates back to the Newmark-Hall (CR-0098) shapes where portions of the design spectra were approximated as two straight lines (in log-log space) with amplitudes proportional to peak ground acceleration and velocity. Third, the NEHRP treatment of site conditions considers nonlinear soil response and affects all portions of the spectrum. The spectra for site categories other B are anchored to $F_aA_a$ and $F_VA_V$, respectively, where $F_a$ and $F_V$ are site coefficients that depend on soil category and on ground motion levels $A_a$ and $A_V$, respectively (see Table 2).

To compare statistical shapes to the NEHRP design spectral shapes for each category, it was assumed that $A_a = A_V$, which is the case for much of California in the current NEHRP maps. In addition, because the shapes depend upon rock motion amplitude ($A_a$ and $A_V$) through the Fa and Fv factors (Table 2), the data were separated into distance bins based on expected rock PGA ($A_a$) ranges. The expected rock PGA values were computed from a recently developed empirical attenuation relation which classifies rock as NEHRP sites B plus C (Abrahamson and Silva, 1996). In the comparisons shown in Figure 3, site B statistical shapes represents all distances (**M** > 6.3) since Fa and Fv are 1 for all $A_a$ and $A_V$. The mean magnitude is near **M** 7 with mean distance and PGA of 40 km and 0.15g respectively (Table 3). For sites C, D and E the highest rock PGA ($A_a$) ranges are shown which have sufficient data to constrain shapes ($\geq$ 4 earthquakes and $\geq$ 15 spectra). The C and D sites reflect records selected from distance ranges where the expected rock PGA ranges from about 0.25 to 0.35g for **M** $\geq$ 6.3. For these cases $A_a$ is taken as 0.20g ($A_V$ = 0.2g) to construct the NEHRP shapes. For site E, to provide enough data for stable statistical shapes, the expected rock PGA range combines the two lowest $A_a$ and $A_V$ bins, 0.1 and 0.2g and the expected rock PGA range used was 0.0 to 0.25g.

For sites B and C (Figure 3) the code provision shape exceeds the median statistical shape over the range of 0.1 to 10.0 sec. For site D, the code provision exceeds the statistical shape at long to intermediate periods ($\geq$ 0.3 sec) but is below the data at short periods (< 0.3 sec). For site E, the code provision exceeds the statistical shape but the margin is small at short periods ($\leq$ 0.5 sec).

To assess very high amplitude long period motions, Figure 3 (last plot) compares the NEHRP B shape to the two horizontal components (Professor W. Iwan, personal communication) of the Lucerne site from the **M** 7.3 1992 Landers earthquake. The site is at a fault distance of 1.1 km and has very high motions on the 260° component (fault normal). Interestingly the code provision shape does extremely well from 0.1 to 10.0 sec. However, it must be emphasized that, since the average recorded PGA at the site is about 0.8g, if the shape had been scaled to an $A_a$ = 0.4g, the long period absolute level of the 260° component would exceed the code provision by a factor of about 2. In this case, the code provision in absolute level would be close to the average horizontal component at long periods.

As an additional evaluation of the NEHRP provisions, the nonlinear Fa and Fv factors were estimated using the empirical attenuation relation of Abrahamson and Silva (1996). In this relation, both rock and soil sites are considered with rock generally reflecting NEHRP B and C (hard rock to very stiff soil) and soil NEHRP C and D (stiff soils). To accommodate this classification, the NEHRP factors (Fa and Fv) were adjusted to reflect ratios of site C + D to site B + C. The empirical Fa and Fv factors were computed as ratios (soil/rock) of empirical response spectra, averaged 0.1 to 0.5 sec for Fa and 0.4 to 2.0 sec for Fv, for increasing rock PGA values. The results are shown in Table 2 and reflect comparable values for the adjusted NEHRP factors and empirical factors. The empirical Fa are slightly lower and show a stronger nonlinear effect than the code provision. For Fv, the empirical show little nonlinear effects while the NEHRP Fv factors reflect about the same nonlinearity as the NEHRP Fa factors.

## PROBABILISTIC SEISMIC HAZARD RESULTS FOR SELECTED CITIES AND COMPARISONS TO CODE SPECTRA

The purpose of these calculations and the comparisons that follow is to investigate the consistency of the UBC and NEHRP code spectral shapes with the uniform-hazard spectra (UHS) calculated by probabilistic seismic-hazard analysis (PSHA). In particular, we wish to determine to what extent it is necessary to have two spectral anchor points (as in the NEHRP code), in order to accommodate differences in the shape of the UHS that arise from differences in the nature of the seismic threat at different California cities. For instance, one anticipates that the design spectrum for San Francisco should have more low-frequency energy than the one for Los Angeles because more of the hazard at San Francisco comes from large earthquakes in the San Andreas fault. In addition, we wish to investigate the effect of larger uncertainty in the attenuation functions for low frequencies, which tends to flatten the UHS. The interest here is more on the shape of the UHS, rather than on the absolute amplitudes of the code shapes anchor points.

The source characterizations used for the probabilistic seismic hazard calculations (source geometries, magnitude-recurrence models, maximum magnitude, and their corresponding uncertainties) come from recent published studies performed by others. The source characterization for Los Angeles come from Petersen et al. (1995) and that for San Francisco from Youngs et al. (1994). The source characterization for Sacramento is a combination of the Youngs et al. characterization and USGS (Hansen and Perkins, 1995) sources for the Sacramento region and the Sierra Nevada. Some minor modifications were introduced in these source characterizations, for the sake of simplicity.

The following three attenuation equations for soil were considered: Abrahamson and Silva (1996), Boore et al. (1994), and Sadigh (1988). These attenuation equations apply to UBC soil category S2 and to a mixture of NEHRP soil categories C and D (with a majority of D). These three attenuation equations reflect recent strong-motion data from California and include the effect of fault type (thrust or strike-slip). The Abrahamson and Silva equations also predict different amplitudes for the up-thrown and down-thrown blocks of a trust fault, but this effect was not considered in the calculations. These two effects are important only for Los Angeles area.

PSHA calculations at the three cities are performed for peak ground acceleration (PGA) and for spectral accelerations ($S_a$) at 0.04, 0.1, 0.2, 0.4, 1, 2, and 4 sec (5% damping). These results are used to construct mean UHSs for 90% non-exceedence probability in 50 years. This exceedence probability (equivalent to an annual exceedence probability of $2.1\times10^{-3}$ or a return period of 475 years) is the one implied in both the UBC and NEHRP codes. In addition, we determine the design earthquakes associated with these hazard results, for both PGA and $S_a$(1 sec), using McGuire's (1995) procedure. A design earthquake is defined in terms of a magnitude, a distance, and an attenuation-equation $\epsilon$ (number of standard deviations above the median). Results from the PSHA calculations are shown in Table 4, which also shows the ground-motion amplitudes in the UBC and NEHRP codes, as well as the interim USGS values (Frankel et al., 1995).

The magnitudes and distances that control seismic hazard at these three cities are quite different, as illustrated by the design earthquakes given in Table 4. This is confirmed by Figure 4, which shows the contributions of the various magnitudes, distances, and attenuation $\epsilon$s to the mean hazard for PGA and $S_a$ (1 Hz). Local faults (Hollywood, Elysian Park, and others) dominate the hazard in Los Angeles while the San Andreas fault dominates the hazard in San Francisco, as well as the 1 Hz hazard in Sacramento. The local area source dominates the PGA hazard in Sacramento. These differences have an effect on spectral shapes, as will be shown below

Figure 5 compares the mean UHSs obtained from the hazard calculations to the UBC (Category S2) and NEHRP (Categories C and D) spectra for the corresponding cities. The UBC spectrum works well in Los Angeles but under-predicts intermediate- and long-period amplitudes in San Francisco , where higher magnitudes contribute more to seismic hazard. The NEHRP spectrum does not do much better in San Francisco because the NEHRP maps have $A_a=A_V=0.4g$. A more appropriate value of $A_V$ for San Francisco, together with the two anchor points used by the NEHRP provisions, would provide an adequate fit. The comparisons for Sacramento suggest that the code values for this city are too high. Figure 6 shows comparisons in which the code shapes anchored to values of PGA and $S_a$(1 sec) obtained from hazard results for rock. The UBC spectral accelerations underestimate the UHS at all periods. The NEHRP spectrum for Category D (the category most representative of the soil attenuation equations) is consistent with the UHS at some periods and is conservative at other periods.

## CONCLUSIONS

Comparison of both UBC and NEHRP design spectral shapes with shapes computed from strong motion recordings at sites with appropriate subsurface conditions suggest that the code provide enveloping criterion including cases for sites within 10 km of the fault rupture. The only exception is for NEHRP site D at short periods ($\leq$ 0.3 sec) where the recorded motions exceed the design shapes.

Comparison of the code spectral shapes to the results from the probabilistic analysis (for 90% non-exceedence probability in 50 years) show that the fixed UBC shape underestimates ground motions by 25 to 50% in San Francisco and Sacramento, respectively, but is adequate for Los Angeles. The more flexible NEHRP shape allows for a better fit to the probabilistic results, but only if both anchoring values ($A_a$ and $A_V$) are specified appropriately.

## ACKNOWLEDGMENTS

This investigation was funded by the Strong Motion Instrumentation Program, California Division of Mines and Geology. The authors are grateful for this support and wish to thank all members of the SMIP project committee for their very useful comments on the progress of the project. They also wish to thank Mark Petersen, also of CDMG, for providing information on the seismic-source characterization for Los Angeles.

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Table 1
**PROFILE TYPES**

**UBC**

| Soil Profile Type | Description |
|---|---|
| $S_1$ | A soil profile with either: (1) rock of any characteristic, either shale-like or crystalline in nature, that has a shear wave velocity greater than 2,500 feet per second or (2) stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays. |
| $S_2$ | A soil profile with deep cohesionless or stiff clay conditions where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays. |
| $S_3$ | A soil profile containing 20 to 40 feet in thickness of soft- to medium-stiff clays with or without intervening layers of cohesionless soils. |
| $S_4$ | A soil profile characterized by a shear wave velocity of less than 500 feet per second containing more than 40 feet of soft clays or silts. |

**NEHRP Provisions**

| | |
|---|---|
| A | Hard rock with measured shear wave velocity, $\overline{V_S} > 5{,}000\ ft/\text{sec}$ |
| B | Rock with $2{,}500\ ft/\text{sec} < \overline{V_S} \leq 5{,}000\ ft/\text{sec}$ |
| C | Very dense soil and soft rock with $1{,}200\ ft/\text{sec} < \overline{V_S} <= 2{,}500\ ft/\text{sec}$ |
| D | Stiff soil with $600\ ft/\text{sec} \leq \overline{V_S} \leq 1{,}200\ ft/\text{sec}$ |
| E | A soil profile with $\overline{V_S} < 600\ ft/\text{sec}$ or any profile with more than 10 ft of soft clay |
| F | Soil requiring site-specific evaluations |

**Assumed UBC and NEHRP Profile Relationships**

| UBC Profile | NEHRP Profile |
|---|---|
| S1 | B and C |
| S2 | C and D |
| S3 | E |

Table 2
**Fa AND Fv VALUES**

| Soil Profile Type | Fa For Shaking Intensity Levels | | | | |
|---|---|---|---|---|---|
| | $A_a \leq 0.1$ | $A_a = 0.2$ | $A_a = 0.3$ | $A_a = 0.4$ | $A_a \geq 0.50$[a] |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.2 | 1.2 | 1.1 | 1.0 | 1.0 |
| D | 1.6 | 1.4 | 1.2 | 1.1 | 1.0 |
| E | 2.5 | 1.7 | 1.2 | 0.9 | b |
| F | b | b | b | b | b |
| **Soil Profile Type** | **Fv For Shaking Intensity Levels** | | | | |
| | $A_V \leq 0.1$ | $A_V = 0.2$ | $A_V = 0.3$ | $A_V = 0.4$ | $A_V \geq 0.50$[a] |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.7 | 1.6 | 1.5 | 1.4 | 1.3 |
| D | 2.4 | 2.0 | 1.8 | 1.6 | 1.5 |
| E | 3.5 | 3.2 | 2.8 | 2.4 | b |
| F | b | b | b | b | b |

Note: Use straight line interpolation for intermediate values of $A_a$ and $A_V$
[a] Values for $A_a$, $A_V > 0.4$ are applicable to the provisions for seismically isolated and certain other structure
[b] Site specific geotechnical investigation and dynamic site response analyses shall be performed.

Empirical Fa and Fv Values

| | Adjusted NEHRP* | | Empirical** | |
|---|---|---|---|---|
| $A_a = A_V$ | $F_a$ | $F_V$ | $F_a$ | $F_V$ |
| 0.1 | 1.3 | 1.6 | 1.1 | 1.5 |
| 0.2 | 1.2 | 1.4 | 1.0 | 1.4 |
| 0.3 | 1.1 | 1.3 | 0.9 | 1.4 |
| 0.4 | 1.1 | 1.2 | 0.9 | 1.4 |
| 0.5 | 1.1 | 1.2 | 0.8 | 1.4 |

*For site profile C + D relative to profile B + C
**Based on empirical attenuation relation (Abrahamson and Silva, 1996). $F_a$ averaged from 0.1 - 0.5 sec, $F_V$ averaged from 0.4 - 2.0 sec.

Table 3
**MEAN MAGNITUDES AND DISTANCES FOR THE RESPONSE SPECTRAL SHAPES**

| Site | Number of Records | Number of earthquakes | $\overline{M}$ | $\overline{D}$ | $\overline{PGA}$ (g) |
|---|---|---|---|---|---|
| S1 | 154 | 8 | 6.88 | 35 | 0.16 |
| S2 | 324 | 12 | 6.75 | 33 | 0.17 |
| S3 | 100 | 4 | 6.72 | 50 | 0.10 |
| S1(10)* | 12 | 4 | 7.00 | 6 | 0.72 |
| S2(10)** | 40 | 4 | 6.62 | 5 | 0.41 |
| B | 38 | 5 | 6.95 | 40 | 0.15 |
| C | 116 | 6 | 6.86 | 33 | 0.16 |
| D | 208 | 9 | 6.68 | 33 | 0.18 |
| E | 100 | 4 | 6.72 | 49 | 0.10 |

*SMART 1 array at Lotung classified as E.
**Data restricted to closest fault rupture distance ≤ 10 km.

Table 4
**HAZARD-ANALYSIS RESULTS AND CODE ACCELERATION VALUES**

| City | UBC Z(g) | NEHRP $A_a$(g) $A_v$(g) | USGS Interim hazard maps[1,2] PGA(g) | Probabilistic Seismic Hazard Analysis[1] (UBC S2, NEHRP C-D soil) | |
|---|---|---|---|---|---|
| | | | | PGA(g) $S_a$(1Hz) (g) | Design Earthquake |
| Los Angeles | 0.4 | 0.4<br>0.4 | 0.5 | 0.45<br>0.55 | M 5.9 at 9 km, $\epsilon$=1.5<br>M 6.5 at 10 km, $\epsilon$=0.9 |
| San Francisco | 0.4 | 0.4<br>0.4 | 0.7 | 0.44<br>0.73 | M 7.7 at 14 km, $\epsilon$=0.60<br>M 7.7 at 14 km, $\epsilon$=0.35 |
| Sacramento | 0.3 | 0.2<br>0.3 | 0.12 | 0.15<br>0.22 | M 5.1 at 15 km, $\epsilon$=1.0<br>M 7.8 at 130 km, $\epsilon$=1.0 |

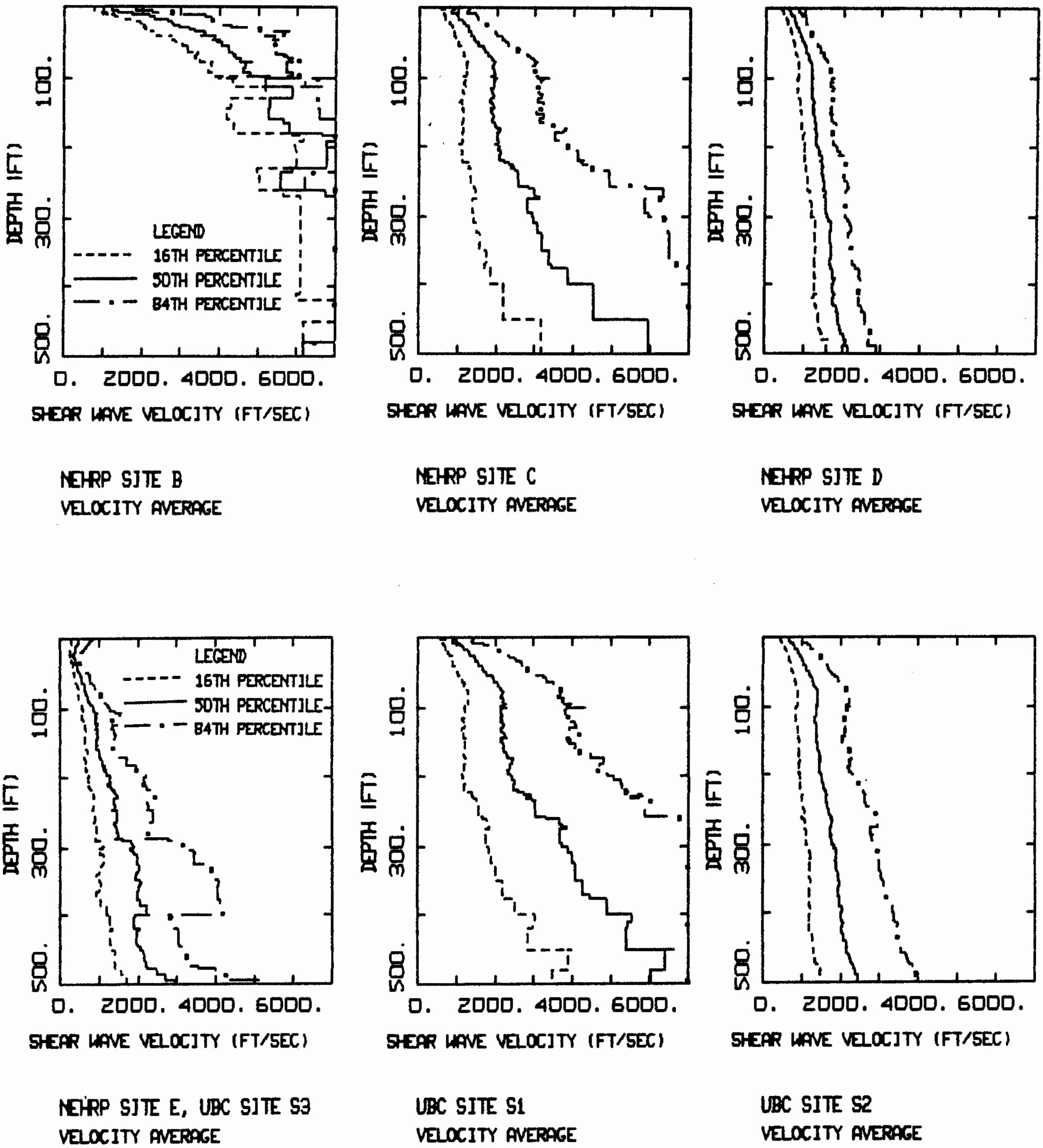



Figure 1. Representative median and ± 1 sigma shear-wave velocity profiles for the NEHRP provisions and UBC site classes (Table 1) (NEHRP site class A, very hard rock, is currently not represented in the profile database).

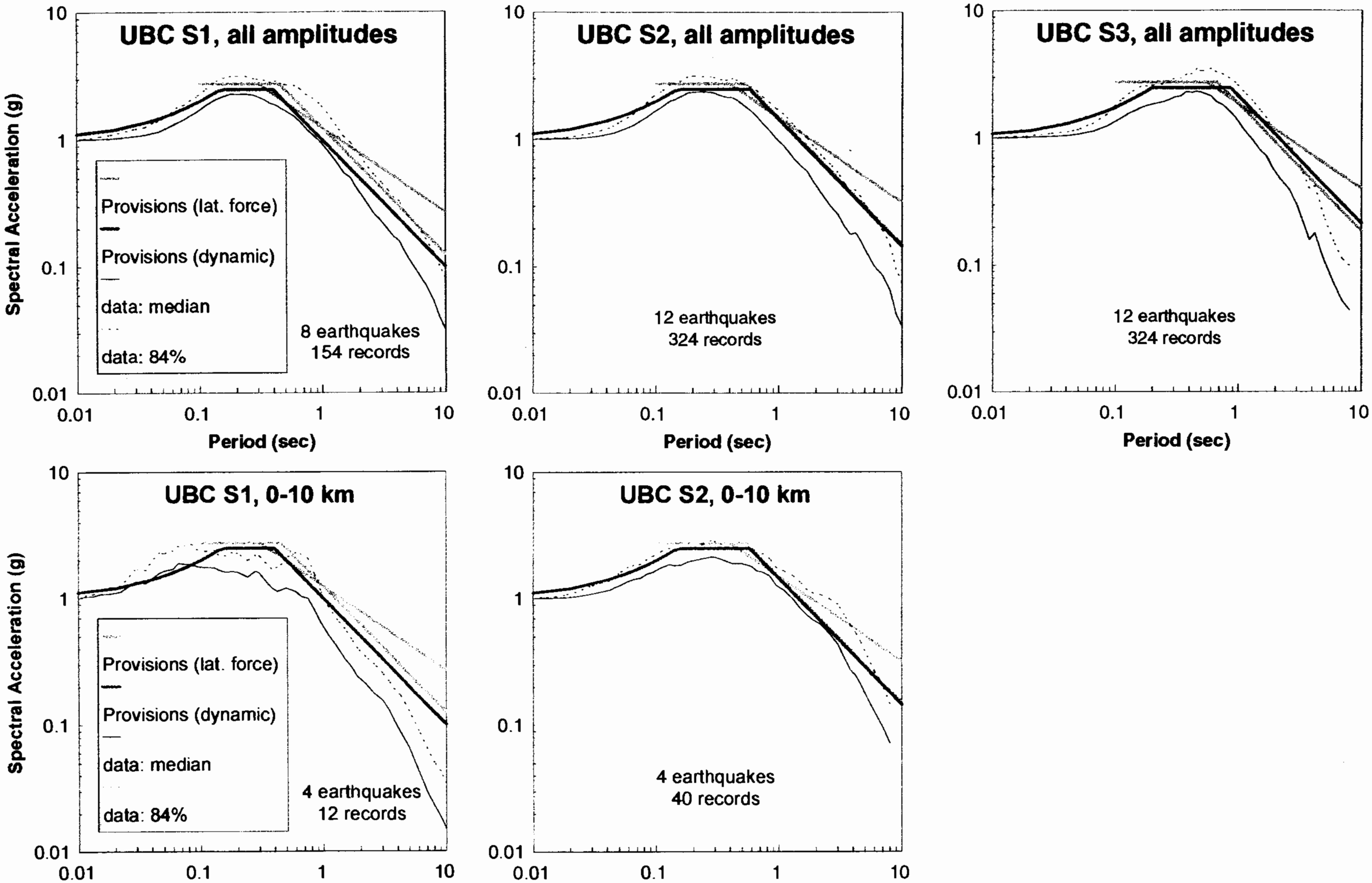



Figure 2. Comparison of UBC code shapes with statistical spectra computed from data recorded at sites classified as S1, S2, and S3 (Table 1). Bottom two figures (S1 and S2) are for sites at fault rupture distances from 0 to 10 km. Site categories S4 and S3 (0 to 10 km) have too few data to provide stable statistical spectra.

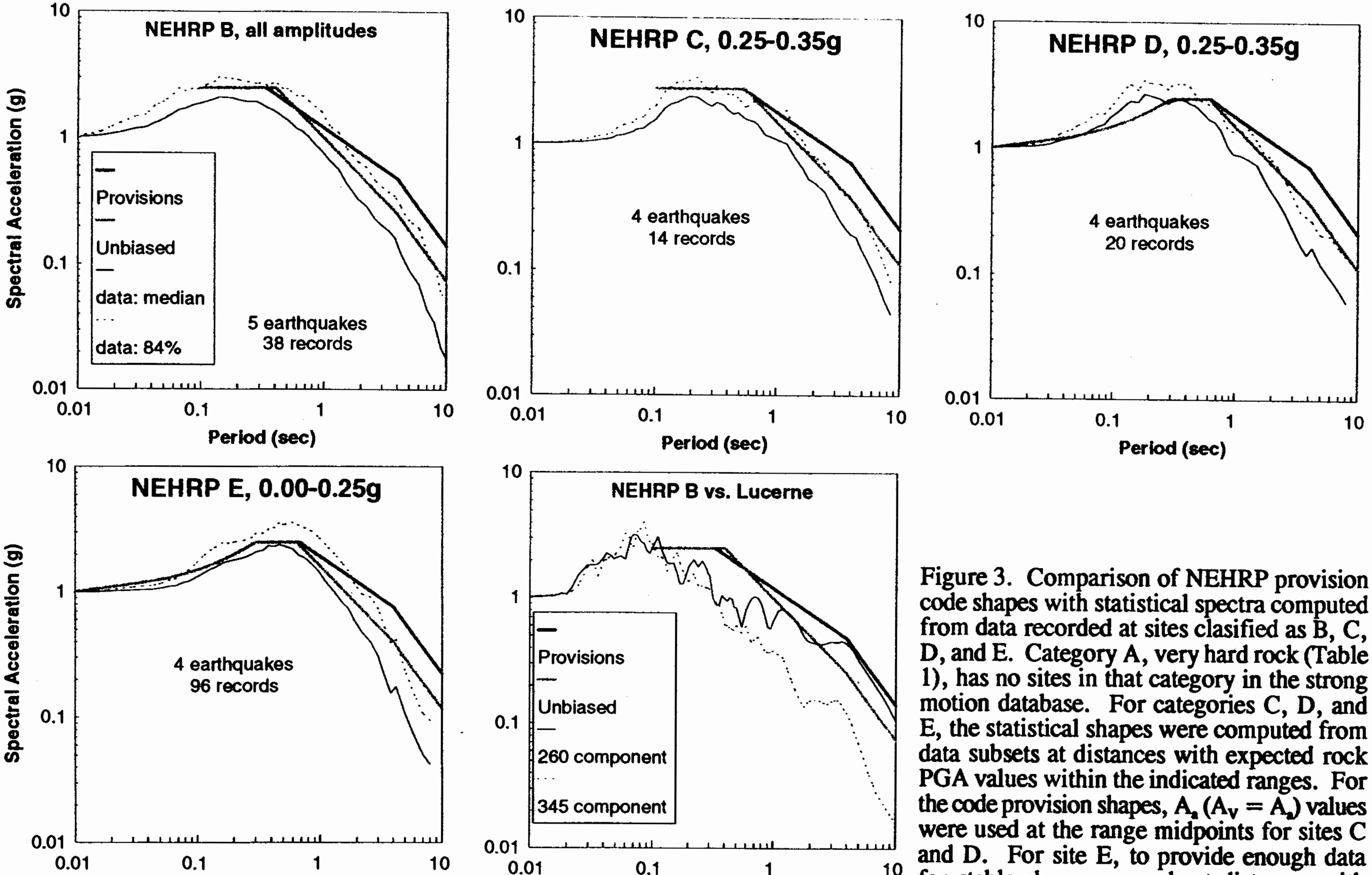



Figure 3. Comparison of NEHRP provision code shapes with statistical spectra computed from data recorded at sites clasified as B, C, D, and E. Category A, very hard rock (Table 1), has no sites in that category in the strong motion database. For categories C, D, and E, the statistical shapes were computed from data subsets at distances with expected rock PGA values within the indicated ranges. For the code provision shapes, $A_a$ ($A_V = A_a$) values were used at the range midpoints for sites C and D. For site E, to provide enough data for stable shapes, records at distances with expected rock PGA values in the range of 0.05 to 0.25g were used along with $A_a = A_V = 0.15g$. The ranges were selected to represent the highest motions with sufficient data to constrain the shapes. For site B, since the Fa and Fv values are independent of rock motion level ($A_a$ and $A_V$, Table 2), all the site B data (Table 3) were used for the shapes. The final plot compares the Lucerne records for the M 7.3 1992 Landers earthquake (fault distance 1.1 km) to the NEHRP site B shape.

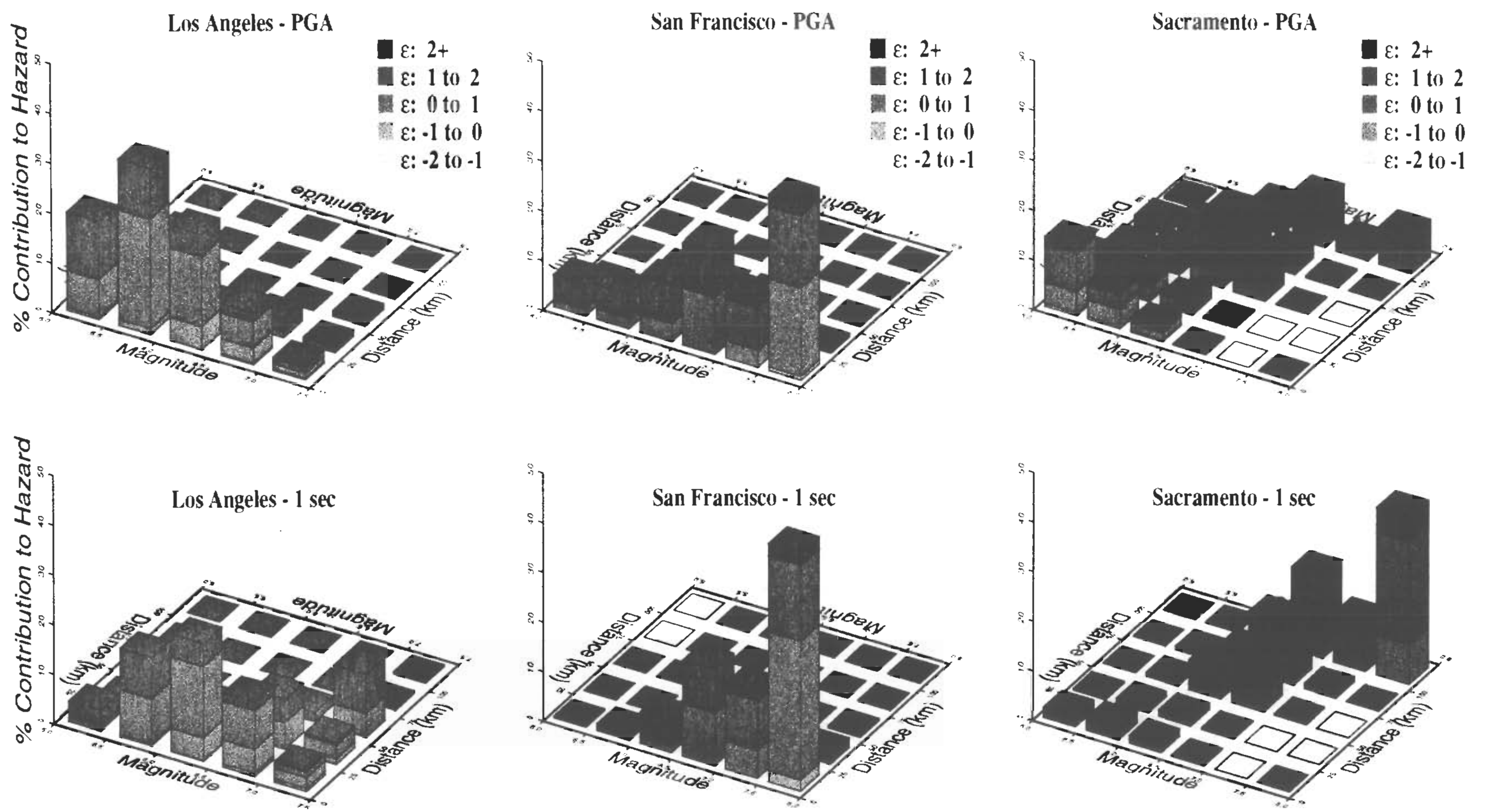



Figure 4. De-aggregation of the mean exceedence probability by magnitude, distance, and ground-motion $\epsilon$ for the three California cities considered in this study. Non-exceedence probability: 90% in 50 years.

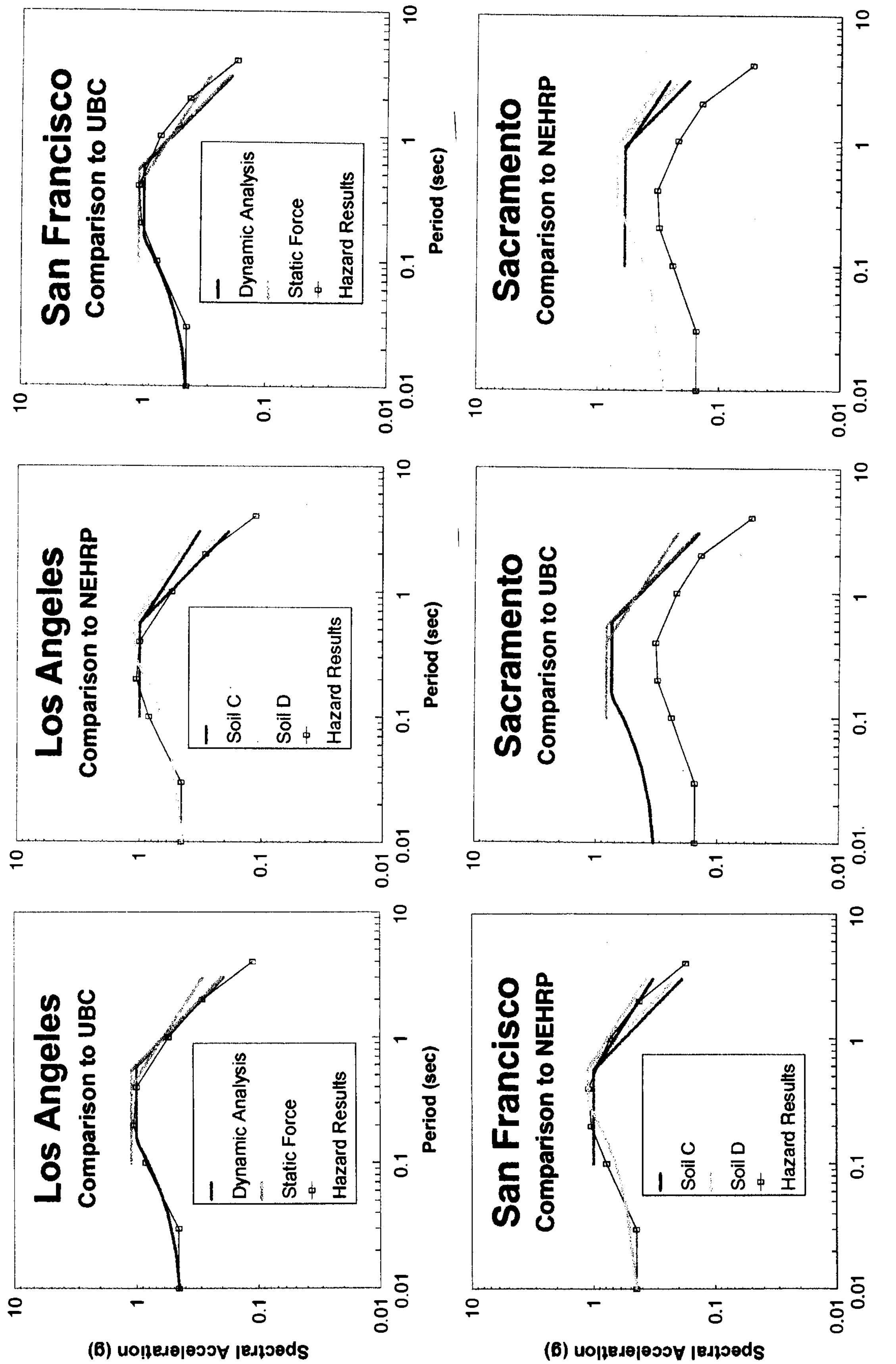



Figure 5. Comparison of UBC and NEHRP spectra for three cities to the corresponding mean uniform-hazard spectra for 90% non-exceedence probability in 50 years.

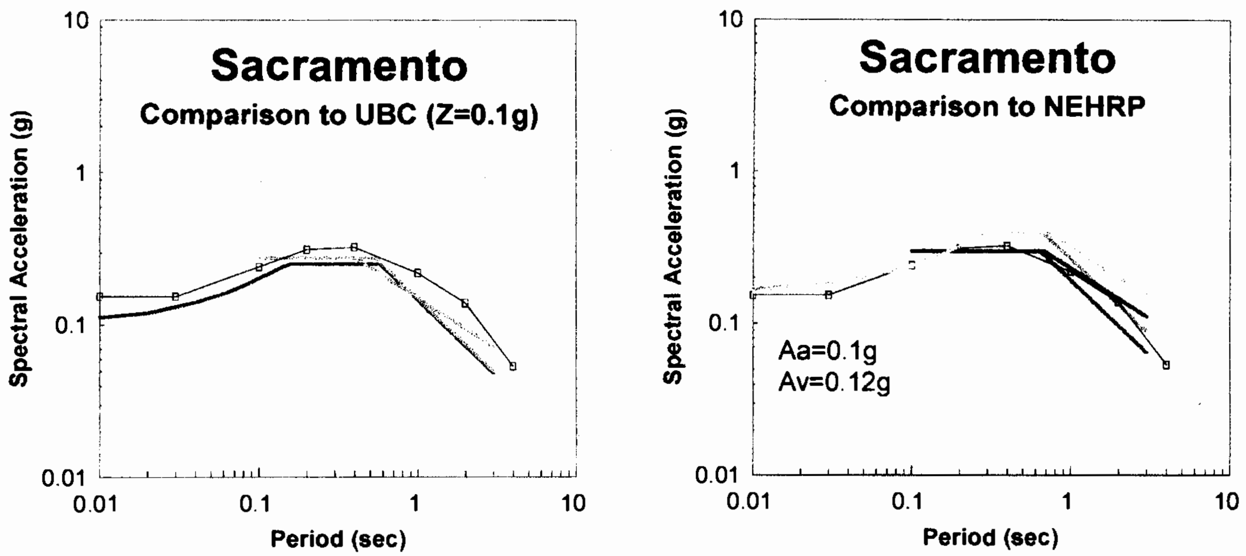



Figure 6. Comparison of UBC and NEHRP spectra for Sacramento to the corresponding mean uniform-hazard spectra for 90% non-exceedence probability in 50 years. Code shapes are anchored to rock seismic-hazard results.

# RECOVERY OF RECORDS FROM CODE-REQUIRED ACCELEROGRAPHS AFTER THE NORTHRIDGE EARTHQUAKE

A. Shakal, C. Petersen, R. Darragh, M. Huang
California Division of Mines and Geology
Department of Conservation, Sacramento, CA

R. Nigbor and K. Madura
Agbabian Associates, Pasadena, CA

## ABSTRACT

The Northridge earthquake provided the first case since the 1971 San Fernando earthquake of effective recovery, processing, use and analysis of data from accelerographs installed in buildings to meet local code requirements. A large number of records were recovered from these accelerographs in an effort funded by the California Strong Motion Instrumentation Program and the National Science Foundation, with Agbabian Associates coordinating and managing field recovery. The records were valuable to assist post-earthquake structural evaluation and form an important complement to data from regular networks. Significant aspects of the code data recovery, processing and analysis are reviewed. Lessons learned from this experience with code instruments are reviewed to improve the success in future events. Recommendations include increasing the documentation of the location, orientation, and maintenance of code-required instruments, and shifting from film to digital recorders for new installations. Also, a goal of increased adoption of the Uniform Building Code (UBC) instrumentation code requirement by cities and counties is discussed. CSMIP will assist city and county code-instrument programs by providing technical and monitoring expertise.

## INTRODUCTION

Recording the response of buildings during strong earthquake shaking is a key element in improving seismic design. There are two basic approaches used in instrumenting structures to record strong motion. One is the extensive-instrumentation approach used by the California Strong Motion Instrumentation Program (CSMIP) in the California Department of Conservation's Division of Mines and Geology (CDMG) in its normal instrumentation. In this approach, 10 to 20 or more single-direction or uniaxial acceleration sensors are strategically located in a building and their signals are cabled to a centrally-located recorder. CSMIP recorded the motions of nearly 60 buildings with this type of instrumentation during the Northridge earthquake (Shakal et al., 1994).

The second instrumentation approach is the much less extensive instrumentation required by the building code in many cities. In this approach, up to three accelerographs, each with 3 sensors, are located in a building at the top, base and approximate mid-height. Code-required instrumentation was the most common instrumentation in place at the time of the

1971 San Fernando earthquake. The experience with that earthquake made it clear that there were several needs not being met:

a) Much more instrumentation was needed, outside and inside buildings, and an additional program was needed to accomplish that;
b) The three points of recording in a building (bottom, middle and top) were not adequate to isolate torsion and other effects on drift; and
c) Centralized recording of the motion, with common timing, was critical to understanding details of the response of the building during earthquake shaking.

One result of the 1971 San Fernando earthquake experience was that CSMIP, a new state-wide program, was initiated, funded by a small fee on building permits. Another result was that new instruments were developed which had separable uniaxial sensors, and recorders were developed which recorded many channels side by side on the same medium, with common timing. The CSMIP program of extensively instrumenting a limited number of structures has been moving forward since then, and the large number of high-quality records obtained in Northridge is an example of the progress (Shakal et al., 1994). Nearly 200 structures, including 135 buildings, 20 dams and 35 bridges have been instrumented. In parallel with these extensive-instrumentation efforts, the code requirement for building accelerographs has continued in several cities.

## CODE ACCELEROGRAPH REQUIREMENTS

The requirements for building accelerographs adopted by many cities follows the requirements in the Uniform Building Code, sometimes in a modified form. The requirement, as it appears in the Uniform Building Code (International Conference of Building Officials, 1994), Chapter 16 Appendix, is:

> Section 1646 - General. In Seismic Zones Nos. 3 and 4 every building over six stories in height with an aggregate floor area of 60,000 square feet or more, and every building over 10 stories in height regardless of floor area, shall be provided with not less than three approved recording accelerographs.
>
> Section 1647 - Location. The instruments shall be located in the basement, midportion, and near the top of the building. Each instrument shall be located so that access is maintained at all times and is unobstructed by room contents.
>
> Section 1648 - Maintenance. Maintenance and service of the instruments shall be provided by the owner of the building, subject to the approval of the building official. Data produced by the instruments shall be made available to the building official on request.

The City of Los Angeles modified this requirement and in recent years their city building code only requires a single instrument in buildings, at the roof level, but the City adopted a more stringent requirement to improve the monitoring of instrument maintenance. At the time of the Northridge earthquake there were nearly 500 buildings with code-required instruments in the Los Angeles area. Significant response data were recorded at many of these buildings during the earthquake.

Not all cities and counties require accelerographs in the large buildings in their jurisdiction. One CSMIP goal is to encourage cities to include the requirement for instruments in their code. This can be done as simply as by adopting the Appendix section of the UBC, or by developing a parallel requirement. CSMIP is available to assist cities and counties on technical aspects of instrumentation.

Another CSMIP goal is to help cities and counties deal with the instruments and with data from the instruments after a significant earthquake. Earthquakes are rare, and regardless of the good intentions of a city or county, institutional memory dims between earthquakes regarding what should be done with records after an earthquake and how to get early information from them. As a state-wide strong motion program with a focussed mission to extensively instrument sites and structures, CSMIP works with the instruments and data on a continuous basis, and can effectively help cities and counties administer and monitor the code-instrument program in their jurisdiction. CSMIP can more effectively help after an earthquake if it works with the city at some level during the period between the events.

This paper highlights key aspects of the data recovered from code-required instruments after the Northridge earthquake. Following the Northridge earthquake CSMIP worked extensively with the City of Los Angeles and other cities. This paper also discusses lessons learned about how the privately-maintained code-required instruments can more effectively contribute data and information important for the building owner and the city or county after future earthquakes.

## THE CITY OF LOS ANGELES EXPERIENCE

The City of Los Angeles has a long history of addressing the need for instrumentation in buildings. When the state-wide CSMIP program began in 1972, the City already had a large enough program that it remained in place, separate from the new State program. After a few years Los Angeles decided that it would be more effective to join the State program, which occurred in the early 1980s.

At that time, the State and the City developed an agreement in which CDMG agreed to recover significant earthquake records from the code accelerographs that the City required in buildings. Since that time, Northridge is the first event that caused strong shaking throughout the Los Angeles area, and thus it was the first event which exercised the agreement. There had been earlier earthquakes which generated some strong motion records, most notably the 1987 Whittier earthquake, during which the shaking was recorded at a small number of buildings. The attempt to accomplish the provisions of the agreement after the Whittier event was an early lesson which influenced the approach CSMIP used for the Northridge earthquake.

Figure 1 illustrates, for the City of Los Angeles, the relationship between the City building department, the building owner, and the State. The City requires the accelerographs to be installed, and requires evidence of periodic maintenance by a company certified by the City. Periodic maintenance records are provided by the company to the City. If a significant earthquake occurs then CDMG is to recover the records and provide copies to the City and the building owners.

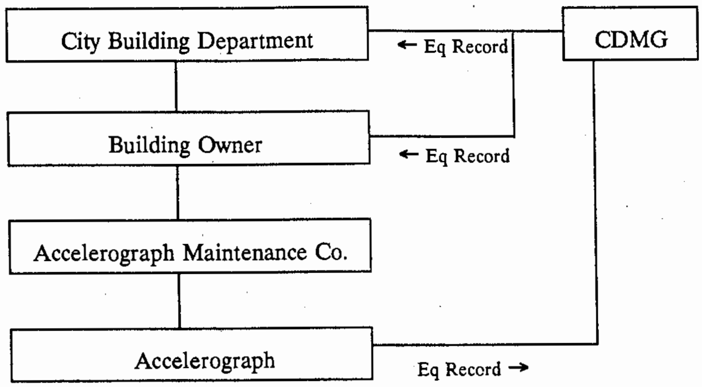



Figure 1. Relationship between the City of Los Angeles, building owner and CDMG for recovery of records from code-required accelerographs after significant earthquakes.

The experience of the Whittier earthquake suggested a modified approach, which was used in the Northridge earthquake. The key difference was to have the Accelerograph Maintenance vendor, already certified by the City, and with full knowledge of the locations and parameters of the instruments in the buildings because of their active contract with the building owner, to be the agent to do record recovery for the State. This leads to a slightly modified model, shown in Figure 2.

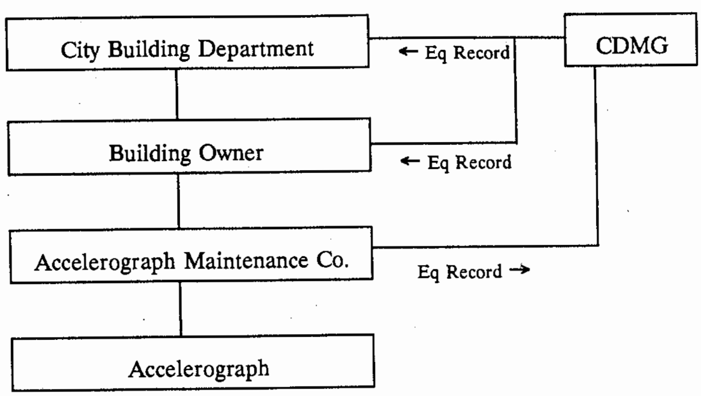



Figure 2. Relationship between the City of Los Angeles, the building owner, the accelerograph maintenance companies and CDMG for recovery of records from code-required instruments after the Northridge earthquake.

This model is much more effective. Of course, significant funding may be required for the State to contract for record recovery by the Maintenance Companies. After the Northridge earthquake, the National Science Foundation (NSF) shared this cost. To make the record recovery as effective as possible, NSF and CDMG jointly funded record recovery, in two separate contracts with Agbabian Associates to direct the activities of the individual Maintenance Companies, and to collect instrument and building information. This relationship is illustrated in Figure 3.

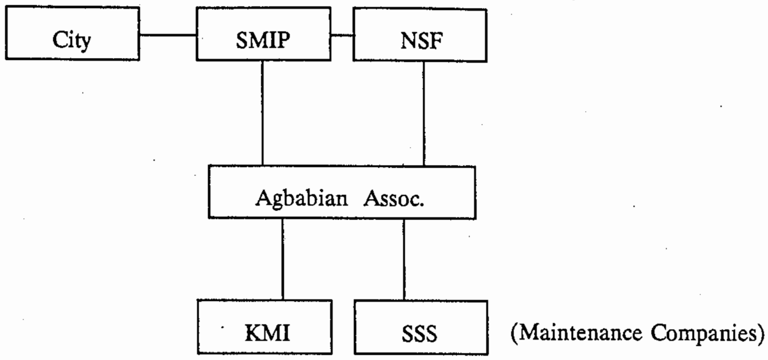



Figure 3. Relationships between The City of Los Angeles, CDMG, NSF, Agbabian Assoc. and accelerograph maintenance companies for the recovery of the Northridge accelerograms.

This framework was put into place rapidly after the Northridge earthquake, with effective and rapid work by Agbabian Associates and the Maintenance Companies, Kinemetrics, Inc. (KMI) and Seismic System Service (SSS), both certified by the City and having contracts with the building owners. A small amount of additional data was recovered by CSMIP staff working directly with building owners. With procedures in place from the Northridge experience, the approach shown in Figure 2 is a good model for how CSMIP can help cities recover data after significant earthquakes in the future.

## CODE-RECORDS FROM THE NORTHRIDGE EARTHQUAKE

The Northridge earthquake yielded a large set of strong-motion recordings, including records from more than 250 ground-response stations, more than 400 buildings and 50 other structures. CSMIP recovered records from 116 ground-response stations and 77 extensively-instrumented structures, including 57 buildings (Shakal et al., 1994). The U.S. Geological Survey's National Strong Motion Instrumentation Project recovered records from more than 37 buildings (some with limited instrumentation), and more than 60 ground-response sites (Porcella et al., 1994). The University of Southern California's Los Angeles Strong Motion Accelerograph Network recovered records from 71 ground-response stations (Trifunac et al., 1994). Records were also obtained from 7 facilities of the Los Angeles Department of Water and Power (Lindvall Richter Benuska, 1994).

An even larger number of records were recovered from the privately-owned and maintained code-accelerographs. A total of 500 records from nearly 270 buildings in the Los

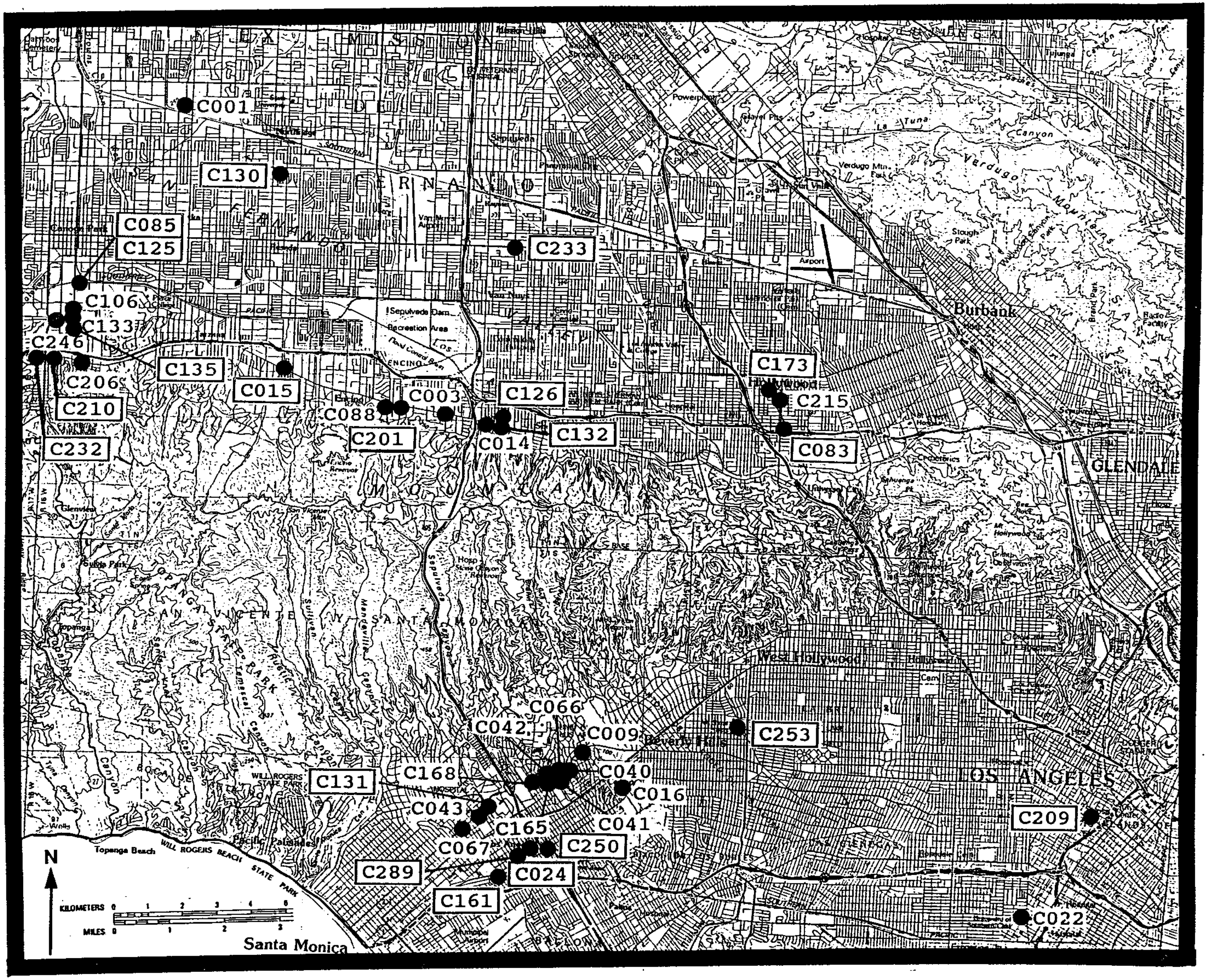



Figure 4. Index map showing 40 Code stations that recorded the Northridge earthquake for which data have been processed. The 20 stations in the first release are identified by a 4-character code. The 20 stations in the second release are identified by a 4-character code in a box. The code is cross-referenced to the station name in Table 1.

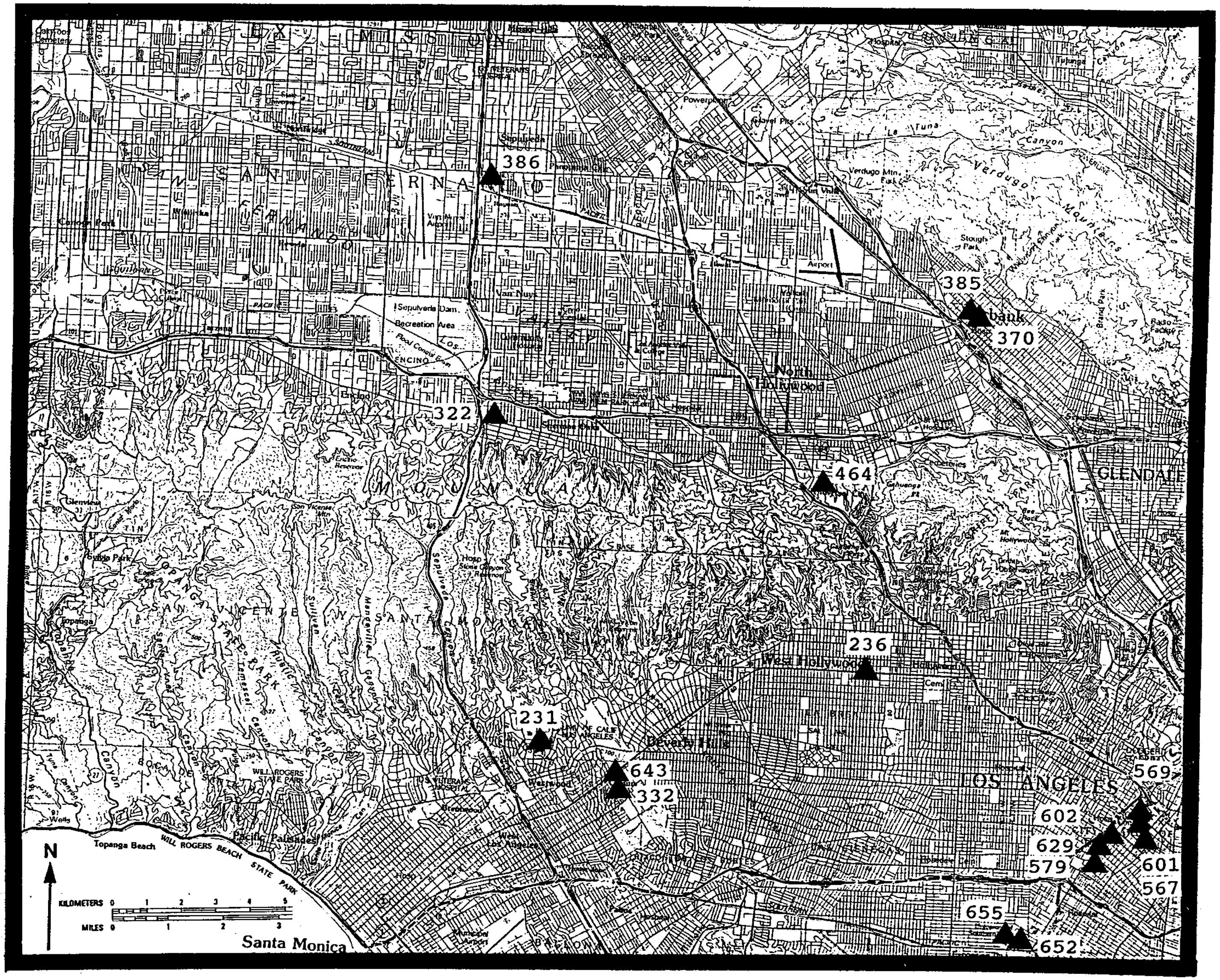



Figure 5. Index map showing the locations of the 17 CSMIP extensively-instrumented buildings in the San Fernando and Los Angeles areas. The stations are identified by a 3-character code that is cross-referenced to the station name in Table 2. The Northridge earthquake data for these buildings have been processed and released. Processed data are available from a total of 38 CSMIP buildings.

Angeles area were recovered and archived by CSMIP with recovery in the field by KMI and SSS under the coordination of Agbabian Associates. That effort is described in Nigbor and Madura (1996). A collected set of the records and locations of the buildings are presented in reports being prepared by CDMG and Agbabian Associates. The first records were digitized and processed rapidly by CSMIP at the request of the City of Los Angeles so that they could be studied by the Committees the City set up to study the steel-frame building problems after the earthquake. The first group of 20 buildings, provided in this manner during June though September, 1994 were collected to form the first code data report, released on September 20 (Darragh et al., 1994a). Additional records were processed according to a generalized priority of locations and building type, with additional specific requests from the City. The second release, with another 20 buildings, occurred in April, 1995 (Darragh et al., 1995). This release brought to 66 the number of code building records digitized and processed. Of course, during this time records from extensively-instrumented buildings of the CSMIP network were also being digitized and processed. The records from a total of 125 of these network stations, including 38 buildings, 79 freefield stations, and 8 other structures have been processed and released (Darragh et al., 1994b).

The 40 buildings included in the first two code-data releases are listed in Table 1. The buildings range in height from 7 stories to 36 stories. Both steel and concrete buildings are included. Only 1/3 of the buildings have more than one accelerograph. Some of the steel frame buildings were damaged, and four are discussed more in Huang et al. (1996). The locations of the buildings from which the code records have been processed and released are shown in Figure 4. Some records from some of the most important buildings could not be digitized because the accelerograph malfunctioned or the records were not readable.

The 38 CSMIP extensively-instrumented buildings for which the records have been processed and digitized are listed in Table 2, for comparison. The buildings range in height from 1 story to 54 stories, and include low-rise buildings not required to be instrumented under the code. Low-rise buildings were also damaged during the Northridge earthquake. Steel frame, concrete frame, concrete shear wall, base-isolated, and other building types are included. The damaged 7-story hotel located in Van Nuys is discussed in Huang et al. (1996). The locations of the 17 CSMIP buildings in the San Fernando and Los Angeles areas are shown in Figure 5.

Figure 6 shows the accelerogram from one of the code buildings as an example. The record is from the roof level of an 17-story building, Canoga Ave #2, in the western San Fernando valley. It shows that higher mode response is dominant in the acceleration record. After the record was digitized and processed, the first mode response with a period over 4 seconds was clear in the displacement record. The record also shows high frequency spikes in the records which may be associated with local damage or related effects near the recorder. The base-level motion is not known because there was only a roof instrument.

Examples of spectra are shown in Figure 7. This figure shows the spectral acceleration (5% damping) at the base of four buildings located in the epicentral area, Woodland Hills, Sherman Oaks and North Hollywood. The spectral acceleration for these areas is similar to or smaller than the UBC spectra. These spectra have been used in the Seismic Safety Commission Case Studies Project (SSC, 1996) as discussed in Turner (1996).

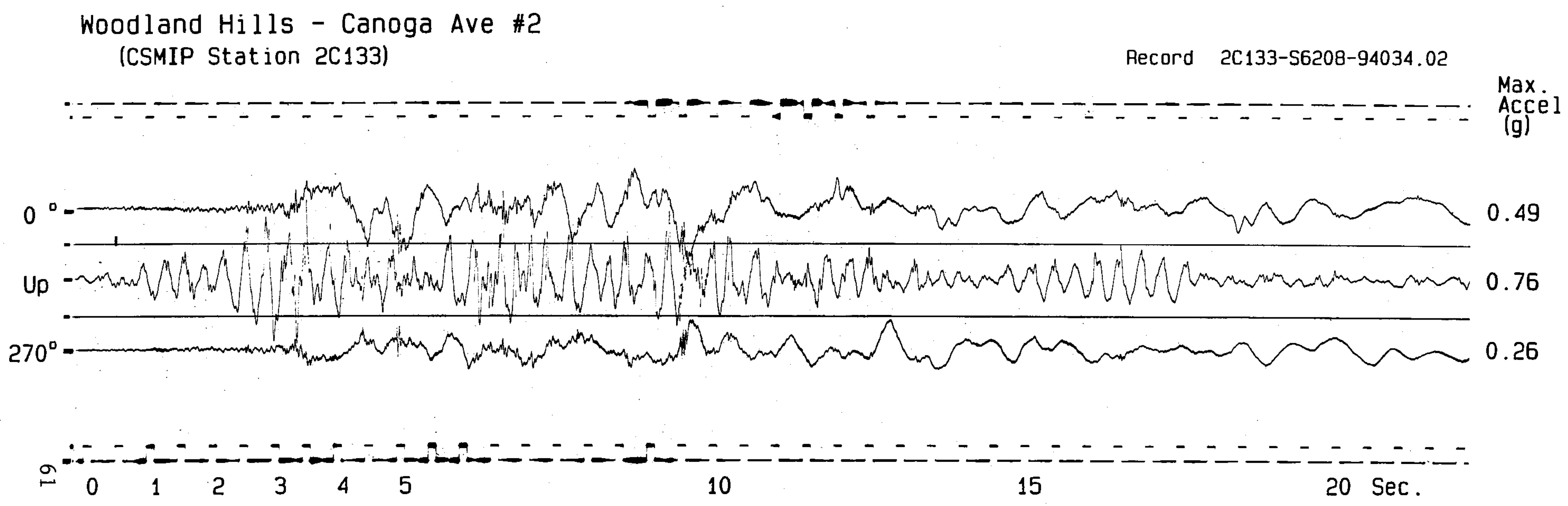
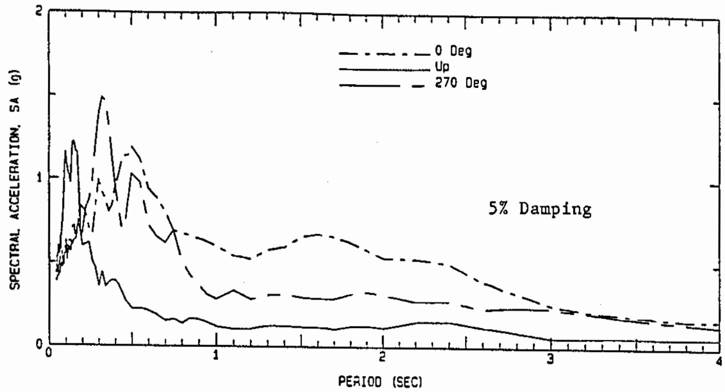
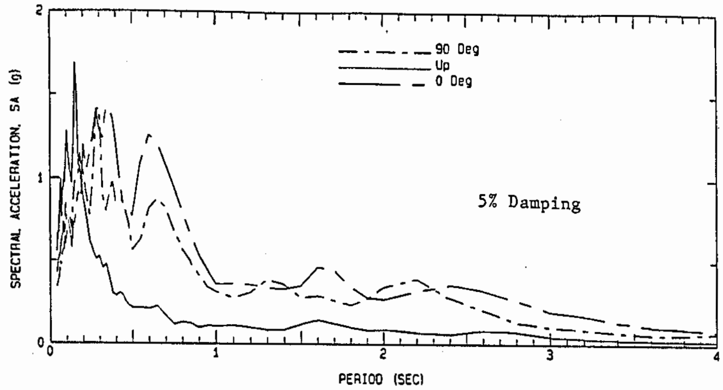



Figure 6. Acceleration record from a code instrument at the roof level of a 17-story building in Woodland Hills.



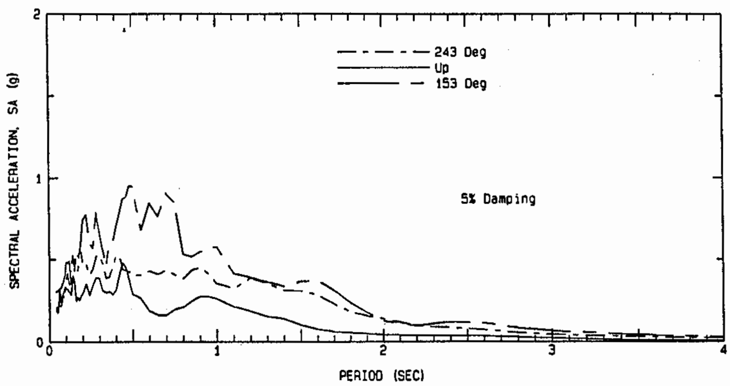

**Epicentral Area**

Northridge - Roscoe Blvd (CSMIP Sta. C130)
Ground Floor of 7-story Bldg.



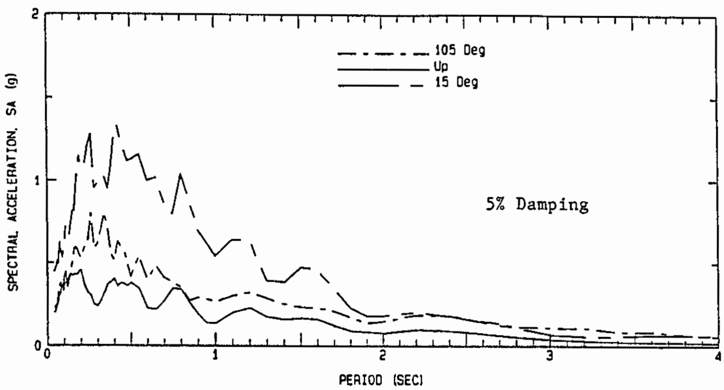

**Woodland Hills/Canoga Park**

Woodland Hills - Oxnard Blvd. #4 (CSMIP Sta. C246)
Basement of 12-story Bldg.




**North Hollywood**

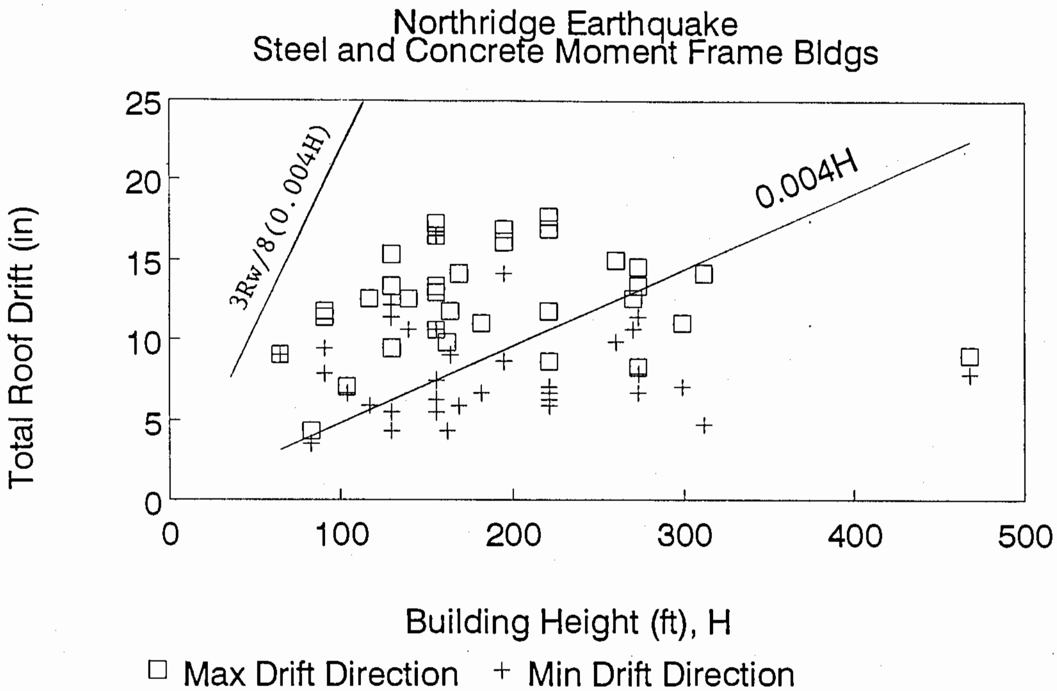
North Hollywood - Lankershim Blvd. #2 (CSMIP Sta. C083)
Basement of 8-story Bldg.



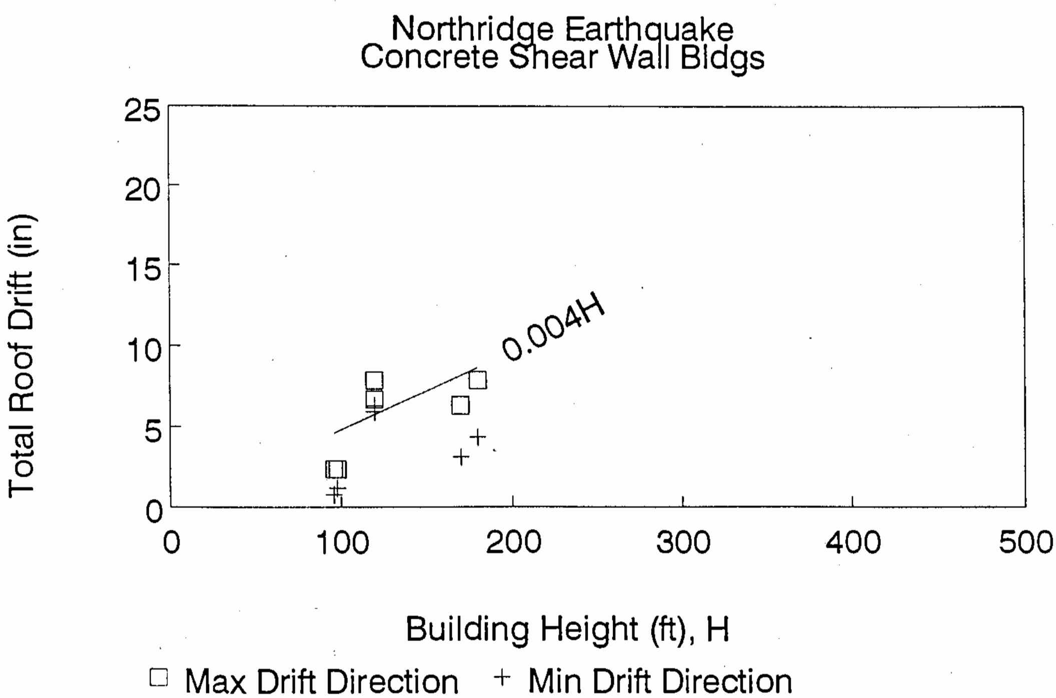

**Sherman Oaks**

**Sherman Oaks - 13-story Commercial Bldg. (CSMIP Sta. 322)**
**Basement of 13-story Bldg.**

Figure 7. Spectral acceleration (5% damped, all three components) from the accelerograph located at the base of four buildings, one located near the epicenter and three on the south side of San Fernando Valley. Records from the top three are from code-required accelerographs.




(a)




(b)

Figure 8. Total drifts at roof versus building height for (a) moment-frame buildings, and (b) concrete shear wall buildings in the Northridge earthquake. The line corresponding to the code drift limit of 0.004 H is also shown.

The records from code-accelerographs are important and have already been used in several important applications:

a) The records have been provided to the building owners who pass them to their structural engineers for post-earthquake evaluation of the building.
b) Data from 4 damaged steel buildings were included in the SAC (1995) project to evaluate analytical methods for identifying critical areas of damage in buildings.
c) The base-level records complement the smaller number of records from stations of the regular networks, and thus play an important role in providing information about the shaking in the building's vicinity. Although the base records will have some soil-structure interaction effects, they still provide very important data when there are no nearby stations of the regular networks.

Some studies focused on Northridge building damage include SAC (1995), SSC (1996), Turner (1996), Kariotis (1996) and Huang et al. (1996).

To get full value from a code-required accelerograph, the data should also be available rapidly enough to be useful in early post-earthquake response. The data can help the building owner and city to evaluate rapidly the response of the building, besides its usual application in longer term studies and computer modelling.

As an example of the value of strong motion data in the assessment of a building's response after an earthquake, Figure 8 shows a plot of total drift at the roof level calculated or estimated from the records for 23 moment-frame buildings of different heights (Huang and Shakal, 1995). The drift limit in the building code is about 0.004 H for these buildings. Total drift in many of the buildings was over this value. Since the designs of moment-frame buildings were mostly controlled by drift, structural members in some of these moment-frame buildings with large drifts may have yielded during the earthquake. If a building owner or response official knew, for example, that the total drift in a building was less than the code drift limit they may concern themselves first with other buildings, unless there are other symptoms of damage. Of course, to provide an accurate measurement of total drift, or allow the determination of base shear, there must be an instrument at the base of the building as well as at the roof level, rather than a single instrument at the roof.

## RAPID DATA RECOVERY

For a rapid comparison to a code limit or other design parameter, the data needs to be available quickly after an earthquake. CSMIP has recently developed a near-real-time data recovery and processing facility that has important benefits in this regard. The system automatically recovers data from field instruments, using dial-up telephone lines, after they have been triggered by earthquake shaking. Once the data have been transmitted to a central bank of computers in Sacramento, the record is processed to yield acceleration, velocity and displacement time series, and response and Fourier spectra. The data can then be transmitted to people previously identified as recipients.

An example is provided by the Northridge aftershock of June 26, 1995 which occurred near Castaic, north of the Northridge mainshock. The magnitude 5 event occurred at 1:40 in the morning. Within 3 minutes the first CSMIP station had transmitted the recorded event to Sacramento, the record was processed, and designated people were notified. Within 30 minutes after the event, the records from all 13 near-real-time stations that recorded the event had been transmitted to Sacramento and the data had been processed. The data from the event and other aspects are documented in a short report released on the day of the event (CSMIP, 1995).

This capability can be applied to near-real-time recovery from any modern digital accelerograph with a modem and telephone communications. The addition of this capability greatly increases the value of the data in the early minutes, hours and days after the earthquake. All the code-accelerographs from which records were recovered after the Northridge earthquake were analog (film) instruments, so no rapid recovery or distribution was possible at that time.

## LESSONS LEARNED REGARDING CODE INSTRUMENTS

Though record recovery was effective and quite successful after the Northridge earthquake, some aspects became clear during the course of these efforts. Addressing these aspects can make code data more useful, and recovery more rapid and effective after future earthquakes.

1) <u>Documentation</u> Earthquakes are rare events, from which data are desired rapidly once an event has occurred. Thus, in-depth research and historical investigation are not practical, and documentation must be readily available that is accurate and reliable, preferably on an automated basis.

a) Building Information: Key information is needed about a building to make the data useful after an earthquake. These include number of stories, construction data and structural system, among others. This information can be maintained by the City with assistance by the State as appropriate.

b) Instrument Location: A record from an instrument after an earthquake is almost useless, and possibly counterproductive, if it is not known where in a building the instrument is located. This means both floor (height in the building) and location on that floor. A very different record may be obtained on the same floor if the instrument is at an outer wall as opposed to being near the central elevator core. In terms of vertical motion, a very different record will be obtained if the instrument is near a column as opposed to the center of a span, where the vertical motion of the floor diaphragm can dominate the record (e.g., Canoga Ave #2 in Fig. 6). This vertical motion can be misinterpreted as the vertical vibration of the building. Unless there are special circumstances, the instrument should be placed near the central core of the building, near a column. This should probably be a recommendation or requirement in a future code revision.

c) Instrument Orientation: The orientation of an instrument seems a trivial issue, but the orientation is now often not recorded for code instruments. As a result, the record is difficult to use effectively, particularly if rapid usage is desired. A building could have very different motion in each direction, either because the structural system in each direction is different or because the input is different in the two directions. Also, a building in which the framing system has been damaged in one direction could have a very different response in that direction. The code now makes no mention of orientation. This should probably be addressed in a future revision, so that there is a specification for which way the instrument should be aligned relative to a chosen building direction.

2) Instrument Maintenance While the Northridge data suggests that the maintenance of instruments was generally adequate, improved and consistent maintenance standards are important. The certification of Maintenance Companies and the screening of periodic test records are important. Also, standardized acceptance levels should be established for test records. The State could provide help to city building departments so they can be as effective as possible without undue investment of staff time.

3) Instrument Upgrade The upgrade of code instrumentation from an analog film recorder to a digital recorder allows shaking data to be recovered, processed and analyzed quickly after an earthquake. Modern digital accelerographs have significantly better accuracy than the older analog units, and do not have significantly higher purchase or maintenance costs. New installations should probably be all digital, and upgrading would be effective in some cases. The State can again provide help to city building departments on digital instrumentation and communication equipment.

4) Extend Code Accelerograph Requirement Many cities require accelerographs in tall structures corresponding to the UBC instrumentation provision. However, a significant number of communities have not adopted such a requirement. Some jurisdictions had buildings with damage, but no measurement of the motion in the buildings. These communities should be encouraged to adopt an instrument requirement. Although modifying the UBC code language to require only two accelerographs at the top and base may be a reasonable step, requiring an instrument only at the top appears unwise after the Northridge earthquake, and in fact Los Angeles is modifying their regulation so that its calls for an instrument at the base as well as at the top (K. Deppe, personal communication).

## ACKNOWLEDGEMENTS

CSMIP acknowledges the City of Los Angeles for their cooperation with this project and extends its appreciation to the members of the Strong Motion Instrumentation Advisory Committee and its subcommittees. Agbabian Associates wishes to acknowledge the record recovery work of Vance Weaver (KMI) and Tony Costa (SSS). The code-instrument data recovery was cofunded with the National Science Foundation (Grant No. CMS-9410894).

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**TABLE 1: SUMMARY OF PROCESSED DATA FROM CODE-REQUIRED ACCELEROGRAPHS IN BUILDINGS**

| Building Name | Sta. No. | No. Chns | Instrument Locations | No. of Stories | Max. Horiz. Accel.(g) Base | Roof |
|---|---|---|---|---|---|---|
| Encino - Ventura Blvd #1 | 2C003 | 3 | Grnd (arcade) | 17 | 0.54 | * |
| Encino - Ventura Blvd #4 | 2C088 | 3 | Roof | 13 | --- | 0.44 |
| Encino - Ventura Blvd #9 | 2C201 | 9 | Bsmt, 6th, roof | 12 | 0.46 | 0.47 |
| Los Angeles - Ave of the Stars #2 | 2C016 | 3 | Roof | 36 | --- | 0.35 |
| Los Angeles - Beverly Blvd #1 | 2C253 | 9 | Grnd, 5th, roof | 10 | 0.22 | 0.47 |
| Los Angeles - McClintock Ave #2 | 2C022 | 9 | Grnd, 6th, roof | 10 | 0.31 | 0.28 |
| Los Angeles - Olympic Blvd #1 | 2C024 | 3 | Roof | 9 | --- | 0.70 |
| Los Angeles - Olympic Blvd #2 | 2C289 | 3 | Roof | 11 | --- | 1.07 |
| Los Angeles - Olympic Blvd #3 | 2C250 | 3 | Roof | 10 | --- | 0.71 |
| Los Angeles - Olympic Blvd #4 | 2C161 | 3 | Roof | 12 | --- | 0.56 |
| Los Angeles - Wilshire Blvd #1 | 2C043 | 3 | Roof | 23 | --- | 0.63 |
| Los Angeles - Wilshire Blvd #2 | 2C040 | 3 | Roof | 17 | --- | 0.54 |
| Los Angeles - Wilshire Blvd #3 | 2C165 | 3 | Roof | 17 | --- | 0.28 |
| Los Angeles - Wilshire Blvd #4 | 2C168 | 3 | Roof | 21 | --- | 0.37 |
| Los Angeles - Wilshire Blvd #5 | 2C066 | 3 | Roof | 14 | --- | 0.30 |
| Los Angeles - Wilshire Blvd #6 | 2C042 | 3 | Roof | 17 | --- | 0.29 |
| Los Angeles - Wilshire Blvd #7 | 2C067 | 3 | Roof | 13 | --- | 0.34 |
| Los Angeles - Wilshire Blvd #8 | 2C041 | 9 | Lobby, 9th, roof | 18 | 0.19 | 0.33 |
| Los Angeles - Wilshire Blvd #9 | 2C009 | 9 | Bsmt, 11th, roof | 21 | 0.24 | 0.20 |
| Los Angeles - Wilshire Blvd #10 | 2C131 | 9 | Bsmt, 12th, roof | 24 | 0.27 | 0.35 |
| Los Angeles - Wilshire Blvd #11 | 2C209 | 6 | Grnd, roof | 9 | 0.22 | 0.39 |
| North Hollywood - Lankershim #1 | 2C173 | 3 | Roof | 8 | --- | 0.35 |
| North Hollywood - Lankershim #2 | 2C083 | 9 | Bsmt, 5th, roof | 8 | 0.31 | 0.36 |
| North Hollywood - Magnolia #1 | 2C215 | 3 | Roof | 12 | --- | 0.74 |
| Northridge - Oakdale Ave #1 | 2C001 | 3 | Roof | 10 | --- | 0.46 |
| Northridge - Roscoe Blvd #1 | 2C130 | 7 | 1st, 4th, roof | 7 | 0.42 | 0.59 |
| Sherman Oaks - Ventura Blvd #6 | 2C014 | 9 | Bsmt, 8th, roof | 15 | 0.47 | 0.48 |
| Sherman Oaks - Ventura Blvd #7 | 2C126 | 3 | Roof | 21 | --- | 0.47 |
| Sherman Oaks - Ventura Blvd #8 | 2C132 | 3 | Roof | 8 | --- | 0.79 |
| Tarzana - Ventura Blvd #10 | 2C015 | 9 | Grnd, 5th, roof | 10 | 0.47 | 0.52 |
| Van Nuys - Sherman Way #1 | 2C233 | 9 | 1st, 6th, roof | 12 | 0.37 | 0.66 |
| Woodland Hills - Canoga Ave #1 | 2C135 | 3 | Roof | 17 | --- | 0.39 |
| Woodland Hills - Canoga Ave #2 | 2C133 | 3 | Roof | 17 | --- | 0.49 |
| Woodland Hills - Canoga Ave #3 | 2C106 | 3 | Roof | 15 | --- | 1.04 |
| Woodland Hills - Oxnard Blvd #2 | 2C210 | 3 | Roof | 21 | --- | 0.31 |
| Woodland Hills - Oxnard Blvd #3 | 2C232 | 3 | Roof | 17 | --- | 0.72 |
| Woodland Hills - Oxnard Blvd #4 | 2C246 | 9 | Bsmt, 6th, roof | 12 | 0.44 | 0.33 |
| Woodland Hills - Ventura Blvd #5 | 2C206 | 6 | 6th, roof | 12 | * | 0.55 |
| Woodland Hills - Victory Blvd #1 | 2C085 | 3 | Roof | 12 | --- | 0.57 |
| Woodland Hills - Victory Blvd #2 | 2C125 | 3 | Roof | 8 | --- | 0.79 |

--- No accelerograph.

* Accelerograph did not function.

**TABLE 2: SUMMARY OF PROCESSED DATA FROM CSMIP EXTENSIVELY-INSTRUMENTED BUILDINGS**

| Building Name | Station Number | No. Chns | Sensor Locations | Max. Horiz. Accel.(g) Base | Struct. |
|---|---|---|---|---|---|
| Burbank - 6-story Commercial | 24370 | 13 | Grnd, 2nd, 3rd, roof | 0.35 | 0.49 |
| Burbank - 10-story Residential | 24385 | 16 | 1st, 4th, 8th, roof | 0.35 | 0.79 |
| Colton - 1-story School Gym | 23540 | 13 | Grnd, top walls, roof | 0.04 | 0.23 |
| El Segundo - 14-story Office | 14654 | 16 | 1st, 4th, 9th, roof | 0.13 | 0.25 |
| Lancaster - 5-story Hospital | 24609 | 12 | 1st, 4th, roof | 0.07 | 0.28 |
| Los Angeles - 2-story Fire Command | 24580 | 16 | Foundation, 1st, 2nd, roof | 0.22 | 0.35 |
| Los Angeles - 3-story Commercial | 24332 | 15 | Level B and mall, 2nd, roof | 0.33 | 0.97 |
| Los Angeles - 5-story Warehouse | 24463 | 13 | Bsmt, 2nd, 3rd, roof | 0.26 | 0.29 |
| Los Angeles - 6-story Parking Structure | 24655 | 14 | 1st, 4th, roof | 0.29 | 1.21 |
| Los Angeles - 6-story Office | 24652 | 14 | Bsmt, 1st, 3rd, roof | 0.24 | 0.59 |
| Los Angeles - 7-story UCLA Math-Sci. | 24231 | 12 | 1st, 3rd, 5th, roof | 0.29 | 0.77 |
| Los Angeles - 7-story Univ. Hospital | 24605 | 24 | Foundation, lower, 4, 6, roof | 0.37 | 0.21 |
| Los Angeles - 8-story CSULA Admin. | 24468 | 16 | Grnd, 2nd, roof | 0.17 | 0.25 |
| Los Angeles - 9-story Office | 24579 | 18 | Bsmt, 2nd, 5th, roof | 0.18 | 0.34 |
| Los Angeles - 13-story Office | 24567 | 15 | Bsmt, 2nd, 8th, roof | 0.18 | 0.37 |
| Los Angeles - 14-story Hollywood Str. | 24236 | 12 | Bsmt, 8th, 12th, roof | 0.28 | 0.49 |
| Los Angeles - 15-story Gov. Office | 24569 | 15 | B level, 2nd, 8th, roof | 0.21 | 0.52 |
| Los Angeles - 17-story Residential | 24601 | 14 | 1st, 7th, 13th, roof | 0.26 | 0.58 |
| Los Angeles - 19-story Office | 24643 | 15 | Level D, 1st, 2nd, 8th, roof | 0.32 | 0.65 |
| Los Angeles - 52-story Office | 24602 | 20 | Lvl E, A, 14, 22, 35, 49, roof | 0.15 | 0.41 |
| Los Angeles - 54-story Office | 24629 | 20 | P4, grnd, 20, 36, 46, pent | 0.14 | 0.19 |
| Newport Beach - 11-story Hospital | 13589 | 18 | Service level, 3rd, 6th, roof | 0.08 | 0.26 |
| North Hollywood - 20-story Hotel | 24464 | 16 | Bsmt, 3rd, 9th, 16th, roof | 0.33 | 0.66 |
| Pasadena - 6-story Office | 24541 | 16 | Bsmt, 2nd, 6th, attic, roof | 0.17 | 0.21 |
| Pasadena - 9-story Commercial | 24571 | 15 | Bsmt, 2nd, 5th, roof | 0.19 | 0.29 |
| Pasadena - 12-story Commercial/Office | 24546 | 15 | 2nd, 7th, 12th, roof | 0.18 | 0.32 |
| Pasadena - 12-story Office | 24566 | 15 | Grnd, 5th, 6th, roof | 0.23 | 0.31 |
| Pomona - 2-story Commercial | 24511 | 10 | Bsmt, 2nd, roof | 0.06 | 0.22 |
| Rancho Cucamonga - Law & Justice | 23497 | 16 | Foundation, bsmt, 2nd, roof | 0.05 | 0.10 |
| San Bernardino - 1-story Commercial | 23622 | 10 | Grnd, roof | 0.08 | 0.15 |
| San Bernardino - 5-story Hospital | 23634 | 12 | 1st, 3rd, roof | 0.08 | 0.35 |
| San Bernardino - 5-story CSUSB Lib. | 23285 | 10 | Bsmt, 3rd, roof | 0.04 | 0.21 |
| San Bernardino - 6-story Hotel | 23287 | 9 | 1st, 3rd, roof | 0.07 | 0.23 |
| Seal Beach - 8-story Office | 14578 | 28 | Bsmt, 1st, 2nd, 6th, roof | 0.08 | 0.15 |
| Sherman Oaks - 13-story Commercial | 24322 | 15 | 2nd bsmt, grnd, 2nd, 8th, roof | 0.46 | 0.90 |
| Sylmar - 6-story County Hospital | 24514 | 13 | Grnd, 3rd, 4th, roof | 0.80 | 1.70 |
| Van Nuys - 7-story Hotel | 24386 | 16 | Grnd, 2nd, 3rd, 6th, roof | 0.47 | 0.59 |
| Whittier - 8-story Hotel | 14606 | 12 | 1st, 5th, roof | 0.19 | 0.49 |

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# IMPLICATIONS OF THE STRONG-MOTION RECORDS FROM A RETROFITTED CURVED BRIDGE ON SEISMIC DESIGN AND PERFORMANCE

W.D. Liu, A. Kartoum, S. Dhillon, X. Chen, R.A. Imbsen
Imbsen & Associates, Inc.
Sacramento, California

## ABSTRACT

Structural responses recorded during three recent earthquakes (1992 Landers and Big Bear and 1994 Northridge) were used to evaluate the performance of the retrofitted single column viaduct (Route 10/215 Interchange in Colton). The column deformation experienced range from low in the Northridge earthquake up to about 70% yield deformation level during the Landers earthquake. These results were compared with the deformation-based design practice for column retrofit. Effect of foundation contribution was also compared with the design practice. Results from this study support the successful performance of other similarly retrofitted bridges during the Northridge earthquake.

## INTRODUCTION

While most post-earthquake investigations focus on the structure failure, it is equally important to identify and verify the effectiveness of successful design schemes which meet the desired seismic performance goal. The connector interchange structures are typically very important structures because of their strategic location in the urban transportation network. Therefore, these bridges have been given priority for seismic upgrading.

The June 28, 1992, Landers and Big Bear earthquakes with magnitudes, $M_s$, 7.5 and 6.6, respectively, were among the largest magnitude earthquakes recorded in the United States. This was the first time that a major curved bridge was monitored during a strong earthquake. More importantly, this was the first time that a bridge, retrofitted with steel confinement casings around columns, was monitored during a strong earthquake.

During the January 17, 1994, Northridge earthquake (Magnitude $M_w$ 6.7), the highway bridge structures in the Los Angeles area suffered significant damage. Among these damaged structures, two curved viaducts in the Route 14/5 Interchange suffered partial collapse. These are single column viaducts designed and constructed in the early 1970s which had not been retrofitted for column deficiencies.

A similar curved, single column viaduct structure, the Route 405/10 Interchange, survived the strong ground shaking. This bridge is about 15 miles south of the epicenter, four miles west of the collapse section on I10 and five miles east of a Santa Monica site where horizontal ground motion up to 0.93 g was recorded. Regardless of the close proximity to the epicenter and the expected strong shaking, the bridge suffered only the toppling of rocker bearings at the abutment. The instrument on the bridge recorded a peak vertical acceleration of 1.83 g. The peak horizontal motions recorded were 1.0 g at the west abutment and 0.52 g (transverse) at top of Bent 9. Other than the bearing damage, the bridge did not suffer any noticeable damage. The satisfactory performance of this bridge was attributed to the retrofit scheme used for the columns and the footings. Unfortunately, the instrumentation on this bridge is not sufficient to conduct more detailed performance assessment.

These two bridges are geometrically similar to the fully instrumented interchange in Colton. To establish an important benchmark, the Route 10/215 Interchange and the recorded ground motions during the past three earthquakes are used in this study. The emphasis is on the behavior of the retrofitted single column bent, and how they compare with current design practice using a deformation-based design methodology. In addition to the column performance, a detailed evaluation of the fully instrumented Bent 8 allows an assessment of the foundation effect and correlation with current design practice for calculation of foundation stiffness. The overall dynamic characteristics of the bridge have also been determined based on measurements to allow a comparison with the analytical model prediction.

To account for the expected pounding effect at the span hinges, a procedure based on the restitution coefficient approach is proposed to account for energy loss.

A previous study by Fenves & Desroches (1995) focused on the global response correlation with the analytical model. In this paper, we will focus on the seismic performance and implication to the design practice.

## DESCRIPTION OF THE BRIDGE

The Route 10/215 Interchange is located near the city of Colton, in San Bernardino County, California. The interchange is comprised of several bridges, including the NW Connector Overcrossing that connects eastbound Route 10 from Los Angeles to northbound Route 215 to San Bernardino. This bridge has been instrumented to monitor seismic performance by the California Division of Mines and Geology as part of its Strong Motion Instrumentation Program (CSMIP).

The NW Connector Overcrossing is a curved, cast-in-place, concrete box girder bridge, as shown in Figure 1. The alignment of the bridge consists of a compound curve having radii of 1200 feet and 1300 feet. There are 16 spans, having a total length of 2540 feet and width of 41 feet. The superstructure has five intermediate span hinges to accommodate movements and to separate post-tensioned concrete spans from conventionally reinforced spans. The bridge substructure is comprised of monolithic abutments and single column bents. The columns are 5.5 feet by 8.0 feet flared octagonal shaped. The foundations consist of pile footings with 70 ton driven concrete piles at the abutments and bents, except for Bent 2 which has 100 ton piles. Both Bent 3 and Bent 8 are founded on footings with 36 piles.

The bridge was originally constructed in 1972 with several inherent seismic vulnerabilities that include: inadequate widely spaced ties in the columns, no top mat of reinforcement in the footings, and insufficient support length at the hinges. In 1991, the bridge was seismically retrofitted by placing steel jackets ($^1/_2$" thick) around columns, strengthening footings and abutments, and by installing new restrainers at the hinges. At 12 of the 15 bents, full-height jackets were provided and the footings were enlarged with driven steel piles provided to increase the moment capacity of the foundations. At Bents 8, 12 and 14, steel jackets were provided at the lower 18 feet above the footing, and the footings were not modified. Abutment catchers were provided to prevent drop-span failure if abutment backwall failed.

## DESCRIPTION OF THE INSTRUMENTATION

A total of 34 sensors are used on the bridge, as shown in Figure 1. The maximum acceleration values observed in each sensor are summarized in Table 1 for the three earthquakes. Table 2 shows the sensor layout by individual frames.

The instrumentation layout plan was carefully thought out with the following features:

a. Each frame was instrumented with at least three sensors (one longitudinal and two transverse). This allows a complete characterization of the lower horizontal modes of each frame.

b. The instrumentation at Bent 8 includes four sensors at the foundation and four sensors at the deck. This provides a unique opportunity to study in detail the transverse response of the bent including:

   - the effect of soil-structure interaction at the base;
   - the effect of the rotational support input motion;
   - the effective stiffness property (i.e., moment of inertia) of the retrofitted column; and
   - the modeling assumptions typically used for the single column bent.

   Similar bent transverse study can also be conducted for Bent 3.

c. The instrumentation on Span 7 includes four vertical sensors to quantify the vertical and torsional vibration of the cantilever deck segment.

d. Across each expansion joint (span hinge), there is at least two transverse sensors. In four hinges, longitudinal sensors are also used. Given the longitudinally restrained boundary conditions at both abutments, these pairs of sensors at hinges can be studied to establish the sequence of impacting occurred.

The free-field ground motions are recorded in the nearby CSMIP Station 23542. However, they are not synchronized with the sensors on the structure.

### Strong-Motion Data – Free Field Motions

The three-component free-filed motions were rotated to the longitudinal (tangential) and transverse directions at Bent 8. The 5% damped response spectra are calculated. The spectral accelerations for these three vibration periods are summarized in Table 3. In the transverse direction, the Landers earthquake is much higher for periods greater than 1.5 sec. For intermediate period range between 0.4 to 0.8 sec., the Northridge earthquake is much stronger than the others.

Strong-motion data for the superstructure deck with and without the high frequency spikes were reported by Huang & Shakal (1995) for the Landers and Big Bear earthquakes. It is inferred that the spikes were generated by the impact between adjacent deck sections across the span hinges. (Malhotra, et al, 1994)

## RESPONSE INTERPRETATION OF BENT 8

The pile supported single column bent is instrumented with eight sensors: four on the deck level and four on the footing level. To facilitate interpretation, the transverse response of the bent structure is idealized as a three degrees of freedom system: translation of the deck and translation and rotation of the pile cap, as shown in Figure 2.

### Analytical Formulation

The total motion at the bent cap is composed of two components: column deformation u, and the foundation contribution $\bar{u}$ which can be defined as follows:

$$u^t(t) = \bar{u}(t) + u(t) \tag{1}$$

$$\bar{u}(t) = u_o(t) + L\,\theta_o(t) \tag{2}$$

where L is the height to the C.G. of the deck. Given the foundation stiffness coefficients for the translational and rotational degrees of freedom, $k_u$ and $k_\theta$, the effective foundation stiffness is defined as:

$$\bar{k} = \left(\frac{1}{k_u} + \frac{L^2}{k_\theta}\right)^{-1} \tag{3}$$

This is schematically shown in Figure 2(d). The total lateral stiffness of the bent including the foundation contribution is:

$$K_{eff} = k\left(1 + \frac{k}{\bar{k}}\right)^{-1} \tag{4}$$

This is shown in Figure 2(e). This allows a systematic evaluation of the column behavior and the foundation effect.

### Column Behavior Characteristics

By eliminating the foundation contribution, $\bar{u}$, the measured response data can be used to quantify the behavior characteristics of the column, i.e., the column stiffness k, damping c, the fixed-base vibration frequency $\omega_{FB}$,

$$\omega_{FB} = \sqrt{k/m} \tag{5}$$

where m is the tributory mass of the superstructure. The equation of motion for the fixed-base bent is

$$m\ddot{u}^t + c\dot{u} + k u = o \tag{6}$$

or

$$\ddot{u}^t + 2\xi\omega_{FB}\dot{u} + \omega_{FB}^2 u = o \tag{7}$$

Using measured time history response $\ddot{u}^t(t)$ at the deck (channel 20) and the derived column deformation time history u(t), the parameters $\omega_{FB}$ and $\xi$ can be determined by the equivalent linearization technique.

Using this approach, the column properties were identified for the three earthquakes. This was done using the entire measured duration. The results are summarized in Table 4.

The column stiffness k is calculated based on the tributory weight of 2973 kip at Bent 8 (including the box-girder superstructure and half of the column weight). The normalized force-displacement hysteretic loops are shown in Figures 3(a) through 3(c) for the three earthquakes. The maximum column deformations vary from 5.6 inches for the Landers earthquake, 3.5 inches for the Big Bear earthquake, and 1.25 inches for the Northridge earthquake. These displacements cover a wide range. As shown in Table 4, the fixed-base vibration frequency and the effective linear stiffnesses of the column vary consistently with the amplitude.

To further evaluate the column behavior during these earthquakes, the response time histories were divided into 10 second duration windows. For each time window, the root-mean-squared (RMS) displacement and normalized force responses were calculated. These force-displacement pairs are shown in Figure 4 which shows clearly a nonlinear relationship. At the lower RMS displacement, the stiffness (as indicated by the slope) is much higher, and gradually softens as the RMS displacement reached 1.5" to 2.5" level.

## Correlation with Analytical Capacity Prediction

In the current practice, a deformation-based design methodology is being promoted. For each critical cross section, the moment-curvature relationship is computed based on the nonlinear material constitutive relationships for concrete and steel reinforcement. The objective is to quantify the deformation capacity provided by the design reinforcement details, and to allow a judicious selection of design criteria (e.g. not to allow concrete strain beyond a certain level).

For the retrofitted column sections with partial height steel casing, it is important to quantify the increase in lateral stiffness. This is caused by the bond transfer between the original column and the steel shell and results in partial composite action. For flexural columns with aspect ratio around L/D = 6, laboratory test showed a modest increase of lateral stiffness by 10% to 15%. (Chai, 1996)

This can be accounted for in the component stiffness calculation. As shown in Figure 5, the effective moment of inertia varies along the column height. In the center portion of the encased region, the full composite action is possible to be developed. From there on to the ends of the casing, partial composite action exists.

For the Bent 8 column, several moment-curvature relationships were used for the upper unencased section (flared), the lower gapped zone, and the varying degrees of composite action. Using these results, the fixed-base column lateral force-deformation curve was developed, as shown in Figure 5(e). Based on the bilinear idealization, the column with steel casing retrofit will yield at a displacement level of about 3.5 inches. This compares reasonably well with the measured results, as shown in Figure 4.

The secant stiffness in the transverse direction corresponding to the yield of the steel reinforcement is 1630 kip/ft. This is consistent with the stiffnesses derived from the measured results because of the higher deformation level implied in the calculation, as shown in Figure 4.

## Foundation Flexibility Effects

Based on available soil boring data, the standard procedures as summarized in the FHWA Report entitled *Foundation Design to Resist Earthquake Loads*, are used to calculate the pile group stiffness coefficients. For the transverse translation and the rotation about the longitudinal axis, the combined pile group and pile cap stiffnesses are:

$$k_u = 1.86 \times 10^5 \text{ kip/ft}; \; k_\theta = 7.22 \times 10^7 \text{ kip-ft/rad.}$$

The effective foundation stiffness as defined in Eq. (3) is

$$\bar{k} = 28{,}832 \text{ kip/ft}$$

It is of interest to note that the translation foundation stiffness contributes to only 15% of the total foundation effect on the bent cap displacement. For design purpose, the rocking effect of pile group foundation is more critical to the seismic response prediction.

As shown in Eq. (4), the foundation contribution to the bent cap displacement is determined by $(1+ k/\bar{k})$. These results are summarized in Table 4 for the three earthquakes. Based on these results, the overall transverse vibration frequency is only affected by 3% to 6% during these three earthquakes. The bent cap displacements during these three earthquakes were affected by 7% to 12% due to foundation flexibility. This compares very well with observed data.

## MODELING REFINEMENT FOR SPAN HINGE IMPACT

The nonlinear behavior (cable stretching and deck impacting) across the span hinge is a critical feature of the overall bridge response under strong earthquake shaking. During the 1989 Loma Prieta earthquake, this has been observed in the Route 24/580/980 Interchange, a highly curved viaduct. (Liu, et al, 1994).

This deck impacting can also be seen in the transverse shear force hysterestic loop of the column. When the bridge moves outward (negative displacement), the adjacent span separates; when the bridge moves inward (with positive column displacement), the deck impact at the inside edge occurs and additional force develops. This is most clearly shown in Figure 3(b) for the Big Bear earthquake.

One of the critical issues has been how to model the stiffness characteristics and energy loss during impact. A usual practice is to use a very large elastic impact stiffness to prevent the overlapping of adjacent deck. However, this may result in the unusually high force. Kawashima and Penzien (1976) had conducted a correlation study with shake table testing of a curved bridge model. They recommended the use of an elastic impact stiffness which is ten times the axial stiffness of the deck. This will assure that the duration of impact is sufficiently short. However, no energy loss was considered.

The *coefficient-of-restitution* approach can be used to model the finite duration impact and energy loss. (Cross and Jones, 1993) In short, this impact restitution approach states that the relative separation velocity immediately after the impact, $v_s$, is a fraction of the relative approaching velocity before the impact, $v_A$ :

$$v_s = e\, v_A \tag{8}$$

where

e = coefficient of restitution ($0 \le e \le 1$)
e = 1 perfect elastic rebound
e = o no impact

This is schematically shown in Figure 6. During the impact, the two adjacent sections stick together and the connection can be represented using an equivalent spring-damper at the point of contact for the duration of impact. To assure the impact duration is short, the impact stiffness should be much greater (say ten times) than the structural stiffness.

For the given coefficient of restitution, it can be shown that the equivalent dashpot coefficient $c_I$ during impact is

$$\xi_I = \frac{c_I}{c_{cr}} = 10.5 \frac{-\frac{\ln e}{\pi}}{\sqrt{1+\left(\frac{\ln e}{\pi}\right)^2}} \tag{9}$$

where $c_{cr}$ is the critical damping of the structural frame. For various values of e, the equivalent dashpot can be determined. This approach allows the energy loss during impact to be taken into account in the analytical model.

## GLOBAL DYNAMIC CHARACTERISTICS

Based on the measured responses at each frame, Fourier amplitude spectra were computed. These were used to identify the dominant frequencies, as shown in Figure 7. Using the cross power spectral density functions calculated relative to a fixed reference point in each frame, the phase angles were determined and the vibration mode shapes of the bridge can be portrayed.

Using the data collected during Landers earthquake, the first three modes are shown in Figure 8. Because of the noise in the data and the signal processing procedure, two very closely-spaced frequencies and shapes were determined for the first mode, as shown in the Figure. However, based on the distribution of substructure stiffnesses as reflected by the column heights, we believe there is only one mode indicated by the solid line in the Figure.

## SUMMARY AND CONCLUSIONS

Data collected from three recent earthquakes at the I10/215 Interchange were utilized to evaluate the seismic performance including the component column behavior, foundation flexibility effect, overall structural periods, and mode shapes.

The most significant findings were that the behavior characteristics of the retrofitted columns can be predicted quite well using the deformation-based methodology. Since the response levels in these three earthquakes vary from low (Northridge earthquake) to high (Landers earthquake), these comparisons provided a valuable benchmark. As indicated in the measured data, columns were loaded well beyond cracking load and reached 70% of the calculated yield deformation.

The nonlinear force-deformation behavior was observed in the measured data which is very consistent with the analytical prediction. Because of the excellent correlation, the expected seismic performance of the retrofitted column can be assured. Note that the peak transverse acceleration at top of Bent 8 was 0.39 g (Channel 20) and 0.51 g (Channel 19) across the span hinge during the Landers èarthquake. This further verified the excellent performance of Route 405/10 Interchange with similar steel jacket retrofit for the single column bent. For that bridge, the peak acceleration recorded at the top of Pier 9 (a 38.5 feet tall column) was 0.52 g in the transverse direction. The measured responses at these two bridges may be considered as the minimum strength of the retrofitted design.

Using the recorded data, additional comparisons were made to benchmark the foundation stiffness calculation procedure typically used in practice. First, it is recognized that most of the foundation flexibility is caused by the rocking of the footing. Secondly, the total deck displacement caused by foundation is predicted to be 7% to 12% which compares very well with the measured data.

It is recognized by several previous research studies that the hinge nonlinear effects, including cable stretching and deck impact, are important. An analytical model is proposed which allows the consideration of energy loss during impact by the restitution coefficient approach. Further studies will be required to calibrate the appropriate restitution coefficient, e, for the various superstructure types.

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**Table 1: San Bernardino Interstate Route 10/215 Interchange – NW Connector Overcrossing – CSMIP Station No. 23631**

| Sensor # | Motion Direction | Location | $A_{max}$ Landers Earthquake | $A_{max}$ Big Bear Earthquake | $A_{max}$ Northridge Earthquake |
|---|---|---|---|---|---|
| 1 | Longitudinal | Deck, West Abutment 1 | 0.57g | 0.43g | 0.15g |
| 2 | Vertical | Deck, West Abutment 1 | 0.20g | 0.10g | 0.06g |
| 3 | Transverse | Deck, West Abutment 1 | 0.25g | 0.19g | 0.10g |
| 10 | Longitudinal | Deck Hinge Near Bent 3, West Side | 0.45g | 0.34g | 0.15g |
| 7 | Transverse | Deck Hinge Near Bent 3, West Side | 0.36g | 0.38g | 0.17g |
| 8 | Transverse | Deck Hinge Near Bent 3, East Side | 0.59g | 0.48g | 0.18g |
| 4 | Longitudinal | Footing, Bent 3 | 0.10g | 0.09g | 0.06g |
| 6 | Transverse | Footing, Bent 3 | 0.10g | 0.11g | 0.10g |
| 11 | Transverse | Deck between Bents 5 & 6, Mid-Span | 0.39g | 0.28g | 0.12g |
| 9 | Vertical | Deck, Bent 7 | 0.21g | 0.17g | 0.11g |
| 14 | Vertical | Deck between Bents 7 & 8, Mid-Span | 0.36g | 0.31g | 0.20g |
| 15 | Vertical | Deck Hinge Near Bent 8, North Side | 0.35g | 0.32g | 0.16g |
| 16 | Vertical | Deck Hinge Near Bent 8, South Side | 0.45g | 0.33g | 0.18g |
| 12 | Vertical | Deck, Bent 8, North Side | 0.26g | 0.21g | 0.13g |
| 13 | Vertical | Deck, Bent 8, South Side | 0.38g | 0.23g | 0.31g |
| 5 | Vertical | Footing, Bent 8, North Side | 0.11g | 0.08g | 0.03g |
| 23 | Vertical | Footing, Bent 8, South Side | 0.07g | 0.08g | 0.04g |
| 17 | Longitudinal | Deck Hinge Near Bent 8, West Side | 0.66g | 0.34g | 0.08g |
| 18 | Longitudinal | Deck Hinge Near Bent 8, East Side | 0.71g | 0.58g | 0.08g |
| 22 | Longitudinal | Footing, Bent 8, South Side | 0.17g | 0.25g | 0.08g |
| 19 | Transverse | Deck Hinge Near Bent 8, West Side | 0.51g | 0.51g | 0.16g |
| 20 | Transverse | Deck Hinge Near Bent 8, East Side | 0.39g | 0.33g | 0.15g |
| 24 | Transverse | Footing, Bent 8, South Side | 0.18g | 0.15g | 0.13g |
| 25 | Transverse | Deck Hinge Near Bent 10, West Side | 0.33g | 0.29g | 0.14g |
| 26 | Transverse | Deck Hinge Near Bent 10, East Side | 0.31g | 0.25g | 0.15g |
| 28 | Longitudinal | Deck Hinge Near Bent 11, West Side | 0.29g | 0.42g | 0.12g |
| 33 | Longitudinal | Deck Hinge Near Bent 11, East Side | 0.82g | 0.68g | 0.09g |
| 29 | Transverse | Deck Hinge Near Bent 11, West Side | 0.29g | 0.30g | 0.18g |
| 30 | Transverse | Deck Hinge Near Bent 11, East Side | 0.43g | 0.41g | 0.26g |
| 31 | Transverse | Deck Hinge Near Bent 14, West Side | 0.36g | 1.02g | 0.47g |
| 32 | Transverse | Deck Hinge Near Bent 14, East Side | 0.47g | 0.67g | 0.31g |
| 34 | Longitudinal | Deck, North Abutment 17 | 0.36g | 0.23g | 0.16g |
| 35 | Vertical | Deck, North Abutment 17 | 0.13g | 0.11g | 0.05g |
| 36 | Transverse | Deck, North Abutment 17 | 0.15g | 0.20g | 0.14g |

**Table 2: Interstate Route 10/215 Interchange Sensor Locations by Frame**

| Structure Frame | Direction* | Sensor ID | |
|---|---|---|---|
| | | Deck | Foundation |
| 1 Abutment 1 to Hinge 1 (Bents 2 and 3) | L | 10 | 1, 4 |
| | T | 7 | 3, 6 |
| | V | – | 2 |
| 2 Hinge 1 to Hinge 2 (Bents 4, 5, 6 and 7) | L | 17 | – |
| | T | 8, 11, 19 | – |
| | V | 9, 14, 15, 16 | – |
| 3 Hinge 2 to Hinge 3 (Bents 8 and 9) | L | 18 | 22 |
| | T | 20, 25 | 24 |
| | V | 12, 13 | 5, 23 |
| 4 Hinge 3 to Hinge 4 (Bents 10 and 11) | L | 28 | – |
| | T | 26,29 | – |
| | V | – | – |
| 5 Hinge 4 to Hinge 5 (Bents 12 and 13) | L | 33 | – |
| | T | 30, 31 | – |
| | V | – | – |
| 6 Hinge 6 to Abutment 17 (Bents 14, 15 and 16) | L | – | 34 |
| | T | 32 | 36 |
| | V | – | 35 |

* L: Longitudinal
T: Transverse
V: Vertical

**Table 3: Summary of Free Field Spectral Acceleration**

| | PGA | T = 0.5 sec. | T = 1.5 sec. |
|---|---|---|---|
| **Longitudinal** | | | |
| Landers | 0.08 g | 0.25 g | 0.10 g |
| Big Bear | 0.05 g | 0.18 g | 0.15 g |
| Northridge | 0.08 g | 0.16 g | 0.02 g |
| **Transverse** | | | |
| Landers | 0.08 g | 0.22 g | 0.30 g |
| Beg Bear | 0.11 g | 0.18 g | 0.15 g |
| Northridge | 0.07 g | 0.38 g | 0.02 g |

**Table 4: Column Stiffness, Damping and Foundation Effect at Bent 8**

| | $f_{FB}$ (Hz) | $\xi$ | k (kip/ft) | $u_{max}$ (inch) | $1+\frac{k}{\bar{k}}$ |
|---|---|---|---|---|---|
| Landers EQ | 0.73 | 3.9% | 1942 | 5.60 | 1.07 |
| Big Bear EQ | 0.83 | 4.4% | 2511 | 3.50 | 1.09 |
| Northridge EQ | 0.96 | 4.1% | 3359 | 1.25 | 1.12 |

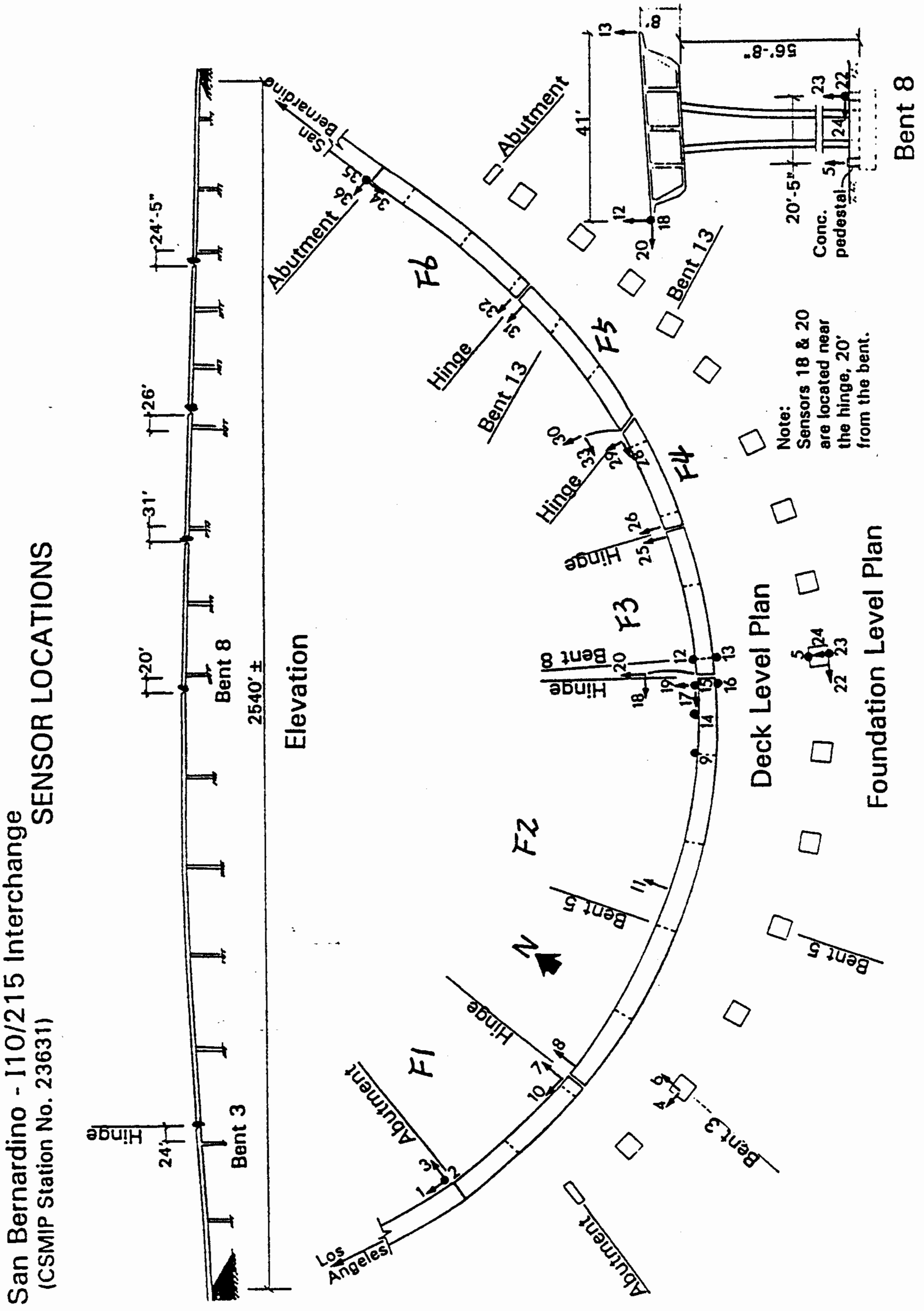


Figure 1: Plan, Elevation and Instrumentation Locations of the NW Connector in the I10/215 Interchange

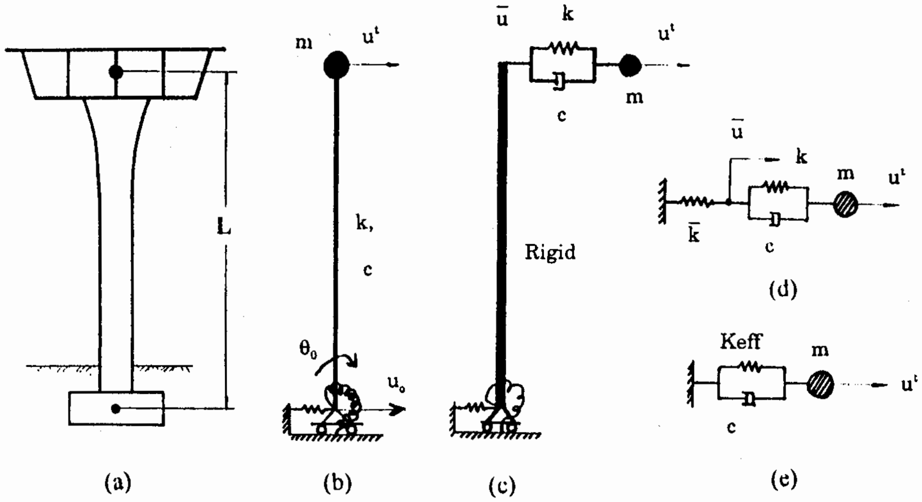


**Figure 2: Idealization of a Single Column Bent**

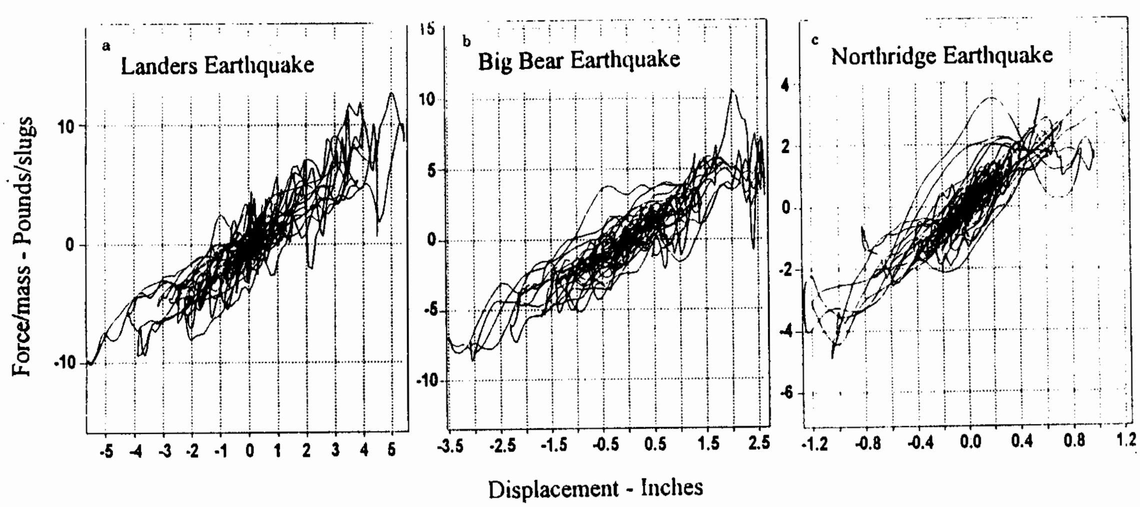


**Figure 3: Normalized Force-Displacement Hysteresis Loop for the Transverse Response of Bent 8**

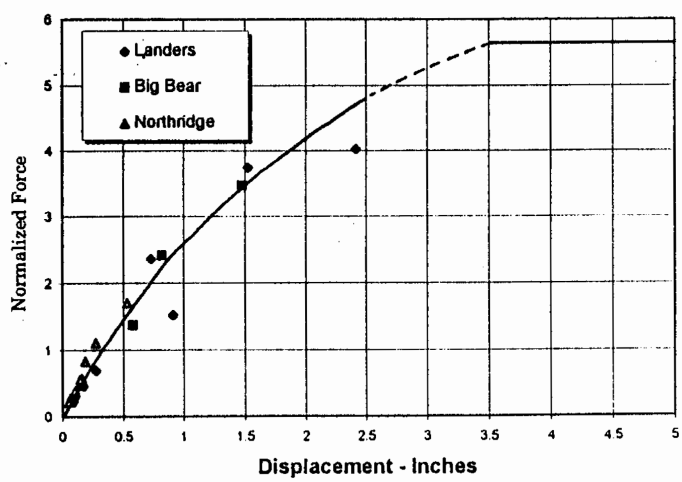


**Figure 4: Nonlinear Force-Displacement Relation Based on RMS Force-Displacement Pairs from Measured Data**

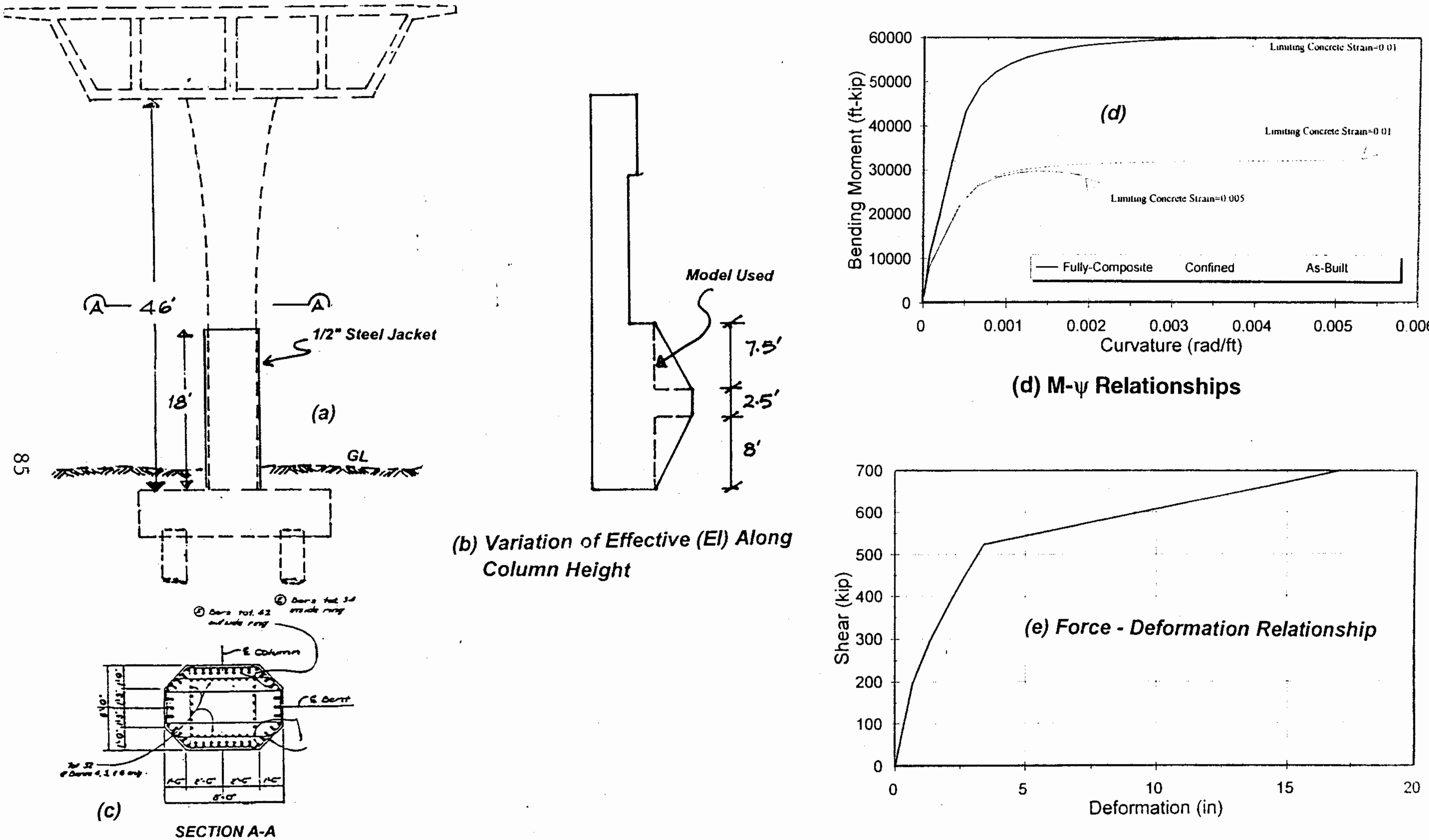


Figure 5: Component Behavior Characterization of the Retrofitted Column

# Impact of Adjacent Frames

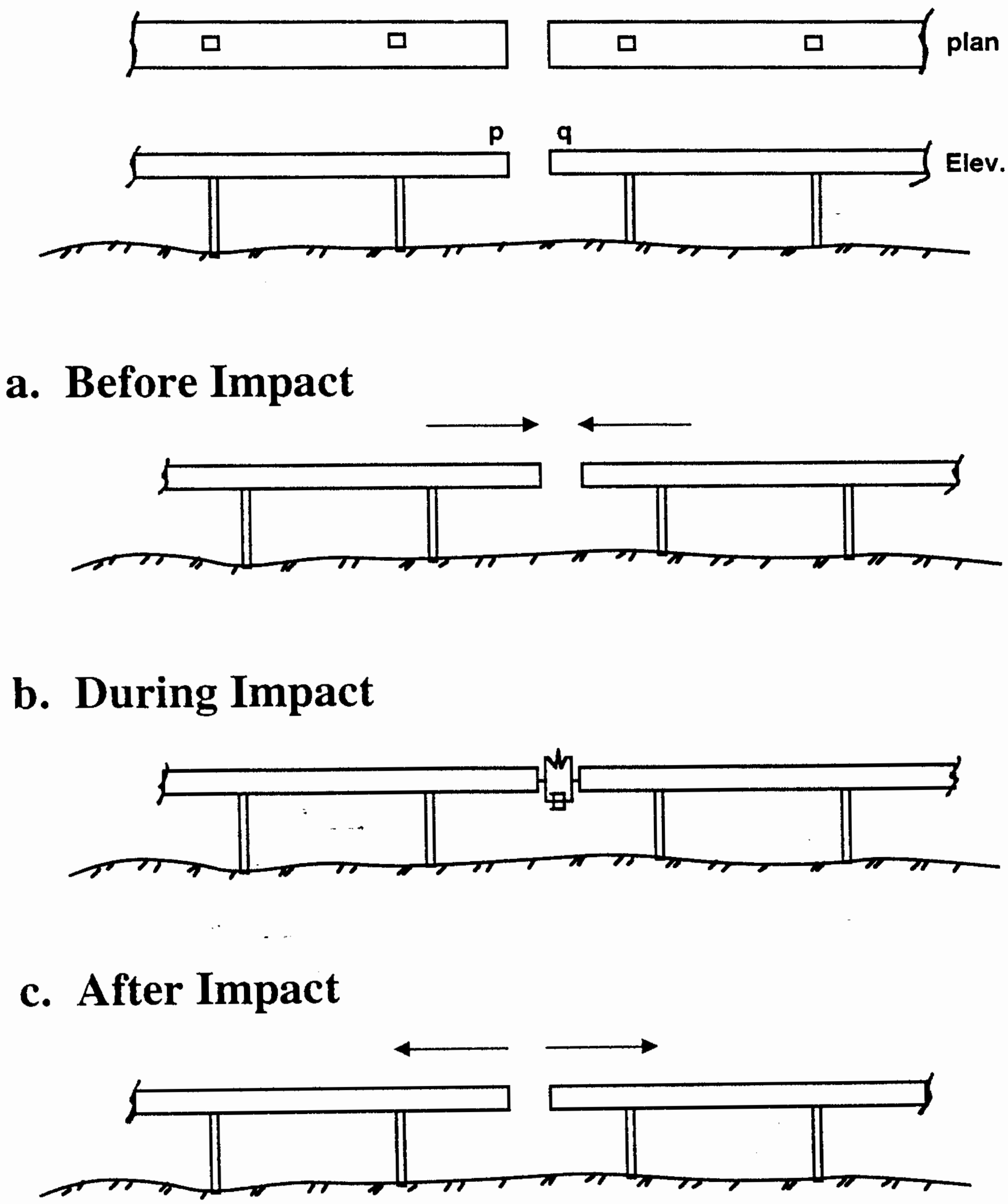


Figure 6: Modeling of Impact between Adjacent Frames

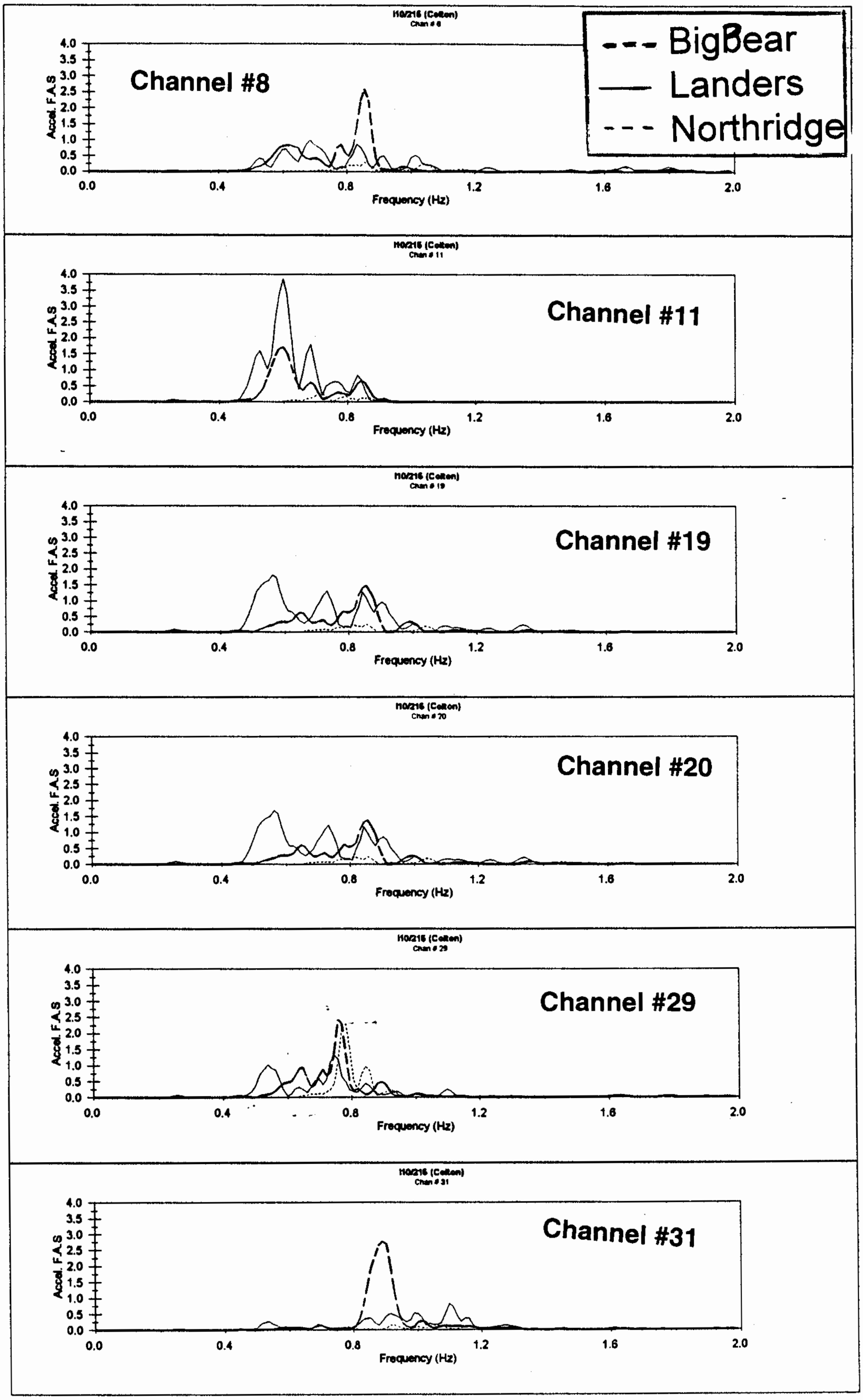


**Figure 7: Fourier Amplitude Spectra for the Three Earthquakes**

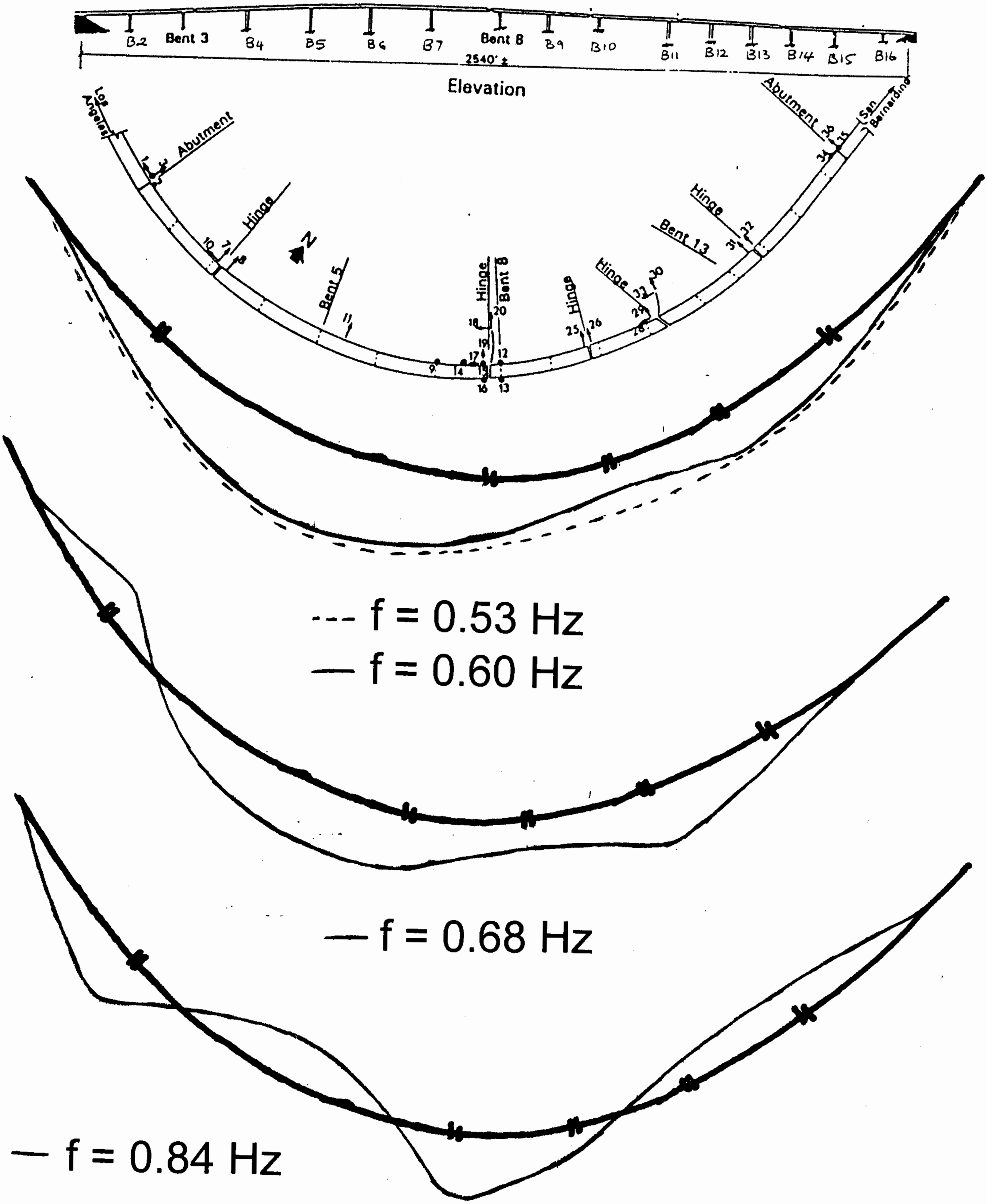


**Figure 8: Dominant Vibration Mode Shapes**

# CASE STUDIES OF 25 BUILDINGS SUBJECTED TO STRONG SHAKING IN THE NORTHRIDGE EARTHQUAKE

Fred Turner

Staff Structural Engineer
California Seismic Safety Commission

## ABSTRACT

This paper summarizes the key findings and recommendations from a Seismic Safety Commission publication titled *Northridge Buildings Case Studies Project* (SSC 94-06) and how it influenced the Commission's report titled *Turning Loss to Gain* (SSC 95-01). The paper also describes how the ground motions recorded during the 1994 Northridge Earthquake were characterized by engineers during short seismic evaluations of 25 buildings.

## INTRODUCTION

We are all fortunate that relatively few people were killed or injured in the January 17, 1994 Northridge Earthquake. While the media focused its attention on the event, the public policy arena in Sacramento was not nearly as engaged as it was after the 1989 Loma Prieta earthquake. This unusually mild reaction was puzzling considering that the Northridge event had easily an order of magnitude more population in the regions of MMI VIII or greater than the Loma Prieta Earthquake. Had the Northridge event occurred at another time of day, easily 10 times as many people would likely have been killed in collapsed buildings including some of the buildings that were later chosen as case studies by the Seismic Safety Commission. After the Loma Prieta Earthquake, the focus of public policy attention was on hundreds of new seismic safety laws proposed in Sacramento. In contrast, after Northridge, much of the focus was on the adequacy of building codes and practices.

Members of the public and the media raised serious questions about the ability of the building industry to design and construct reliable, earthquake-resistant buildings. Their concerns were influenced more by large economic losses than by the earthquake's actual or potential human losses. Since our building codes are intended to provide life safety and not economic protection from earthquakes, two schools of opinion emerged. School “A” felt that on the whole, buildings affected by the Northridge earthquake provided acceptable levels of risk as evidenced by the low life loss, which met the limited objectives of the building code seismic provisions. School "B" opined that Northridge was just a moderate earthquake that occurred at an opportune time and that both economic losses and the potential for life loss was unacceptable. On February 9, 1994, Governor Pete Wilson issued an Executive Order (W-78-94) asking that the Seismic Safety Commission examine the adequacy of building codes and practices.

This Executive Order set the stage for several information collection efforts by the Commission. The Commission knew it needed to base its assessment of the adequacy of building practices on more specific information than it had collected in the two months of hearings and interviews following the earthquake. So in May 1995, the Commission issued an emergency $242,000 contract to Rutherford and Chekene to gather information about the seismic performance of two dozen or so buildings in the region of strong ground shaking. Rutherford and Chekene’s team develop specific findings and recommendations for each building that would later influence the Commission’s general conclusions and recommendations about the adequacy of the state's building codes and practices.

The subject of this paper was funded in large part by a state general obligation bond, Proposition 122, that is a seismic retrofit program for state and local government buildings. This $300 million measure passed shortly after the Loma Prieta Earthquake in 1990. The Seismic Safety Commission is authorized to spend up to one percent of the bond fund to help improve seismic retrofit practices. The author manages this effort for the Commission.

Rutherford and Chekene and the Commission eventually selected 26 case study proposals that were significant from a public policy and structural engineering viewpoint. A special effort was made to include buildings retrofitted prior to the earthquake and instrumented buildings. Twenty-nine damaged and undamaged buildings in 25 case studies were eventually published, ten of which were retrofitted. Both large and small structural engineering firms that practiced in the region of damage were selected. Each had different levels of experience and expertise in earthquake engineering. The selection of individual buildings was influenced greatly by the limited availability of local firms to perform the work during the summer of 1994 since all local firms were involved in recovery efforts. Also some candidate buildings were not selected because building owners were reluctant or declined to allow their buildings to be included when told of the nature of the investigations. By August 1994, Rutherford and Chekene had hired 30 investigators in 18 separate firms to undertake case studies with individual budgets ranging from $2500 to $10,500 each. The Commission had hoped to complete the studies within two months, that is, by the end of September 1994.

## CHARACTERIZATION OF GROUND MOTION IN THE CASE STUDIES

Each case study investigator was asked to "summarize the effects of site geology, geotechnical improvements, soil failure, and response to ground motions and their impact on building response"(SSC 3032, 1994). However, since the Commission relied on each individual investigator to obtain ground motion records that were relevant to their buildings, the preliminary results were mixed. The Commission realized in late August that investigators were taking several different approaches to characterize ground motions. Some quoted peak ground accelerations, some spectral response acceleration values consistent with the predominant periods of vibration for the subject buildings. One investigator estimated "effective peak acceleration" using personal observations of downed chimneys and masonry walls. Several investigators stuck to simple building code approaches arguing that periods of vibration could not be estimated for their buildings.

So the Commission arranged a meeting in late August 1994 with Bill Holmes and Helen Ferner of Rutherford and Chekene, as well as Bruce Norton, Kevin Coppersmith, Geoff Martin, Joe Penzien, Tom Tobin, Tony Shakal, and Commissioners Lloyd Cluff, Paul Fratessa, and Jeff Johnson to get a meeting of the minds on how to characterize ground motion in both the Case Studies and the Commission's response to the Governor's Executive Order. More directly, an e-mail message to Tom Tobin suggested that:

> We need a knock-down bolted-door meeting to arrive at a consistent way to characterize ground motion parameters with engineering significance at each case study site. (Anon)

After some very frank discussions about the shortcomings of earthquake engineering ensued, this group agreed to reference linear elastic spectral response accelerations in the ranges of the first periods of vibration for each building. At the time, this seemed to be the direction that future building codes and practices would be taking. Dr. Tony Shakal of CSMIP was very helpful as he then directed his office to prepare response spectra from stations in the region of strong ground shaking.

Unfortunately some of the case studies had already been completed by the time investigators received SMIP's response spectra, so the final report still contains references to more distant records and peak values. However, the introduction to the case studies has much of the relevant response spectra available in late 1994, so readers can still piece together building performance using nearby records with a little effort.

Rutherford and Chekene described the difficulty of characterizing ground motions in their introduction to the Case Studies :

> Definition of ground motions at the various building sites proved to be the most vexing difficulty for the investigators. Only one case study building, the Holiday Inn on Orion Boulevard, was instrumented. Estimating the ground motion at a particular building site from a recorded motion several miles away is fraught with uncertainty. Ground conditions, topography, basin edge effects, and directionality may affect the earthquake record and duration at any building site in unpredictable ways. These concepts are further complicated by considerations of vertical accelerations and their phasing with the horizontal accelerations and the acceleration and velocity pulse shapes. California Division of Mines and Geology staff assisted the project by providing ground motion acceleration response spectra from the nearest available recording station for each of the case studies.
>
> In cases where the recorded ground motion is located very close to the case study building and the ground conditions are sufficiently similar that the investigator felt the recorded ground motion was representative of the motion at the case study building site, this record is included in the case study report. For the other buildings where the recorded ground motions are not very close to the case study building, the nearest ground motion is noted in the report and the response spectra are given in Figures in the Introduction.
>
> Table 1 indicates how close the nearest response spectra is to the case study building. Engineers can draw their own conclusions about the likely ground motions at any particular case study site based on proximity and ground conditions.
>
> The Area Map, shown in Figure 1, was prepared by the California Governor's Office of Emergency Services. This map indicates the location of the California Strong Motion Instrumentation Program recording stations and MMI contours in addition to the case study building locations. All except three of the buildings are located in areas with reported shaking intensities of MMI VIII or higher. (SSC 94-06)

The Van Nuys Holiday Inn, a seven-story reinforced concrete frame constructed in 1996 was instrumented by SMIP prior to the 1971 San Fernando Earthquake. Additional instruments were added after that event and worked well in the 1994 Northridge earthquake. However, there were no free-field sensors located near the site. This greatly hampered the ability of case study investigators to compare free-field ground motions with building response. As a further example:

> There were few free-field instruments in the immediate vicinity of damaged steel-frame buildings, so the levels and character of shaking experienced by these buildings are not well understood. (SSC 95-01)

**Table 1. Distance of Response Spectra to Case Study Building**

| Case Study | Building, Location | Recording Station | Distance from Case Study Building |
|---|---|---|---|
| 1.1 | Sherman Oaks Towers, Sherman Oaks | 322 | 0.9 |
| 1.2 | Saint John's Hospital and Health Center, Santa Monica | 538 | 1.8 |
| 1.3 | Precast Building, Van Nuys | 386 | 4.0 |
| 1.4 | Eight Story Concrete Shear Wall Building, Van Nuys | 386 | 0.3 |
| 1.5 | Retail Facility, Topanga Plaza, Canoga Park | C246 | 0.6 |
| 1.6 | Bullock's Department Store, Northridge Fashion Center, Northridge | C130 | 1.8 |
| 1.7 | Department Store, Northridge Fashion Center, Northridge | C130 | 1.0 |
| 1.8 | JFK Senior High School, Administration Building A, Granada Hills | Rinaldi | 0.3 |
| 1.9 | CSU Northridge Oviatt Library, Northridge | C130 | 1.3 |
| 1.10 | Santa Monica College Precast Concrete Parking Structure, Santa Monica | 538 | 1.0 |
| 1.11 | CSU Northridge Parking Structure C, Northridge | C130 | 1.5 |
| 1.12 | The Newhall Land & Farm Building, Santa Clarita | 279 | 1.6 |
| 1.13 | Ductile Concrete Frame Building, Sherman Oaks | 322 | 0.3 |
| 1.14 | Holiday Inn, Van Nuys | 386 | 4.0 |
| 1.15 | Concrete Tilt ups, Chatsworth | C130 | 3.6 |
| 2.1 | Moment Frame Building, Sherman Oaks | 322 | 2.5 |
| 2.2 | Division Office Building Moment Frame, Santa Clarita | 279 | 1.6 |
| 2.3 | Concentric Braced Frame, North Hollywood | C083 | 0.75 |
| 3.1 | Unreinforced Masonry Building, Hollywood | 303 | 1.9 |
| 3.2 | St. Monica's Parish Elementary School, Santa Monica | 538 | 0.9 |
| 3.3 | Unreinforced Masonry Building, Los Angeles | 655 | 3.1 |
| 3.3 | Unreinforced Masonry Building, Hollywood | 303 | 1.3 |
| 4.1 | Three Story Wood Apartment Building, Northridge | C130 | 2.2 |
| 4.2 | Multi Story Wood Condominium, Sherman Oaks | 322 | 1.7 |
| 4.3 | Two Story Damaged Dwelling, Granada Hills | Rinaldi | 3.6 |
| 4.4 | One Story Retrofitted Dwelling, Hollywood | 303 | 1.1 |

Table 1. Distances (miles) from Ground Motion Recording Stations to Case Study Buildings

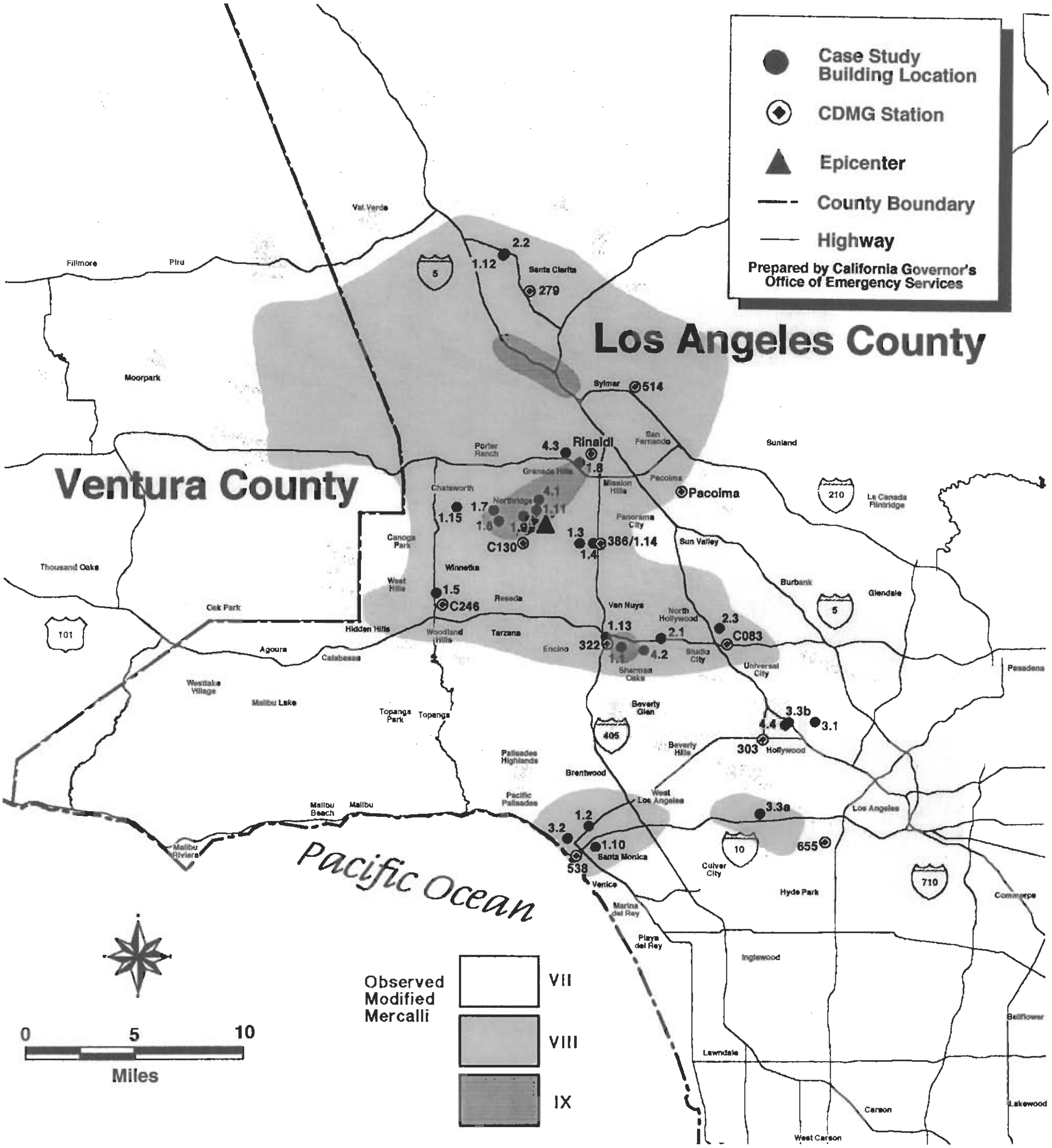


Figure 1. Area Map of Case Study Buildings, ground motion recording station locations, and Observed Modified Mercalli Intensity (Prepared by Cal OES)

The lack of free field sensors or "reference stations" is perhaps one of the best lessons SMIP and other strong motion instrumentation programs can capitalize on from the Northridge earthquake. The Commission later recommended in its response to Governor Wilson's Executive Order, *Turning Loss to Gain* , that:

> SMIP give high priority to establishing a network of reference stations to measure ground motions in major urban areas of California. (SSC 95-01)

Since then, SMIP increased its emphasis on installing reference stations and submitted a TriNet proposal to FEMA that, if funded, would further accelerate such installations.

Still other case study investigators encountered problems securing ground motion records in time to compare them with actual building response. For example, the John F. Kennedy High School, which suffered only nonstructural damage in the 1971 San Fernando earthquake, sustained major structural damage in 1994. The nearest free field station was the LA DWP Rinaldi station, but the investigator was not aware of response spectra's availability until late September, well after the case study was written. Other instruments funded by USGS, NSF and private owners also had data that were not processed and disseminated in a timely manner. As a result of these and other similar experiences, the Commission recommended that:

> SMIP should exert leadership by organizing a workshop involving the other operators of strong-motion instrument networks in California to coordinate the deployment and operation of these networks.
>
> As a result of these workshops, SMIP should compile a list of all strong-motion instruments and their locations in the state and find ways to improve the overall performance of the systems. Furthermore a mechanism should be developed to provide the processed data from earthquakes in a timely manner. These tasks should be completed by July 1995.
>
> Public funds should not be used for the purchase, deployment, or upgrading of strong-motion instrument networks operated by private organizations unless there is a plan for the maintenance of the instruments and an agreement for the timely release of data to the public. (SSC 95-01)

In February 1995, SMIP met with representatives of instrumentation networks to start on this effort to improve communications. In spite of SMIP's limited resources and the shortcomings of other networks, the Commission strongly endorsed SMIP as a valuable part of California's effort to reduce the risk from earthquakes:

> The Commission believes that SMIP proved its worth during this earthquake and its aftermath. Within a day of the main shock, SMIP had issued a Quick Report... SMIP also processed data from five stations during the first week of February. Processed data for more than 70 stations were released by December 1994. The timeliness and quality of these data were extremely valuable. (SSC 95-01)

Two case study investigators independently suggested that the effects of high modes of vibration may have played roles in damage to buildings. The Sherman Oaks Tower, a 12 story concrete shear wall building had only minor damage to reinforcing steel at the end of a wall just below the sixth floor. Similarly the Holiday Inn in Van Nuys experienced its most severe damage in columns at the fourth and fifth floors. The investigator noted that the lateral force capacity at the fourth floor was less than at lower floors and surmised that the second mode may have contributed significantly to the building's response placing a high shear demand in the upper floors.

These case studies raise questions about analytical techniques and lateral force distribution assumptions that certainly merit further investigation. (SSC 94-06)

The Commission in 1995 drew several conclusions regarding Northridge Earthquake ground motions from its case studies and other Executive Order investigations:

Although some recorded accelerations in this earthquake were especially high, most spectra generally agreed with those recommended by site-specific geotechnical studies as the basis for the design of special structures. Similar response spectra have been calculated from data from numerous earthquakes since the 1971 San Fernando earthquake and should be expected in future events.

Because of damage from this earthquake, questions have been raised concerning the adequacy of the building code's definition of the forces that earthquakes can impose on buildings. Code writers and designers know that code spectral values will likely be exceeded in large earthquakes and that this was anticipated when the code was written.

The recorded data from the Northridge earthquake are still being evaluated and subject to different interpretations. Strong motion instruments also were not located in many areas that suffered the most severe damage. Generally speaking, the motions recorded near the Northridge epicenter were compatible with those used as the basis for the code, but the motions exceeded those assumed in the code in some cases. At some locations, particularly in the near-source area and in areas with unique local geology, shaking exceeded the assumptions underlying design values in the short-to mid-period range. This shaking appears to have affected low- and mid-rise buildings and caused response in higher modes of vibration for tall buildings. Velocity- and displacement-sensitive structures also may have been affected by the velocity pulses described earlier. Near-source and local geologic effects must be considered in the design of structures. There is no compelling evidence that changes to the code's assumed force levels are necessary. However, changes are necessary regarding the treatment of the effects of near-source and local geologic conditions.

## OBSERVATIONS AND RECOMMENDATIONS ABOUT BUILDING PRACTICES

By and large, poor quality in design and construction was the biggest source of earthquake damage in the Northridge Earthquake, as observed in other earthquakes throughout the world. Only a few building case study investigators suggested that low original design force levels were primary contributors to damage. The case studies provide first-hand examples of the adverse effects of, among other things, design omissions, questionable installation practices, and incomplete load paths:

Case Study 1.14, Holiday Inn, presents an example of reinforcing in a beam-column joint that was specified on the original plans but was apparently not present in a column damaged by the Northridge earthquake. Improved inspection practices for construction could significantly reduce occurrences of this type of omission.

Case Study 1.15, Three Tilt-Up Buildings, indicates that a variety of wood connection hardware details and installation practices, including gaps between wood framing, may have contributed to partial collapses.

Assumptions that were made by the designer of a URM retrofit (presented in Case Study 3.1) were apparently never verified in the field and contributed to the partial

> collapse of this building. Improved plan checking, field inspection, and observation of construction by the designer could have caught this discrepancy before the retrofit was completed. This is also an example of damage caused by building elements that are not well tied together.
>
> Case Study 3.3, Two Unreinforced Masonry Buildings, shows that the lack of URM wall braces and inadequate veneer ties were responsible for out-of-plane failure of wall elements. This condition was further exacerbated by poor mortar quality in the second building. Inadequate out-of-plane anchorage was responsible for damage to another URM, presented in Unretrofitted URM Saint Monica's Parish Elementary School, Case Study 3.2.
>
> Case Study 1.6, Bullock's Department Store, Northridge Fashion Center, presents a graphic example of what can happen to a partially retrofitted building with an incomplete load path.

The case studies also provide examples of damage that the Structural Engineers Association of California and the International Conference of Building Officials have since begun to address with changes to the building code. These include steel moment frame joints, steel braced frames, and tiltup wall-to-roof connections.

Several building systems suffered from deformations during the Northridge earthquake far in excess of that estimated by their original designers. In some cases these deformations caused failure in parts of the buildings that were not intended to act as part of the earthquake force-resisting systems. In effect, these large deformations were not compatible with the building vertical load carrying systems and caused collapse or otherwise incipient situations. Recent code changes will require designers to make more realistic estimates of building deformations and to protect from collapse those parts of buildings such as concrete columns that are susceptible to incompatible deformations.

Many building components have not been thoroughly tested for earthquake resistance. The Northridge event served to confirm or call their future use into question. For example, observations of concrete walls indicate that staggering horizontal steel splices is most likely not a necessary expense. But sliding failures at horizontal construction joints in non-load bearing concrete walls indicated a need for a subsequent change in the code. Case studies of wood buildings provide clear examples of the generally unacceptable performance of drywall panels.

## SUMMARY AND CONCLUSIONS

The Northridge Buildings Case Studies provide an anecdotal, but nevertheless informative summary of the many ways buildings performed and of different ways engineers evaluate buildings after earthquakes. The Seismic Safety Commission relied heavily on the specific examples in case studies to substantiate its general recommendations and overall conclusions regarding the adequacy of the building codes and practices as stated below:

> At the heart of Governor Wilson's executive order is the question: "Is the building code safe enough for earthquakes?" With few notable exceptions, the UBC provides an adequate level of life safety for new construction as long as the code is strictly applied during the design and construction of buildings and as long as the code is enforced with thorough plan reviews and inspection. As long as the current performance objectives are acceptable, the building code itself is not in need of a major overhaul, but far more attention to strict adherence to the code and the elimination of shoddy design and construction is clearly needed for earthquake-safe

> buildings. Recent changes to the earthquake requirements in the building code have not been adequately substantiated and do need to be more comprehensively verified in the future.
>
> In light of the extensive, albeit non-life-threatening, damage to modern buildings, the state should more actively support efforts to develop future codes, establish acceptable levels of earthquake risk in buildings, and develop design guidelines for meeting seismic performance objectives. (SSC 95-01)

The Seismic Safety Commission's actions reaffirmed the importance of California's Strong Motion Instrumentation Program. As future building codes become more transparent with a greater scientific basis, our collective reliance on SMIP to calibrate both ground motions and system response will undoubtedly grow.

## ACKNOWLEDGMENTS

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# STRONG-MOTION RECORDS FROM BUILDINGS DAMAGED IN EARTHQUAKES

M.J. Huang, P.K. Malhotra and A.F. Shakal

California Strong Motion Instrumentation Program
Division of Mines and Geology
California Department of Conservation

## ABSTRACT

Strong-motion recordings have been obtained in several buildings damaged by earthquakes. For post-earthquake inspection of buildings, strong-motion records can provide important information on the integrity of the building structure. This paper examines records from two concrete and four steel buildings damaged in earthquakes. The characteristics of the building response that may be indicative of the structural damage are identified from the records. For these buildings, the fundamental period, the maximum drift between the roof and the base, and the maximum base shear are all higher than the corresponding design values. In addition, high-frequency spikes and highly nonlinear response can be seen in some of the records.

## INTRODUCTION

Strong-motion records have been obtained from many buildings in California. Prior to the 1994 Northridge earthquake there were only a few records obtained in damaged buildings. The Northridge earthquake has increased the number substantially. In over 150 instrumented buildings throughout the Los Angeles area the recorded roof level accelerations exceeded 0.25 g during the Northridge earthquake. Some of these buildings were extensively instrumented by the California Strong Motion Instrumentation Program (CSMIP) and others have minimal instruments as required by the building code (Shakal et al., 1994 and 1996). Many of the buildings experienced high levels of structural response and some suffered structural damage. For most concrete buildings, the structural damage is visible and strong-motion records are not needed to determine that the building suffered damage. On the other hand, damage to steel buildings is mostly hidden unless the structure is out of plumb or has collapsed. Inspection of the steel members and connections requires removal of fireproofing material and non-structural elements. The cost can therefore be very high if many of the connections need to be inspected. In such cases, strong-motion records may be useful in providing information on whether the structure was damaged and where to start the inspection.

Table 1 lists 15 instrumented buildings that sustained structural damage in earthquakes. These buildings include 3 concrete and 12 steel buildings. All of the buildings were damaged in the Northridge earthquake except the Imperial County Services Building which was damaged in the 1979 Imperial Valley earthquake. The first two concrete buildings were extensively instrumented by CSMIP and the other 13 were lightly instrumented by the owners as required by the building code. Only three of the 13 code buildings have both the base and the roof records from the Northridge earthquake. The list of the buildings in Table 1 is based on what the authors know at this time from either available publications or personal communication, and it is by no means complete. The number of buildings will certainly increase as more results of post-Northridge inspections of steel buildings become available.

**Table 1. Summary of Recorded Data from Damaged Buildings**

| Building | No. of Stories | Building Direction 1 | | | | Building Direction 2 | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Base Accel (g) | Roof Accel (g) | Roof Drift (cm)* | $T_1$ (sec) ** | Base Accel (g) | Roof Accel (g) | Roof Drift (cm)* | $T_1$ (sec) ** |
| **Concrete Buildings** | | | | | | | | | |
| Imperial County Services(1979) | 6 | 0.33 | 0.46 | 18 | 2.0 | 0.34 | 0.58 | 8 | 1.2 |
| Van Nuys - Hotel | 7 | 0.47 | 0.59 | 23 | 2.0 | 0.39 | 0.56 | 23 | 2.0 |
| Northridge - Roscoe #1 | 7 | 0.42 | 0.58 | 24 | 1.4 | 0.39 | 0.39 | 30 | 1.3 |
| **Steel Buildings** | | | | | | | | | |
| Woodland Hills - Oxnard #4 | 13 | 0.43 | 0.33 | 33 | 3 | 0.32 | 0.24 | 27 | 2.5 |
| Woodland Hills - Oxnard #5 | 11 | - | 0.33 | nd+ | | - | nr++ | | |
| Woodland Hills - Canoga #1 | 17 | - | 0.39 | 43 | 4.5 | - | 0.23 | 17 | 4 |
| Woodland Hills - Canoga #2 | 17 | - | 0.49 | 45 | 4.6 | - | 0.26 | 16 | 4 |
| Los Angeles - Olympic #1 | 9 | - | 0.69 | 15 | ~2 | - | 0.51 | 32 | 2 |
| Los Angeles - Olympic #2 | 11 | - | 1.07 | 27 | 1.6 | - | 0.67 | 32 | 1.6 |
| Los Angeles - Olympic #4 | 12 | - | 0.55 | 39 | 1.8 | - | 0.38 | 16 | 2.2 |
| Los Angeles - Wilshire #7 | 14 | - | 0.29 | 28 | 2.6 | - | 0.34 | 17 | 2.4 |
| N. Hollywood- Lankershim #1 | 7 | - | 0.33 | 20 | 2.2 | - | 0.30 | 29 | 2.2 |
| Sherman Oaks - Ventura #7 | 22 | - | 0.46 | 34 | 4.5 | - | 0.34 | 29 | 4.5 |
| Encino - Ventura #1 | 17 | 0.54 | nr | | | 0.54 | nr | | |
| Encino - Ventura #12 | 20 | 0.41 | >0.6 | nd | 2.5 | 0.46 | >0.5 | nd | 2.2 |

* - maximum roof drift is estimated from the roof record for the building without the base record.
** - $T_1$, largest value of the fundamental period obtained from the record
\+ - nd, record has not been digitized
\+\+ - nr, no roof record

To study the feasibility of identifying structural damage from strong-motion records, two concrete and four steel buildings from Table 1 are discussed herein. Characteristics of the building response, such as building period, base shear, drift, damping, high-frequency spikes or abnormal response that may be indicative of structural damage, are extracted from the records for these six buildings. The Imperial County Services Building has been studied before and the other five buildings were studied in the SAC project (SAC, 1995) and the Seismic Safety Commission (SSC) project (SSC, 1996). The processed data for these buildings are available from CSMIP (Darragh et al., 1994 and 1995).

## CONCRETE BUILDINGS

### El Centro - Imperial County Services Building

The Imperial County Services Building, a 6-story concrete frame and shear wall building, suffered significant structural damage during the 1979 Imperial Valley earthquake (Figure 1). The building was demolished after the earthquake and replaced with a 2-story steel frame building. At the time of the earthquake, the building was instrumented by CSMIP with 13 sensors in the building and three sensors at a reference free-field site. This is the first case of an instrumented building sustaining significant structural damage.

Figure 2 shows the locations of the sensors in the building (Rojahn and Ragsdale, 1980). The earthquake resistance of the building was provided by shear walls in the transverse (N-S) direction and by moment-resistant frames in the longitudinal (E-W) direction. There were four interior shear walls below the second floor while the only shear walls above the second floor were the exterior walls. During the 1979 earthquake, all the beams and columns on the first story were damaged with the most severe failure occurring in the four columns at the east end of the building, as shown in Figure 1. More detailed description of the damage is given in the ATC-9 report (ATC, 1984)

The strong-motion records from this building were studied by numerous investigators (e.g., Housner and Jennings, 1982; Rojahn and Mork, 1982; Kreger and Sozen, 1989; Mau and Revadigar, 1994). The acceleration records in the longitudinal direction are shown in Figure 3. The records show that the response between 5 and 7 seconds is dominated by the second mode with a period of about 0.3 seconds. Between 7 and 11 seconds, the building responded with a period of about 1.6 seconds. In addition, high-frequency spikes appear at near 11 seconds and can be seen in all the upper floor records, in both directions. The time near 11 seconds has been interpreted as the time when the columns failed and the building dropped (Housner and Jennings, 1982; Rojahn and Mork, 1982). After the column failure, the building period was lengthened further to nearly 2 seconds. The building period from pre-1979 ambient measurements was 0.65 seconds (Rojahn and Mork, 1982).

The base shear force, estimated from the acceleration records, has a maximum value of about 0.24W for the east-west direction, which is about 4 times the design base shear (0.06W). The absolute displacements, obtained from the accelerations in Figure 3, are shown in Figure 4. These displacements are dominated by the ground motion with periods longer than 3 seconds. The building response is indicated by the relative displacements between the upper floors and the ground floor, which have a period of 1.6 to 2 seconds. The drift can be computed by differencing the displacements at various floors. The maximum drift is about 18 cm, 0.7% of the building height, between the roof

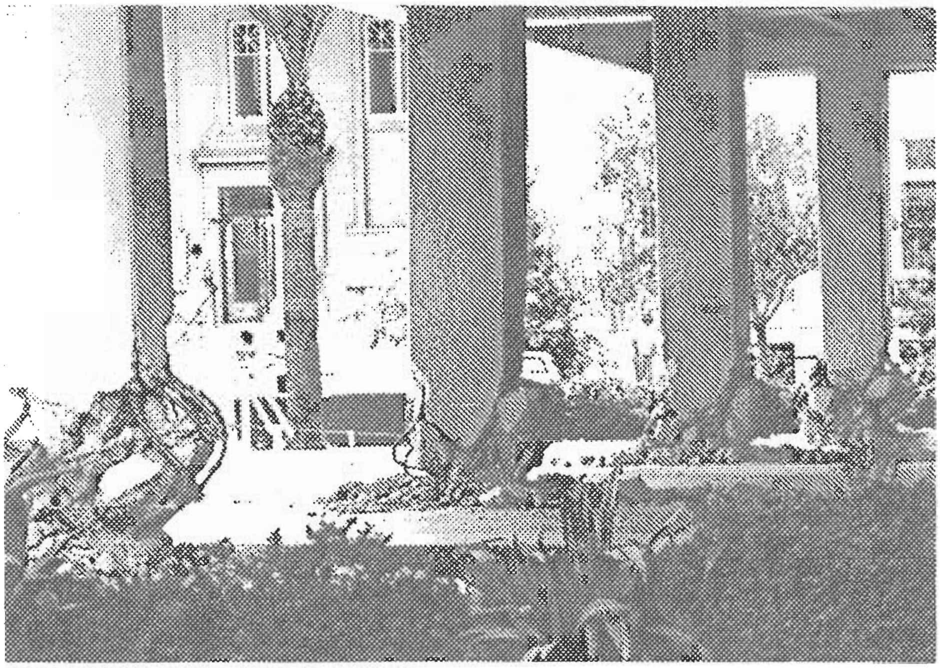


Fig. 1. Line of columns that failed at east end of the Imperial County Services Building during the 1979 Imperial Valley earthquake.

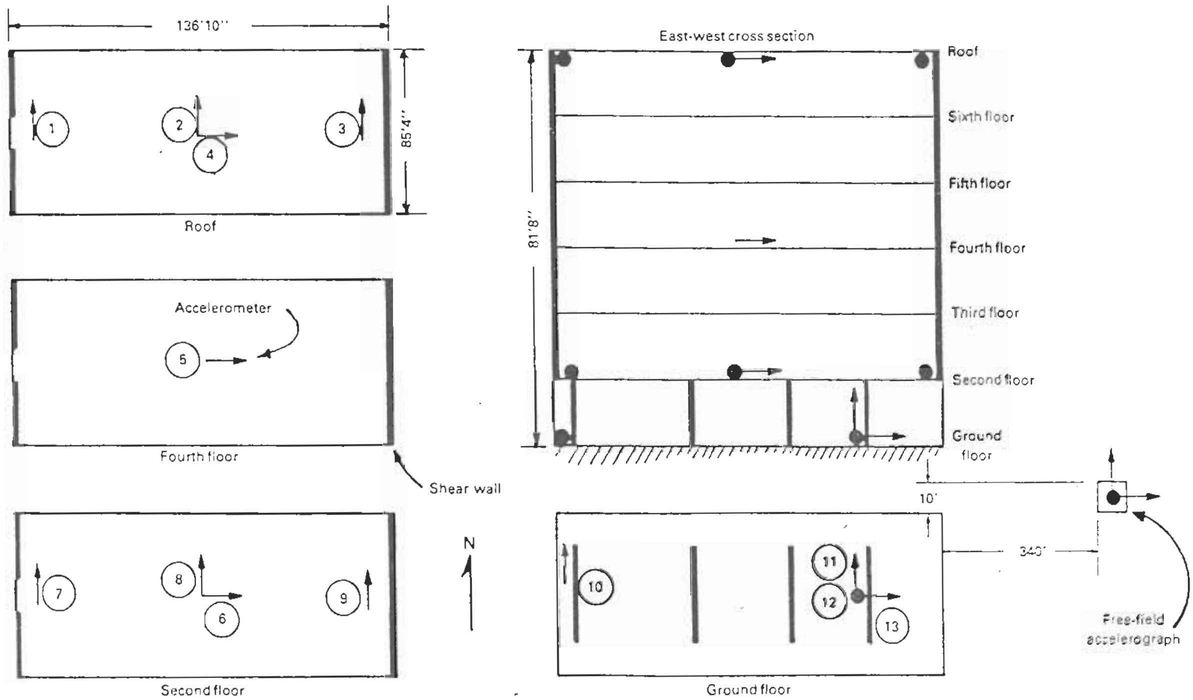



Fig. 2. Locations of accelerometers in Imperial County Services Building and reference free-field site during the 1979 Imperial Valley earthquake (Rojahn and Ragsdale, 1980).

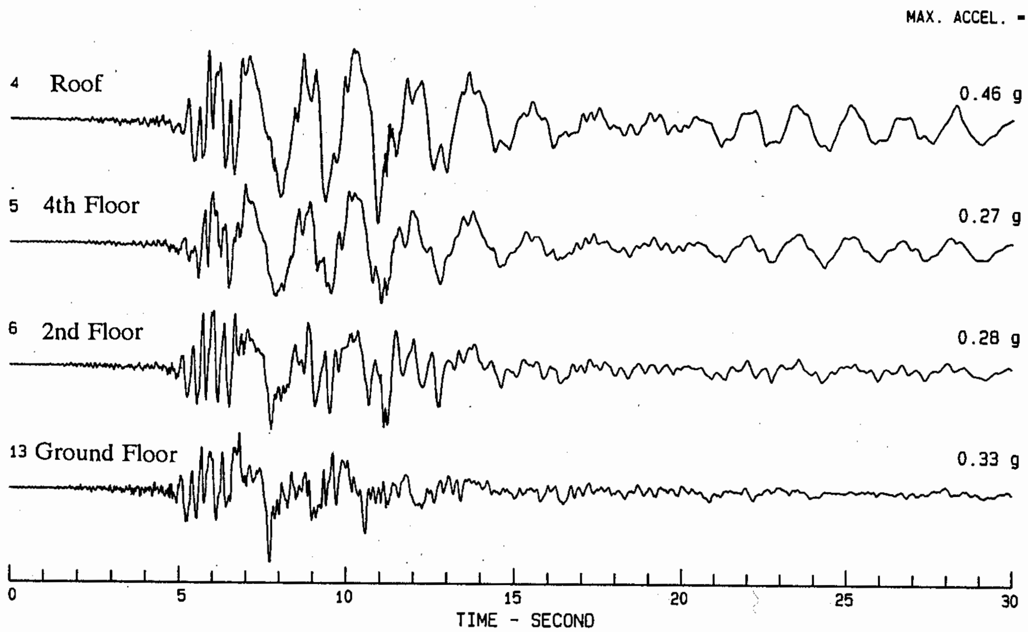



Fig. 3. Accelerations in the longitudinal (E-W) direction recorded at the Imperial County Services Building during the 1979 Imperial Valley earthquake. Failure of columns is inferred to have occurred at about 11 seconds.

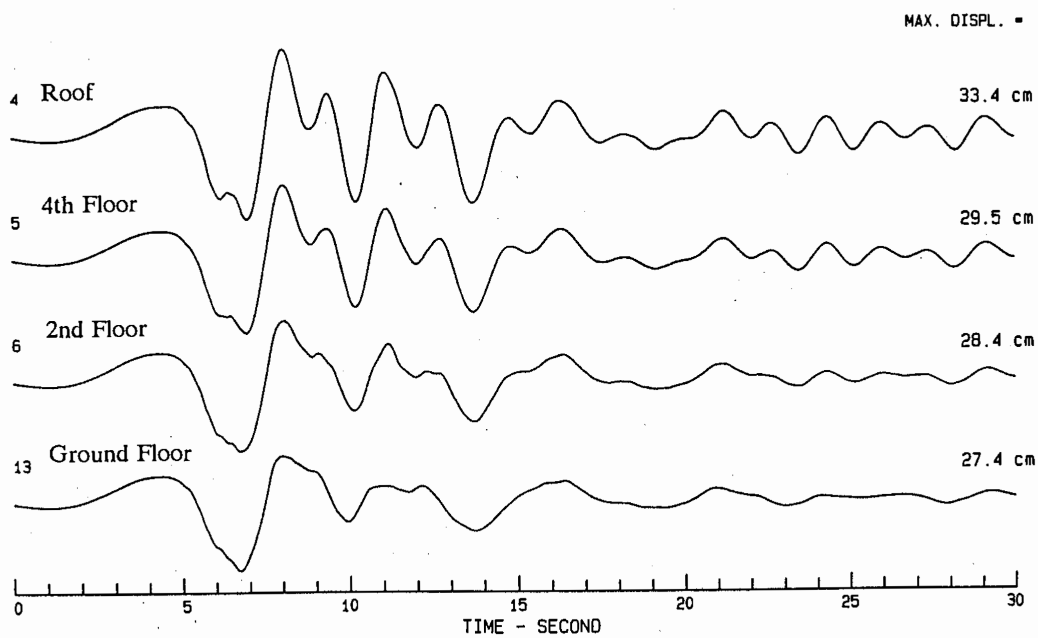



Fig. 4. Displacements corresponding to the accelerations in Fig. 3.

and the ground floor, and is about 8 cm, 1.8% of the story height, between the second and the ground floors. These drifts are larger than the 0.5% code limit

In summary, the records in the longitudinal direction indicate that: (a) the maximum base shear was 4 times the design base shear; (b) the building period was lengthened during the earthquake shaking, up to 3 times the ambient period; (c) large drift occurred in the first story; and (d) high-frequency spikes can be seen in the records. These features imply that the building response was highly nonlinear, and in the absence of inspection or visible damage they may suggest that structural damage might have occurred.

## Van Nuys - 7-story Hotel

A hotel in Van Nuys, a 7-story concrete frame structure, suffered structural damage during the 1994 Northridge earthquake (Figure 5). The building also suffered minor structural damage in the 1971 San Fernando earthquake. It is the closest instrumented building to the epicenter of the 1971 earthquake and was later instrumented more extensively with 16 sensors by CSMIP. Several distant earthquakes including the 1987 Whittier, the 1992 Landers and the 1992 Big Bear earthquakes were recorded at this building. The recorded peak accelerations in the longitudinal direction from these earthquakes are listed in Table 2. The Northridge shaking is the strongest and had the largest drift, as shown in the table. After the Northridge earthquake the building was repaired and strengthened with new concrete shear walls.

**Table 2. Summary of Recorded Accelerations, Drifts and Fundamental Periods from Several Earthquakes for the Van Nuys 7-story Hotel (Longitudinal Direction)**

| | Max. Base Acceleration (g) | Max. Roof Acceleration (g) | Max. Drift Roof-Base (cm) | Fundamental Period (second) |
|---|---|---|---|---|
| Pre-1971 Ambient Measurement+ | - | - | - | 0.52 |
| 1971 San Fernando (M6.5, d*=20 km) | 0.14 | 0.32 | 7.8 | 1.3 |
| Post-1971 Ambient Measurement+ | - | - | - | 0.70 |
| 1987 Whittier (M6.1, d=41 km) | 0.14 | 0.17 | 2.8 | 1.1 |
| 1992 Landers (M7.5, d=187 km) | 0.04 | 0.13 | 3.2 | 1.2 |
| 1992 Big Bear (M6.6, d=152 km) | 0.02 | 0.06 | 1.6 | 1.2 |
| 1994 Northridge (M6.7, d=7 km) | 0.45 | 0.58 | 23.0 | 1.5 - 2.0 |

\+ from Cloud and Hudson (1975)
* d - distance from the epicenter

The sensor locations and the structural system for the building are shown in Figure 6. The earthquake resistance of the building in each direction was provided by perimeter column-spandrel beam frames and interior column-slab frames. The structural damage during the Northridge earthquake was mainly in the longitudinal perimeter frames. As shown in Figure 5, the most severe damage occurred in the columns just below the fifth floor spandrel beam on the south side of the building. More detailed description of the damage is given in the Earthquake Engineering Research Institute (EERI) Reconnaissance Report (EERI, 1995) and in the Seismic Safety Commission Case Studies Report (SSC, 1996).

The Northridge records from this building have been used to study analytical techniques by Islam (1996) in the SSC Case Studies project, and by De la Llera and Chopra (1995) in a CSMIP-funded study. The records indicate that the building experienced significant torsional response. Figure 7 shows the accelerations recorded at the roof, the 6th, the 3rd, and the ground floor levels in the longitudinal (E-W) direction. It can be seen from these records that the second mode response with a period of about 0.4 seconds is significant and determined the peak accelerations for all floors. The high-frequency spikes seen in the Imperial County Services building records (Fig. 3) do not appear in these records. This may be due to the fact that the building did not drop vertically and the sensors were not close to the locations of concrete spalling below the fifth floor.

The fundamental mode response is more apparent in the displacement records, as shown in Figure 8. The fundamental period of the building was lengthened from about 1.5 seconds in the first 10 seconds of the record to about 2 seconds later. The inter-story drift ratio is 1% for the first story and 1.9% for the second story. The maximum drift between the roof and the ground floor is about 23 cm, 1.2% of building height, which occurred at 9.34 seconds. It has been interpreted by Islam (1996) that many structural elements may have yielded at approximately 4 to 5 seconds and the most severe damage to the columns may have occurred at about 9 seconds, which corresponds to the time of maximum acceleration. After the columns were damaged, the building period was lengthened to nearly 2 seconds. The maximum base shear from the first mode is about 0.20W which is about 5 times the design base shear (0.04W, according to 1967 UBC).

The building is quite unique in that various levels of structural response, ranging from ambient vibration to strong earthquake response, have been recorded, as summarized in Table 2. One of the important results from studies of these data is the lengthening of the building period during earthquakes relative to the ambient values. For the longitudinal direction, the period was lengthened to three times the ambient period in the San Fernando earthquake, and to four times in the Northridge earthquake.

In summary, the Northridge records in the longitudinal direction indicate that: (a) the maximum base shear was 5 times the design base shear; (b) the building period was lengthened during the earthquake shaking, from 3 times to 4 times the ambient period; and (c) large story drifts occurred.

Fig. 5. Partial south elevation of Van Nuys 7-story Hotel showing damage to columns (indicated by an arrow) below fifth floor spandrel beams during the 1994 Northridge earthquake.

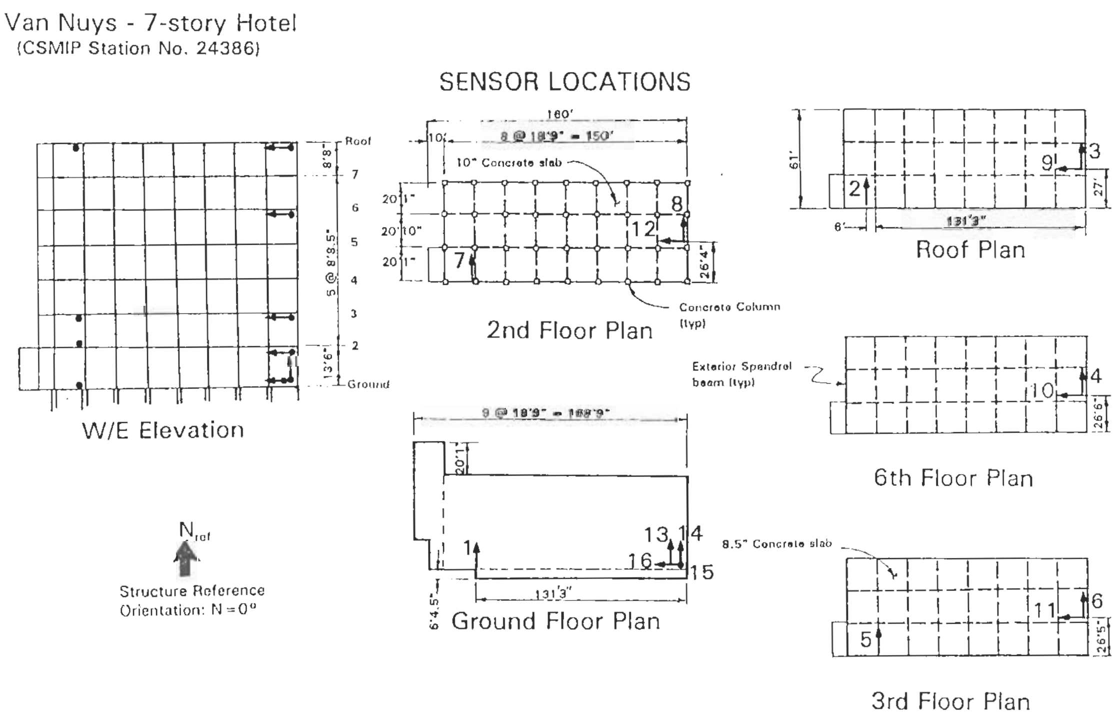



Fig. 6. Locations of accelerometers in Van Nuys 7-story Hotel.

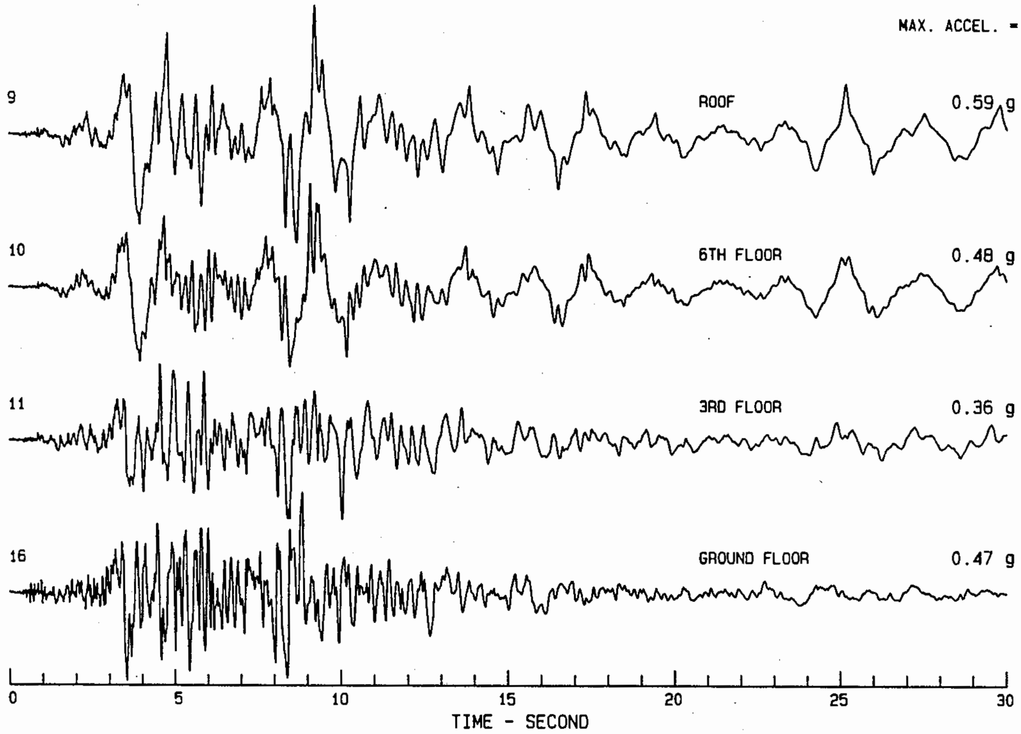



Fig. 7. Accelerations in the longitudinal (E-W) direction recorded at the Van Nuys 7-story Hotel during the 1994 Northridge earthquake. Failure of columns occurred between the 4th and 5th floors.

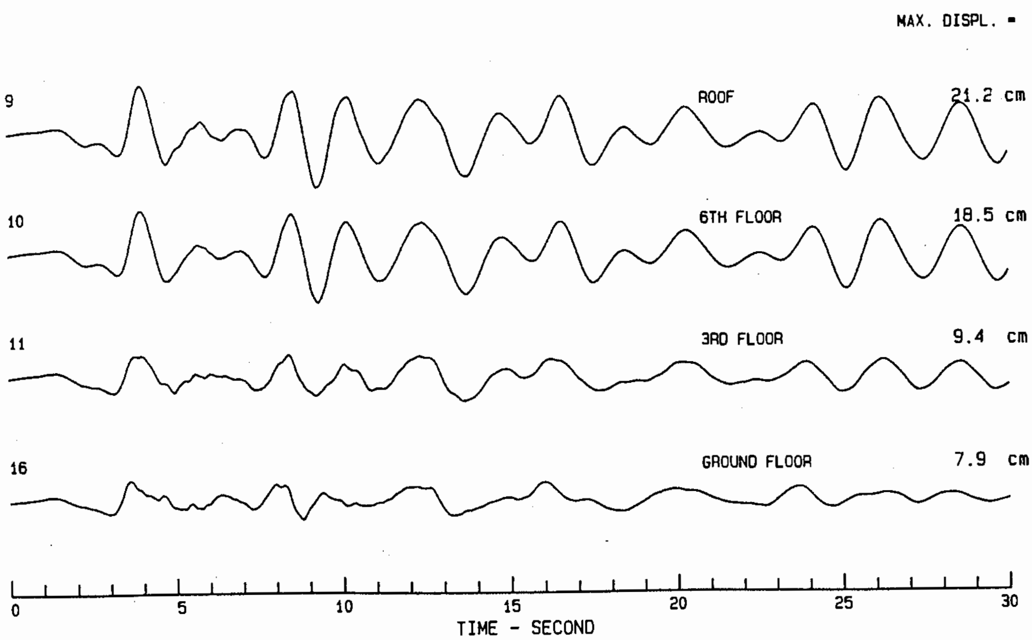



Fig. 8. Displacements corresponding to the accelerations in Fig. 7.

## STEEL BUILDINGS

### Woodland Hills - Oxnard #4 Building

The Oxnard #4 Building in Woodland Hills, built in 1976, is a 13-story office building. The earthquake resistance of the building is provided by steel moment frames on the perimeter. The building is 160′ square in plan. After the Northridge earthquake, 54 of the 551 inspected joints were found to be damaged (Uang et al., 1995a). The west side frame that provided the resistance in the reference north-south direction has more fractured joints than other three frames. Detailed locations of the fractured joints are given in the report by Uang et al. (1995b).

The building was instrumented with three accelerographs by the owner as required by the Los Angeles Building Code. They were located at the basement, the 6th floor and the roof level. Figure 9 shows the acceleration records in the reference N-S direction. The building fundamental period can not be easily determined by a visual inspection of the acceleration records due to prominent higher mode response in the roof record. Only the first 22 seconds of the roof record could be digitized in the initial digitization. The corresponding displacement records are shown in Figure 10. The building's largest response, with a period of about 3 seconds, occurred between 4 and 8 seconds. The ground shaking was so energetic in the beginning that the structural members apparently yielded right at the beginning. From the computer modelling of the structure, Uang et al. (1995) concluded that a significant number of panel zones at the joints yielded and that the panel zones were a major source of energy dissipation during the Northridge earthquake. The total drift between the roof and the basement, computed from the displacement records, has a maximum value of 33 cm in the reference N-S direction, which is about 0.6% of building height. Design of this steel building was probably controlled by the drift limit, 0.5%, rather than by the force. The maximum base shear is about 0.12W which is about 4 times the code-specified force (0.03W).

### Woodland Hills - Canoga #1 and #2 Buildings

The Canoga #1 (West Tower) and Canoga #2 (East Tower) buildings in Woodland Hills are two identical 17-story office buildings. They were designed in 1984. The lateral force resistance is provided by four two-bay steel moment frames. Three moment frames are located on the building perimeter and the fourth moment frame is located one bay from the north face of the building. Although the two towers have identical structural systems, the damage to the East Tower was more severe than the West Tower (Anderson, personal communication). For the East Tower, all fractures occurred at the connections in the N-S frames, with 23 occurring in the frame on the west face and 6 in the frame on the east face (Anderson and Filippou, 1995). The majority of the cracked connections were located between the 12th and the 15th floor levels. Yielding of the beam flanges was also found near the connections at the 13th floor level. A check for vertical plumb of the elevator shaft revealed that the building had a permanent deformation of approximately six inches to the North at the roof level.

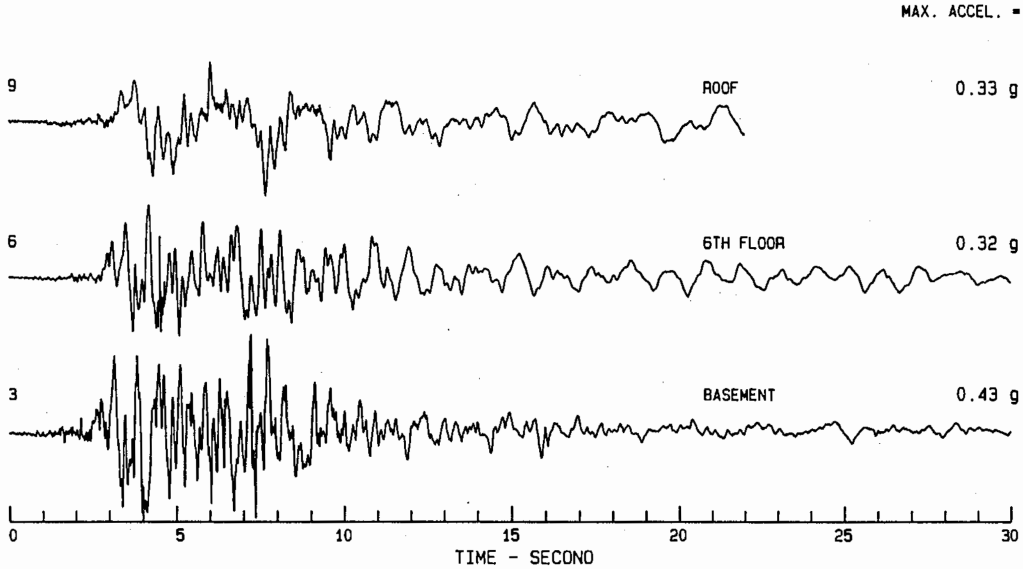



Fig. 9. Accelerations in the reference north-south direction recorded at the Woodland Hills - Oxnard #4 Building during the 1994 Northridge earthquake. After the earthquake, about 10% of the connections in this 13-story steel frame building were found damaged.

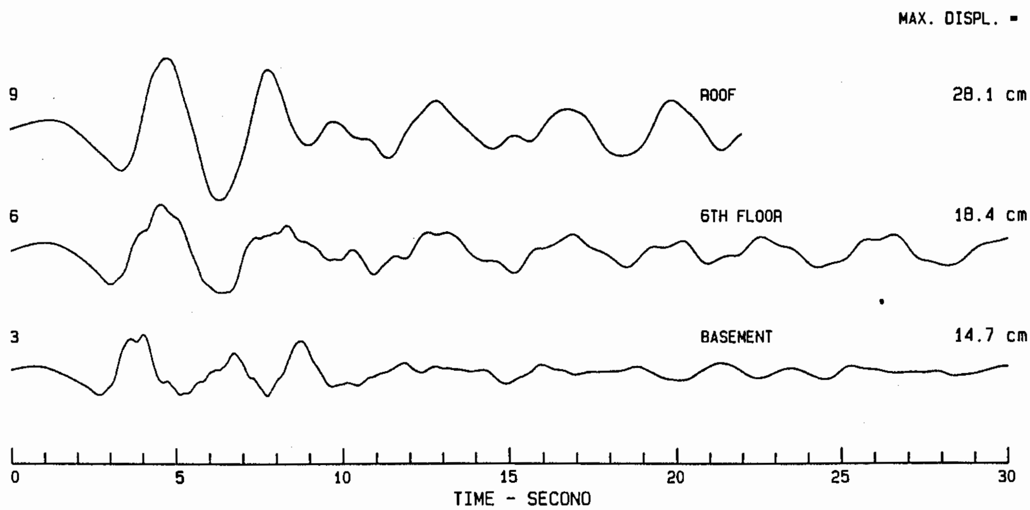



Fig. 10. Displacements corresponding to the accelerations in Fig. 9.

Detailed description of the damage is given in the EERI Reconnaissance Report (EERI, 1995) and the SAC report (Anderson and Filippou, 1995).

Both buildings were instrumented by the owner with one accelerograph at the roof level by the owner as required by the Los Angeles Building Code. The roof acceleration records for the two buildings are shown in Figure 11. In general, the response of the buildings are quite similar with much stronger response in the N-S direction. The peak acceleration near 9.5 seconds is slightly larger for the East Tower than the West Tower. The building is a long-period structure, so the acceleration records were dominated by the higher mode response. In addition, many high-frequency spikes can be seen in the records. The roof displacements for the first 30 seconds are shown in Figure 12, which indicate that the building period was about 4.6 seconds in the N-S direction and about 4 seconds in the E-W direction. By using the basement record of the Oxnard #4 building, which is across the street from this building, one can estimate the maximum drift between the roof and the base. The maximum drift for the East Tower in the N-S direction is about 47 cm, 0.7% of building height, and about 2.5 times that in the E-W direction. The base shear from the first mode, estimated from the roof record, is about 0.05W. The contribution of higher modes to the base shear is significant as compared to the first mode. The design base shear is about 0.05W. The damping ratio for the N-S direction, estimated from the 60-second long displacement records, is about 4%.

Los Angeles - Olympic #2 Building

The Olympic #2 Building in west Los Angeles, built in 1982, is an 11-story building. The building consists of six levels of office space over five levels of parking space. The building has vertical setbacks and re-entrant corners. The earthquake resistance of the building is provided by four steel moment-resisting frames in each direction. Post-earthquake inspection revealed that 258 of the 913 inspected connections suffered varying degrees of damage and the tenants were evacuated (Naeim et al., 1995a). Detailed description of the damage are presented in the SAC report (Naeim et al., 1995b).

The building was instrumented by the owner with one accelerograph on the roof level as required by the Los Angles City Building Code. The roof acceleration records and the corresponding displacements are shown in Figure 13. As seen in these records, between 5 and 10 seconds the building responded predominantly in the second mode with a period of about 0.6 seconds. High-frequency spikes appear near 10.5 and 11 seconds and can be seen on the acceleration records in both directions. This corresponds to the time when the maximum displacement occurred. This time may be interpreted as the time when damage of the structural members occurred. After the damage, the building responded mainly in the fundamental mode with a period of about 1.6 seconds, although the second mode response can be seen in the acceleration records. The response in the reference north-south direction was larger than that in the reference east-west direction and does not have a distinct period of vibration. This may imply that the damage in the north-south direction is more severe than that in the east-west

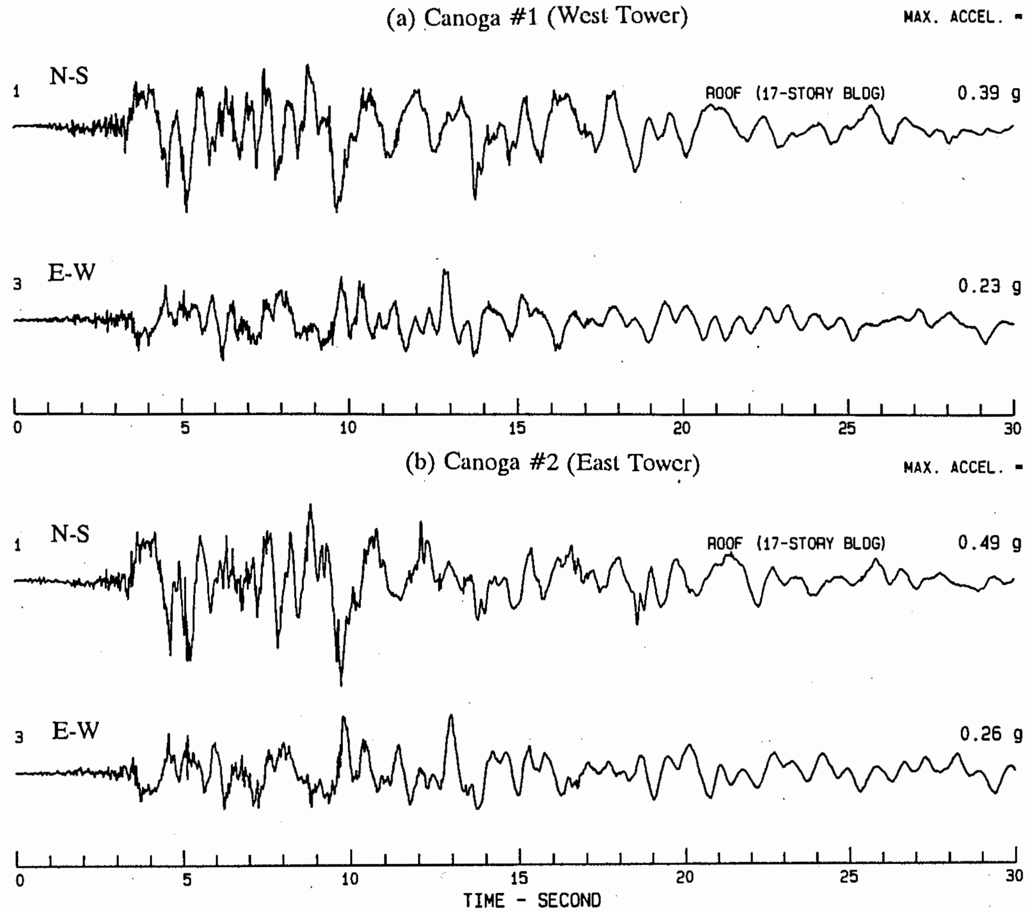



Fig. 11. Horizontal accelerations recorded at the roof level of the Woodland Hills - Canoga #1 (West Tower) and Canoga #2 (East Tower) during the 1994 Northridge earthquake. The East Tower was more severely damaged than the West Tower.

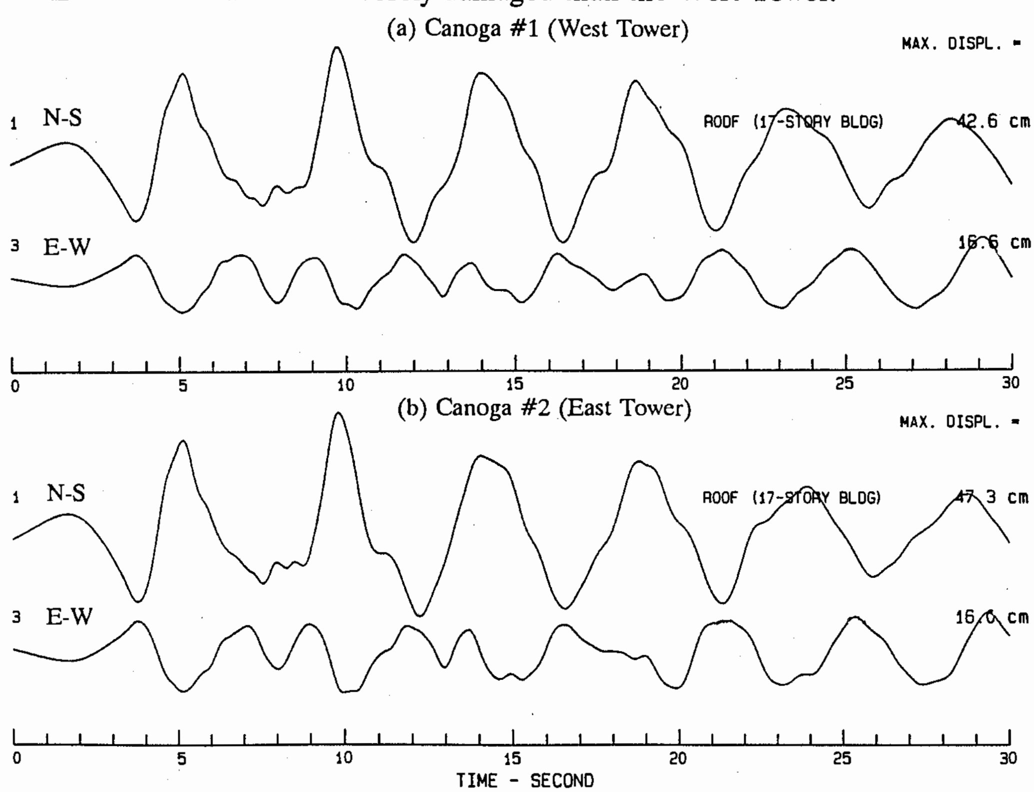
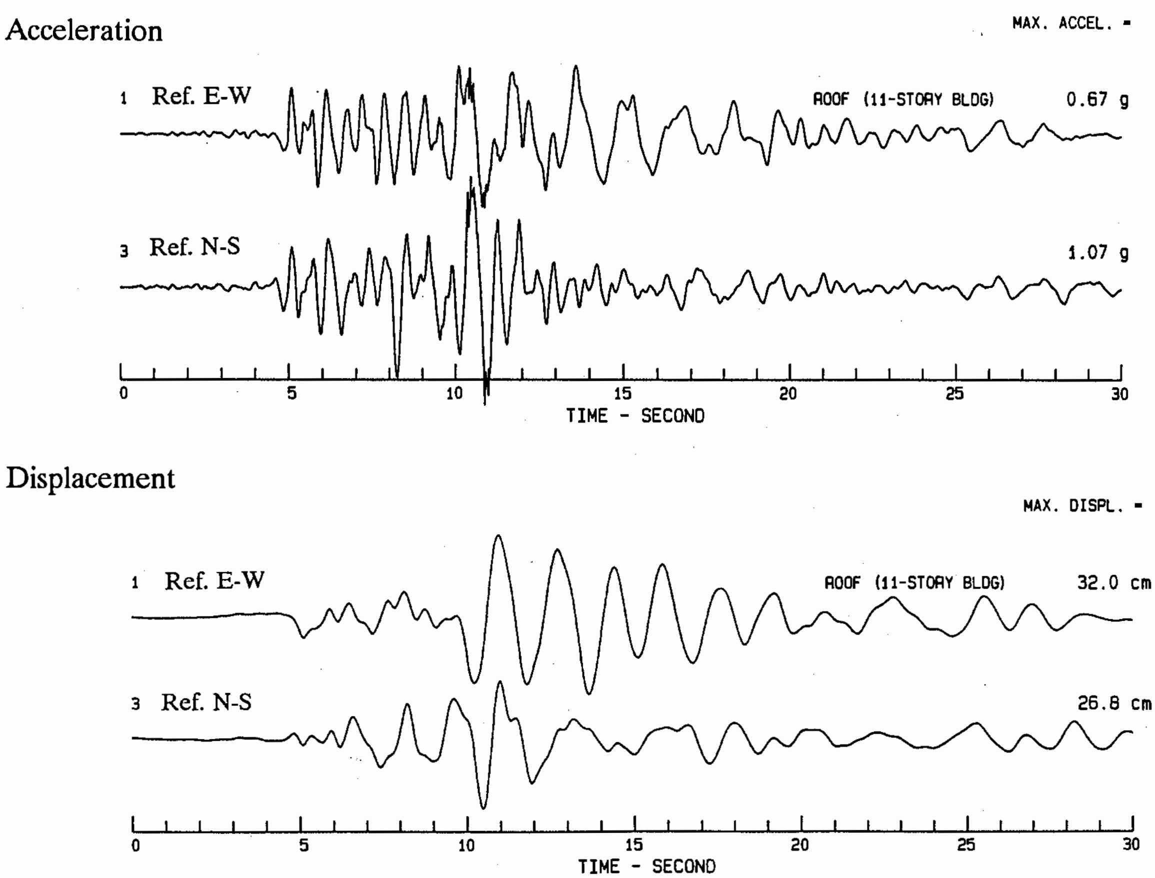



Fig. 12. Displacements corresponding to the accelerations in Fig. 11.

direction, although this has not been confirmed from the inspection. The lack of base records prevents computation of the total roof drift, but the maximum roof drift is estimated to be about 32 cm, 0.8% of building height, in the east-west direction. The maximum base shear from the first mode is about 0.20-0.25W, as compared with the design base shear of about 0.06W.

In summary, the Northridge records from the above four damaged steel buildings indicate that: (a) the period for two perimeter moment frame buildings is longer than the period given by 0.2N, N being number of stories; (b) higher mode response is dominant in all acceleration records; (c) the maximum base shear from the first mode is about 1 to 4 times the design base shear; (d) the total roof drift was 20% to 60% larger than the design drift limit, but the story drift due to higher modes can not be determined directly from the records, and (e) high-frequency spikes are seen in some of the records.




Horizontal accelerations and displacements recorded at the roof level of the Olympic #2 Building in west Los Angeles during the 1994 Northridge earthquake. After the Northridge earthquake, over 25% of the connections in this steel-frame building were found damaged.

## EVALUATION OF BUILDING RESPONSE

Following a destructive earthquake, strong-motion records from a building provide useful information for post-earthquake evaluation of the building. Analysis of the response of steel frame buildings using data recorded at the base and other floors have been shown to be very useful in defining the zone of the building where earthquake damage is most likely (e.g., Kariotis, 1996). In the SAC steel building project, the damaged steel buildings were analyzed using available data to test the ability of currently available computer modeling techniques in predicting the locations of fractured joints. Computer modeling of the building is usually time-consuming and requires years of experience to build a realistic computer model, and also requires a good understanding of the capability of the program used. In general, the results are highly dependent on the accuracy of modeling the existing structural conditions.

Examination of the building records can yield insight into the nature of the response as well as quantitative information on the parameters of the structural response such as building period, effective damping, drift, base shear and overturning moment. For most buildings, some of these parameters can be estimated by visual inspection of the records and a simple calculation. For some buildings, especially when the response is highly nonlinear, more rigorous analysis can be carried out by using system identification techniques to determine the parameters of a linear or nonlinear model of the structure that best fit the recorded response when the model is subjected to the recorded base motion (e.g., Beck and Jennings, 1980).

Most buildings are designed to remain elastic for code-specified forces and drifts, and some structural members are expected to yield to larger earthquake forces. Therefore one can expect no structural damage in a building if the records show that the building response is linear and the earthquake force and drift are smaller than the design values. The period lengthening during the shaking, larger base shear than the design base shear, or large drift is an indication of structural nonlinearity. Structural damage is very likely to occur when the response is highly nonlinear. A failure analysis of the building can be carried out to determine the maximum story drift that each story can withstand, the maximum base shear that causes the structural members to fail, and the period of vibration at which structural damage occurs. These values can then be compared with the corresponding values derived from the records. A building is expected to be severely damaged if the values derived from the record are larger than the estimated damage values. However, due to the uncertain nature of the actual strength in an existing building such a failure analysis will be approximate.

To determine how nonlinear a building response is, application of the system identification technique to the recorded data using a linear or nonlinear model is probably the most economic and systematic approach. Results can be obtained in a timely manner for post-earthquake evaluation. The identification techniques for linear models have been successfully applied to the San Fernando records (e.g., Beck and Jennings, 1980). Many researchers have applied system identification techniques to the detection of changes in structural parameters with a goal of damage detection (e.g., Agbabian, et al., 1991; Mau and Revadigar, 1994; Loh and Tou; 1994). As more building performance information is available, the system techniques can be applied to many Northridge records from damaged as well as undamaged buildings. The results can be correlated with the building performance and some damage indices or criteria may be derived at least statistically.

## SUMMARY

The records from two concrete and four steel buildings are reviewed for possible indications of structural damage from these records. Although no single character in the record can be used to indicate the structural damage, each record has some but not all of the following characteristics: (a) period lengthening, (b) large base shear, (c) large roof drift, c) nonlinear response and (d) high-frequency spikes. More rigorous analyses of these records, such as application of system identification techniques, are needed to study damage indices for detecting structural damage from strong-motion records. As the results of post-Northridge evaluation of more damaged and undamaged buildings become available, these indices may be derived statistically.

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# APPLICATION OF RECORDED MOTION TO POST-NORTHRIDGE EVALUATION OF STEEL FRAME BUILDINGS

John Kariotis

Kariotis & Associates, Structural Engineers
South Pasadena, California

## ABSTRACT

Contradictory recommendations have been made in the Interim Guidelines FEMA 267/ Aug 1995, and the SAC Technical Report. 95-04, as to the value of analysis in post-earthquake evaluation of WSMF buildings.

Analytical studies of the response of welded steel moment frames (WSMF) using data recorded at the base and up the height of the building have been shown to be very useful in defining the zone of the building where earthquake damage is most likely. When damage is found in the zone where analysis would have predicted, this zone must be extensively sampled. Effort expended in a random survey for earthquake-caused damage is not earthquake response related and is not cost-effective.

A random testing procedure is useful to determine the effectiveness of the original quality control procedure. An analysis based damage survey is more productive in finding if the site shaking caused earthquake damage.

## INTRODUCTION

The 1994 Northridge earthquake caused unexpected damage to the joints of welded steel moment frame (WSMF) buildings. The damage was concealed in many buildings by fire protection and ceilings. An external survey of the WSMF buildings did not find interstory drifts that are generally associated with structural damage. Investigations directed by engineers uncovered a level of damage that caused industry, academia and professional societies to convene committees that would advise the engineering profession, the building owners, the building officials and the construction industry on how to conduct an investigation, interpret the results and plan a repair program.

The SAC Joint Venture Partnership convened groups to prepare advisory documents. The group charged with consideration of the possible problems of existing buildings discussed how should WSMF buildings be evaluated, what constitutes a minimum inspection program, and how can earthquake damage be distinguished from quality control deficiencies.

General agreement was that a simplified analysis to identify locations of high stress was useful. The concept was that the significant earthquake damage would logically be in these "high stress" areas. The SAC Joint Venture funded studies of WSMF buildings to find if structural analysis could be utilized to reduce the numbers of joints investigated and/or provide data that could limit the investigation to locations in the building shown to be critical by analysis.

## INTERIM GUIDELINES

The Interim Guidelines, FEMA 267/August 1995, were developed from the SAC Advisories. The Interim Guidelines in Chapter 4, Post-Earthquake Evaluation states that "damaged connections tend to be widely distributed throughout the building frames, often at locations analysis would not predict. This approximates a random distribution". This statement contradicts the general observations in the SAC Technical Report 95-04, December 1995. This states; "All the analytical procedures were able, in at least a limited fashion, to provide an indication of locations within buildings where connection damage was most probable. That is, analytical indicators could be identified in all cases that would provide a better indicator of damage location than random sampling". Not all of the buildings analyzed in these SAC studies had an instrumental record of the shaking at the base of the building. The key to the success of analysis in prediction of the zone of probable earthquake damage is the availability of the base motion record.

The SAC studies also concluded that if incipient cracks were included in the report as earthquake damage the "damage zone" became more random in nature. The reverse of the opinion is: the existence of partial cracks in welds is not earthquake damage and this is supported by the prediction of the zone of column damage by analysis. Column damage is less likely to be related to flaws in the welding quality control program.

The repair of welded steel moment frames has commonly included repair of fractures in columns. Analysis have been very effective in predicting these zones of earthquake damage in columns when recorded data is used for loading of the WSMF structures.

## MODELING OF THE WSMF

The quality of the results of an analysis is highly dependent on the quality of the modeling. The analysts in the SAC research were required to model the building as commonly designed. That is, only the WSMF is used to resist the earthquake loading. However, in reality the beams and girders are composite with the concrete floor slabs. Research by Roberto Leon and others has shown that even welding of steel decking to beams and girders mobilizes the concrete slab placed on the decking as a part of the composite beam. Figure 1 shows the difference in the response to the recorded motion of a so-called "bare frame" from the response of the "probable" model. The difference in the curvature demand on the columns is not explicitly shown in this Figure. The increase in stiffness and strength in the girders causes the interstory displacement to be a "shear type" strain distribution. The curvature demand at each joint is concentrated in the columns.

The analysts were required to have a common method of computing a "demand/capacity" ratio. This was use of a "bare frame" model and response spectrum analysis. The response spectrum used in the analysis was a five percent damped spectrum prepared from the recorded motions . The demand/capacity ratio for the columns of the N-S frames were less than or equal to 1.0 except at the fourteenth floor level. The maximum at this level was 1.23 for an interior column and 1.10 for the corner column. The demand/capacity ratio for the girder at the fifteenth floor level was 1.19 and 1.08 at the interior and exterior bays respectively. If the expected stiffness of the girders has been used in the analysis, the curvature demand in the columns would be increased.

## INFLUENCE OF THE ANALYSIS METHOD

Figure 2 shows the difference between the results of a response spectrum analysis and an <u>elastic</u> time-history analysis. The data shown as story drift is the <u>maximum</u> value for the duration of the time-history analysis. These interstory drifts do not necessarily occur at the same time. If the analysis had been made using the "probable" model, even the lesser interstory drifts would significantly increase the curvature demand in the columns. A review of Figure 1 shows that the curvature demand on the column increases from the eighth floor level to a maximum at the fourteenth floor level.

A two-dimensional nonlinear analysis of the N-S frames resulted in the formation of the yield sequences shown in Figure 3. These calculations used two and one-half percent strain hardening up to 9.06 seconds of the base motion record. At the time the analysis stopped due to a negative tangent stiffness. This negative stiffness was caused by the effects of axial loads in the columns causing secondary moments (P$\Delta$ effects). The strain hardening effects were assumed to be ten percent and the nonlinear program became stable. Three snapshots of the displacement of the building relative to its base are shown in Figure 4.

The damage found by an investigation of <u>all</u> beam-column joints is shown in Figure 5 and 6. The masonry shear wall shown in Figure 6 is supposedly isolated from the frame by slotted holes in the connection angles. It is highly likely that this shear wall modified the probable response of Line 20. There was evidence that Line 5 impacted on the reinforced concrete shear wall parking structure. The effects of this impact on the observed damage cannot be estimated.

These studies clearly show the advantage of analysis using a recorded base motion for prediction of zones of probable earthquake damage. The anomaly at floor levels 7 and 9 on Line 5 may be related to the collision with the parking structure. The column damage at these levels is related to axial tensile stresses in the columns. This axial tensile stress is due to minimal dead load on the corner columns and to overturning moment effects.

## INFLUENCE OF THE CHARACTER OF THE GROUND MOTION

The recorded total energy at any site by the Northridge earthquake when calculated by methods, such Housner Intensity, was significantly less than that associated with a design level spectrum. This should not be considered unusual as the Northridge earthquake is only one of the family of earthquakes that populate a design level spectrum.

Figure 7 compares the five percent damped spectra for four scenarios. These are:

- 1994 UBC, $S_2$ type soils
- UCBC - Division 95, City of Los Angeles
- Recorded N-S component at the site.
- Uniform risk-10% probability of exceedence in 50 year time period-for the Olive View site.

The effects of each of these loadings on the N-S frames of the building is shown in the plot of story displacements. These displacements were calculated by response spectrum analysis using a "bare frame" model. This analysis method is shown by Figures 1 and 2 to be relatively a poor procedure for prediction of possible earthquake damage, but shows the influence of the energy contained in each spectrum.

The Uniform Risk 10-50 spectrum developed for the Olive View site has a predicted ground acceleration of 0.7 g. The effects of the N-S component recorded at the Olive View site on this steel frame building is shown on Figure 3. The nonlinear analysis becomes mathematically unstable at 9.22 seconds. The instability is due to secondary effects caused by axial loading of the columns.

The energy content of a earthquake record was previously related to "Housner Intensity". This energy measure is the integral of the spectral velocity between defined frequencies such as from 0.1 seconds to 5.0 seconds. The principal difference between a spectrum for a 6 3/4 Richter magnitude event (and the Division 95, soil type 2 spectrum) and the UBC design level spectrum is that these use average spectral amplification values in lieu of mean plus one sigma spectral amplification values. Figure 7 clearly shows the difference in spectral acceleration values for periods longer than 2.5 seconds, and that there is a substantial difference of mean and mean plus one sigma spectral accelerations for longer periods.

The character of the Northridge earthquake was such that the excitation of primary mode of tall WSMF's was subdued. However, the energy available for excitation of second, third and fourth modes was nearly equal to a design level earthquake. This information alone indicates that the surveys for detection of earthquake damage should <u>not</u> be randomly distributed over the height of the building.

A very similar building about two blocks east on Ventura Boulevard had all instruments required by the Los Angeles City Building Code function. The copy of the recording tape, Figure 8, can be considered as a record of a forced vibration to about 11 seconds and a free vibration for the remainder of the record. The record shown as longitudinal is in the N-S direction. The fundamental period is estimated as about 2.5+ seconds in the N-S direction and about 2.0+ seconds in the E-W direction. The tenth story record generally shows that response effects in the fundamental mode in the longitudinal direction were not significant. The transverse (E-W) effects of fundamental response mode are more obvious. The record at the arcade (basement) level does not have any visible evidence of soils period effects.

The records of earthquake loading and structural response of a 24 story WSMF building on Wilshire Boulevard near the 405 Freeway have been digitized by CSMIP and spectra has been prepared. The roof record in the narrow direction of the building clearly indicates the free vibration state, Figure 9. The absolute acceleration vs. period plot has peaks at 1.4 and 4.0 seconds. The record perpendicular to Wilshire has similar response but with significantly less peak accelerations. The records at the 12th floor are nearly identical except for reduced accelerations.

These records can be interpreted as response in the fundamental and second mode. The second mode is quickly damped and does not appear in the free vibration response. The survey should be uniformly distributed over the height of the moment frames that are parallel to Wilshire. An investigation of the moment frames perpendicular to Wilshire will be a verification of the quality control program.

## CONCLUSION

If the purpose of the evaluation of welded steel moment frame buildings is to determine if the intensity of ground motion at that site caused damage that warrants repair, and this is the principal purpose of this evaluation, and recorded motions are available, the investigation should be concentrated in zones predicted to have the highest probability of damage by analysis of the building and the records. If damage is found in the zone with the highest probability of damage then the investigation is carried outward from this zone. Determination of the effectiveness of the original quality control program should not be a priority unless so determined by the building owner.

The records obtained by the UBC mandated program are of value to estimate the fundamental modes of the building. Knowing this and the character (Richter or moment magnitude) of the earthquake it can be determined whether fundamental or higher mode response was critical for possible damage. If the records have been digitized and integrated the interpretation of the records is simplified. It is probable that analysis of the records alone is adequate to plan an evaluation program to detect earthquake damage.

If dynamic analysis of the building is believed to be cost-effective, the building should be modeled with its expected stiffness. Our studies have found the elastic or nonlinear time-history studies are most reliable for prediction of possible damage zones. Nonlinear analysis requires significantly more effort and did not contradict the results of the elastic time-history studies. It is likely that elastic time-history studies can adequately locate the zones when the evaluation of the WSMF should begin.

STORY DISPLACEMENTS & INTERSTORY DRIFT
BARE MODEL VS. PROBABLE MODEL
TIME-HISTORY ANALYSIS USING NORTH-SOUTH COMPONENT OF BASE RECORD

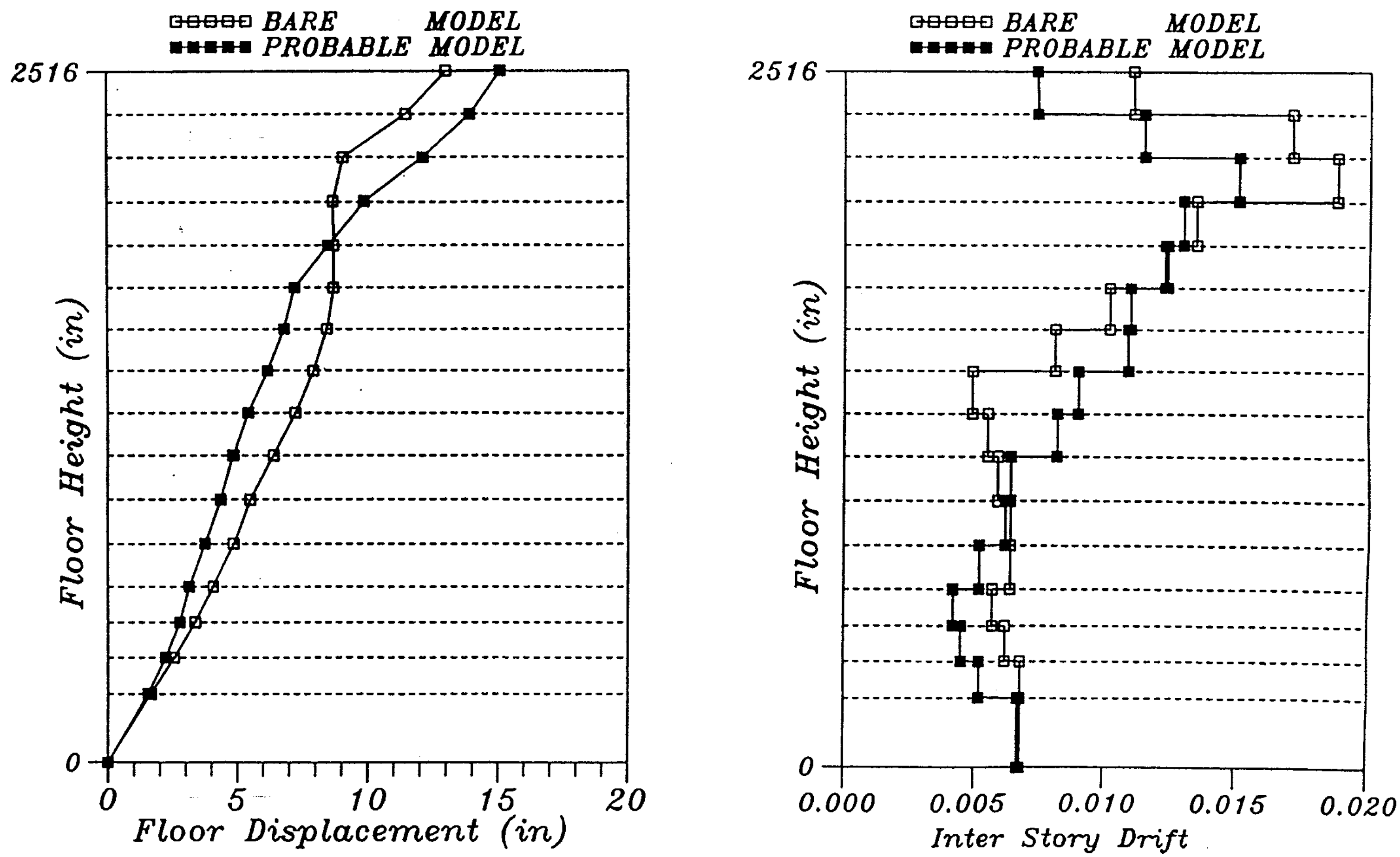



FIGURE 1

STORY DISPLACEMENT & INTERSTORY DRIFT
RESPONSE SPECTRUM VS. TIME-HISTORY ANALYSIS
NORTH-SOUTH COMPONENT OF BASE RECORD - BARE FRAME

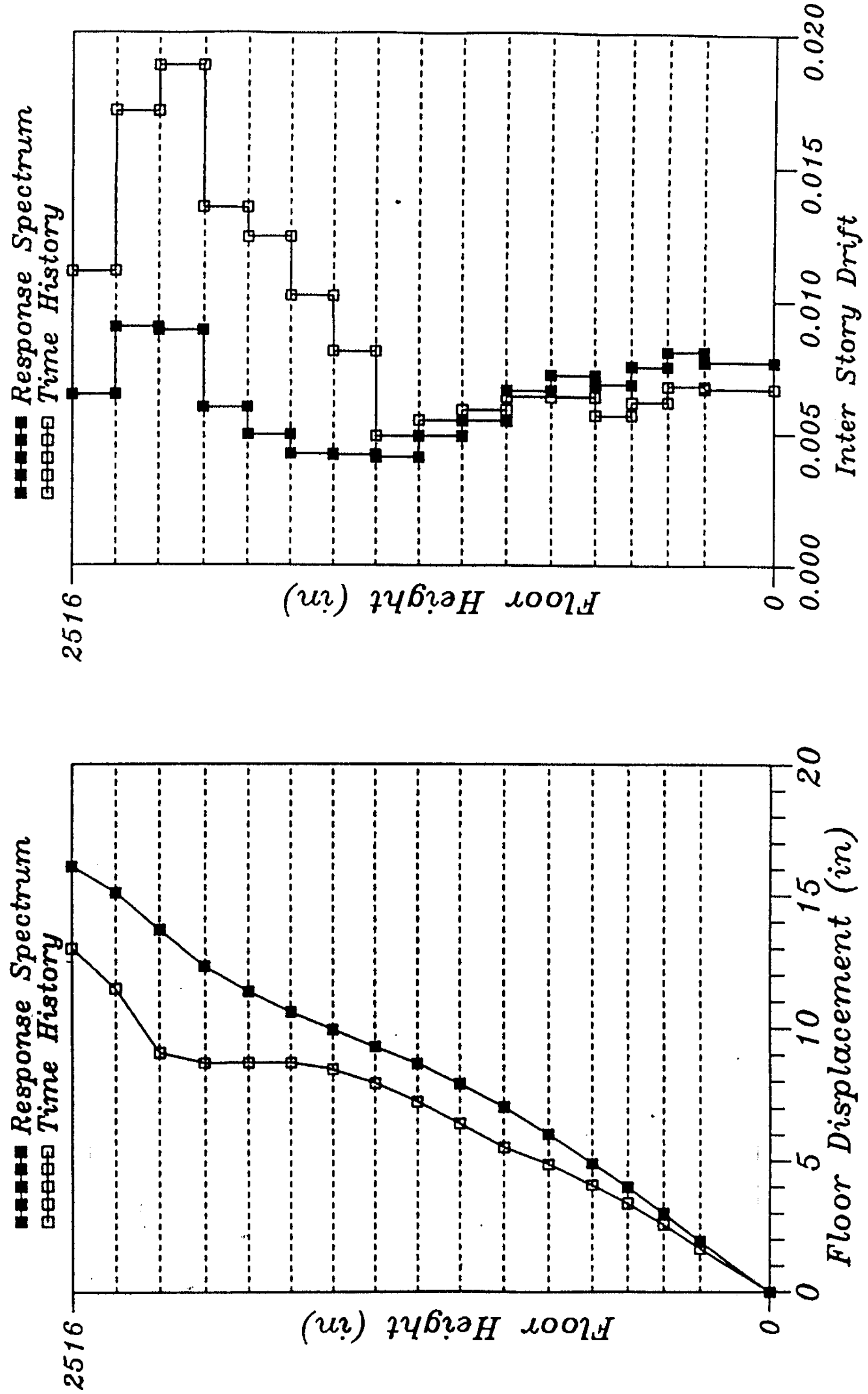



FIGURE 2

SEQUENCE OF FORMATION OF PLASTIC HINGES IN NORTH-SOUTH FRAMES ON LINES 5 & 20
(a)RECORDED BASE MOTION - 2.5% STRAIN HARDENING TO 9.06 SEC.
(b)SYLMAR RECORD - 10% STRAIN HARDENING

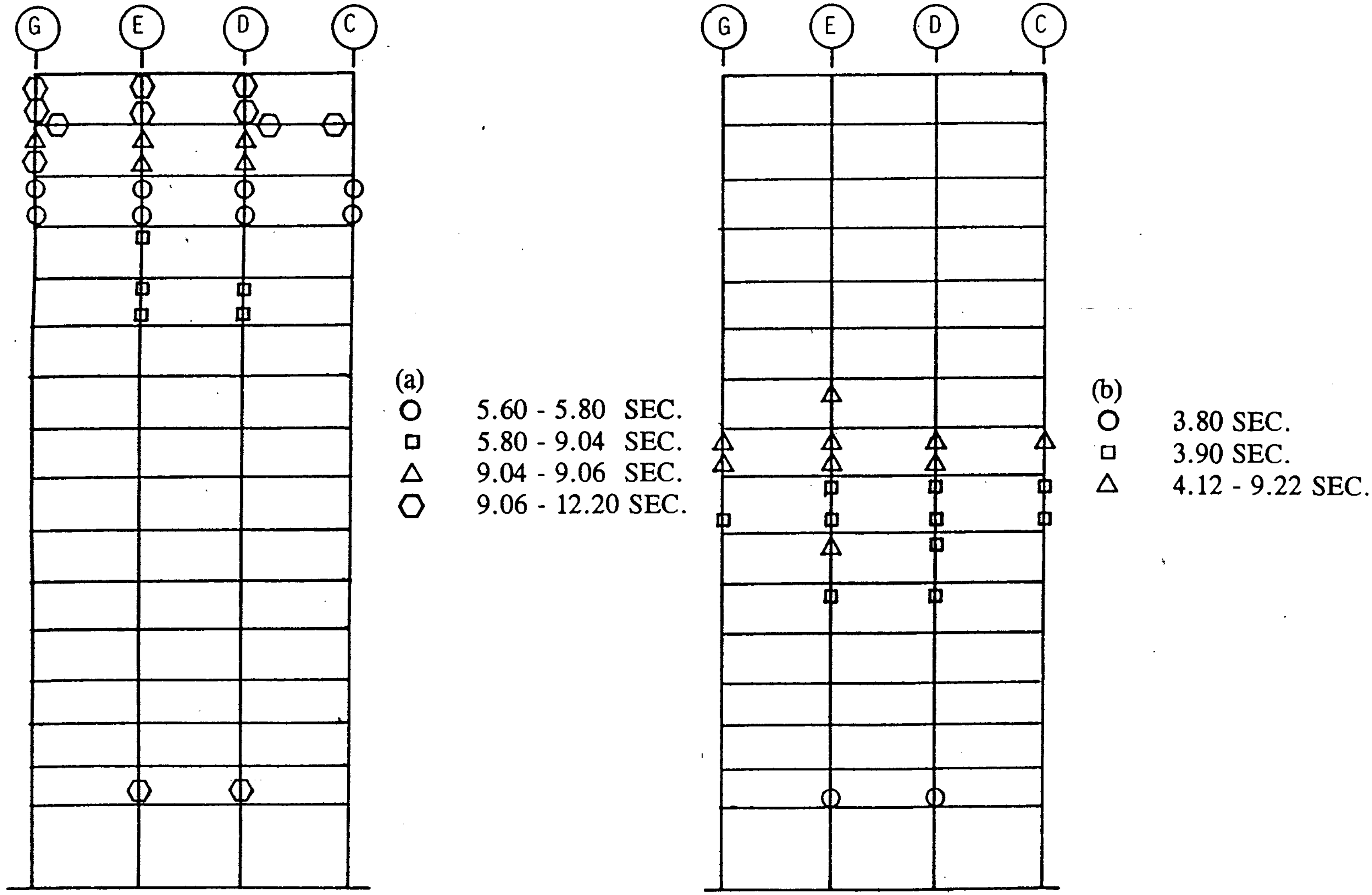



FIGURE 3

RESULTS OF THE NONLINEAR ANALYSES IN THE NORTH-SOUTH DIRECTION

(a) MAXIMUM STORY DISPLACEMENTS AT 9.0 SEC. OF THE BASE RECORD - 2.5% STRAIN HARDENING

(b) MAXIMUM STORY DISPLACEMENTS AT 9.0 SEC. OF THE BASE RECORD - 10% STRAIN HARDENING

(c) MAXIMUM STORY DISPLACEMENTS AT 15.0 SEC. OF THE BASE RECORD - 10% STRAIN HARDENING

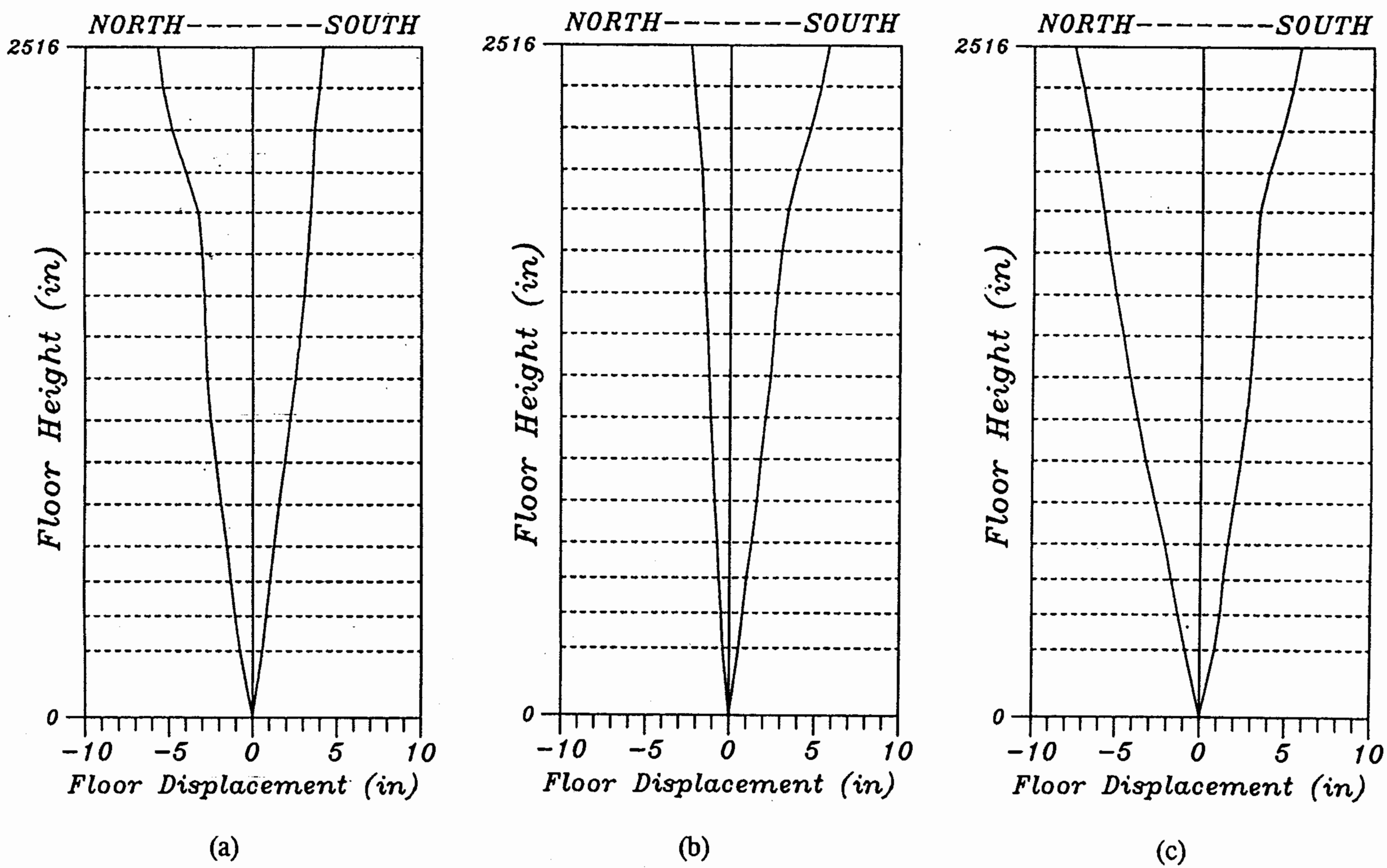
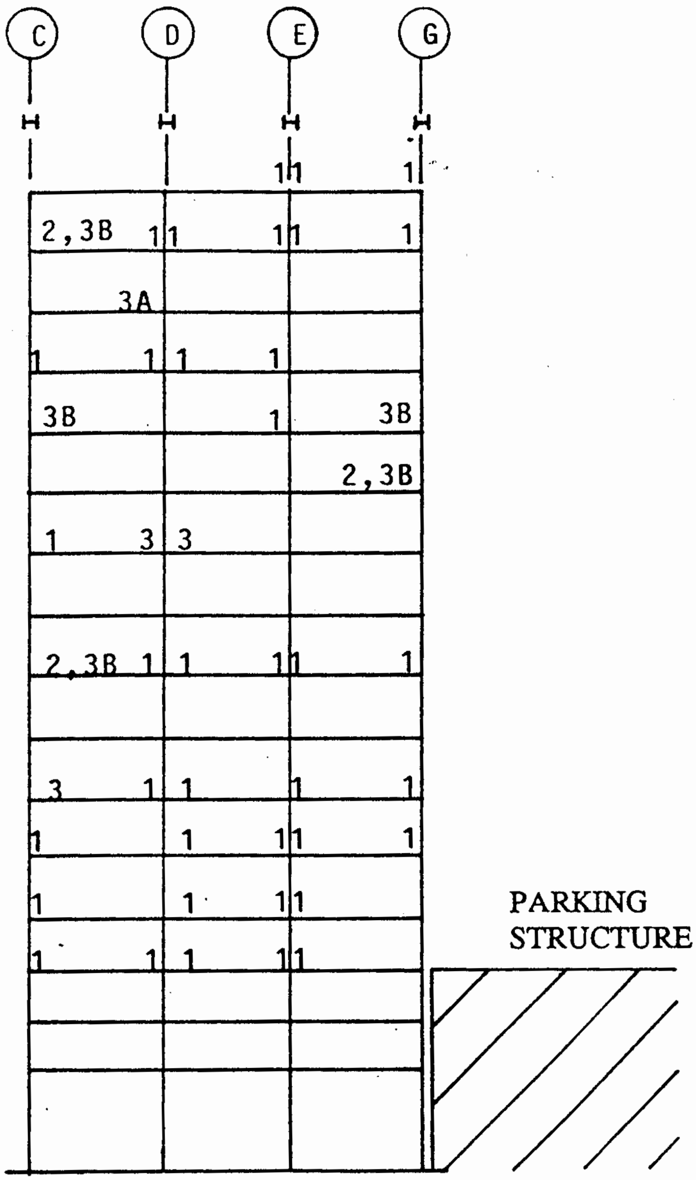



FIGURE 4

EAST ELEVATION - LINE 5



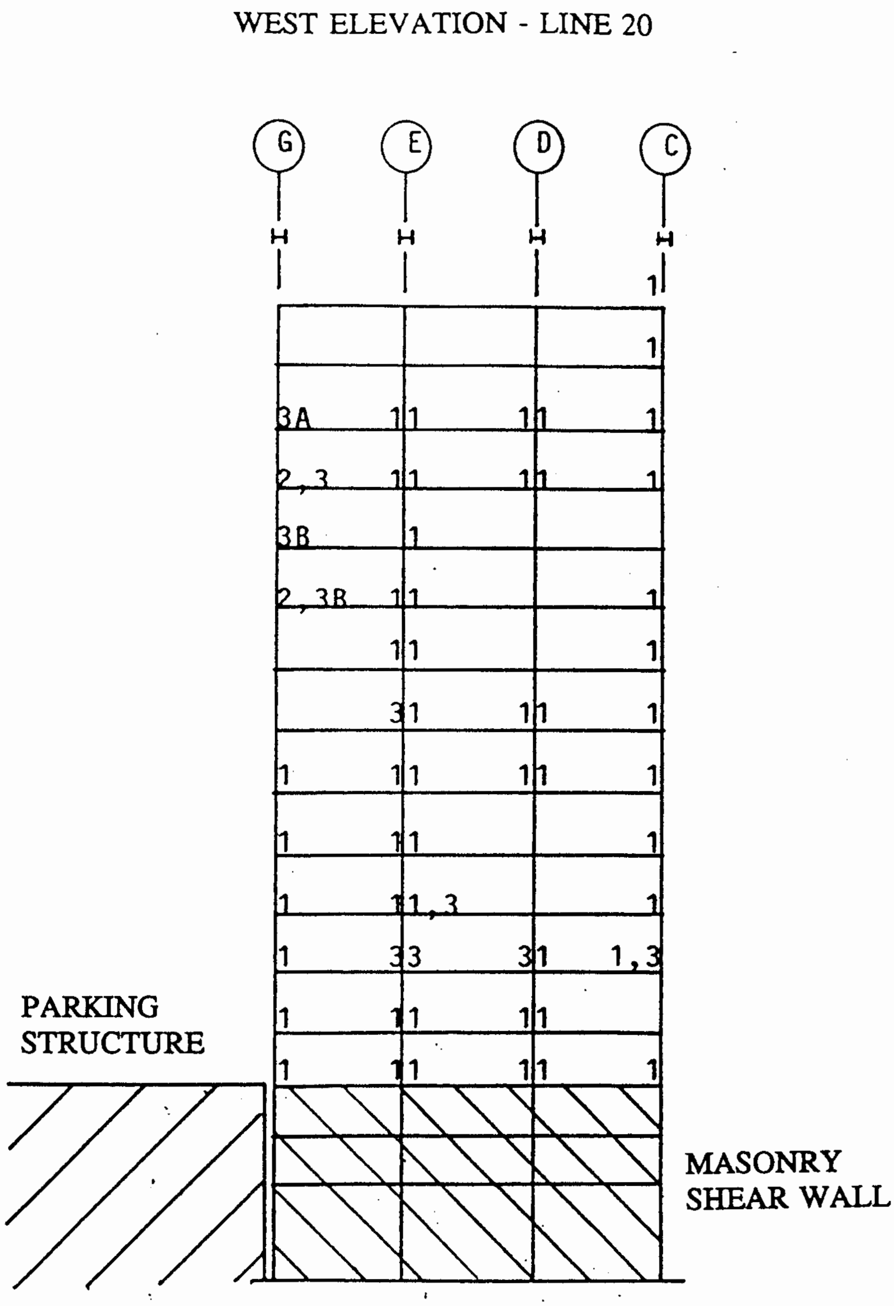

LEGEND

1. PARTIAL CRACK IN WELD OF BOTTOM FLANGE OF BEAM
2. COMPLETE CRACK THROUGH WELD OF BOTTOM FLANGE OF BEAM
3. PARTIAL CRACK INTO COLUMN FLANGE

3A. COMPLETE CRACK THROUGH COLUMN FLANGE

3B. COMPLETE CRACK THROUGH COLUMN FLANGE PLUS CRACK INTO COLUMN WEB

FIGURE 5




LEGEND
1. PARTIAL CRACK IN WELD OF BOTTOM FLANGE OF BEAM
2. COMPLETE CRACK THROUGH WELD OF BOTTOM FLANGE OF BEAM
3. PARTIAL CRACK INTO COLUMN FLANGE
3A. COMPLETE CRACK THROUGH COLUMN FLANGE
3B. COMPLETE CRACK THROUGH COLUMN FLANGE PLUS CRACK INTO COLUMN WEB

FIGURE 6

COMPARISON OF RESPONSE OF BARE FRAME TO FOUR SPECTRA
NORTH-SOUTH DIRECTION

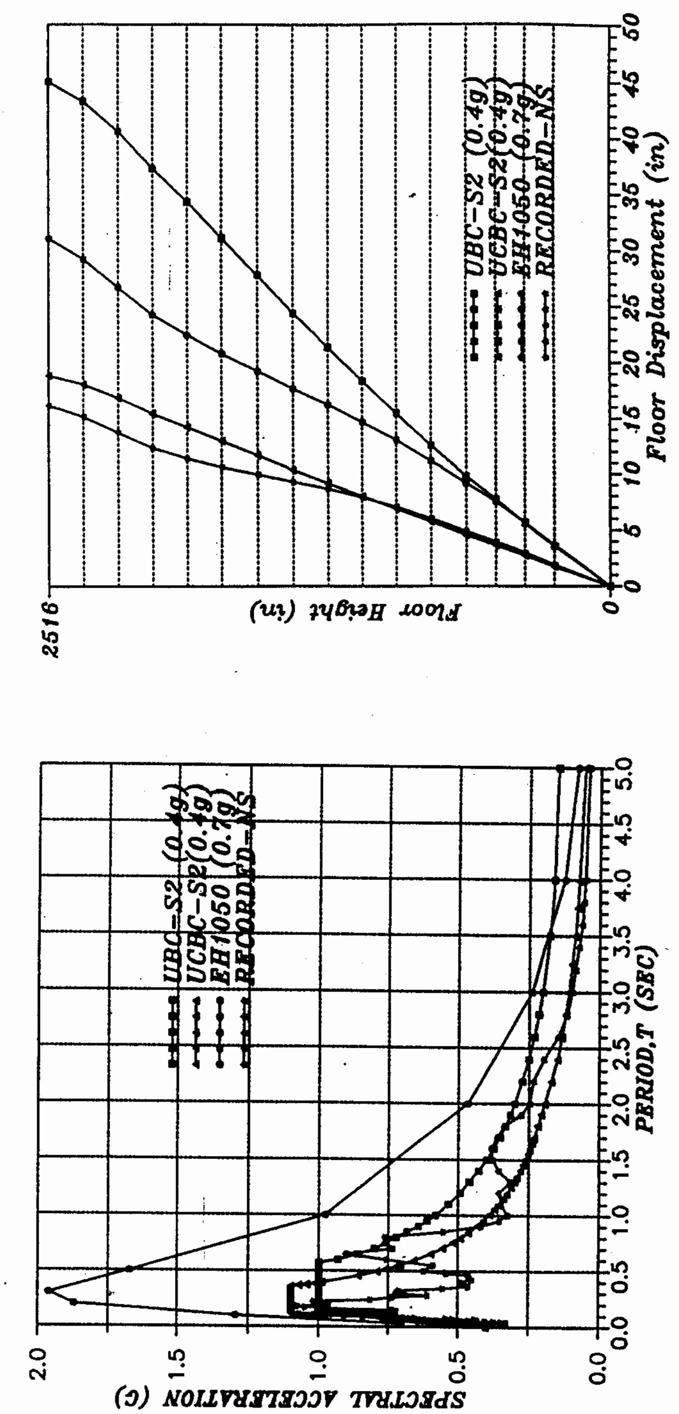
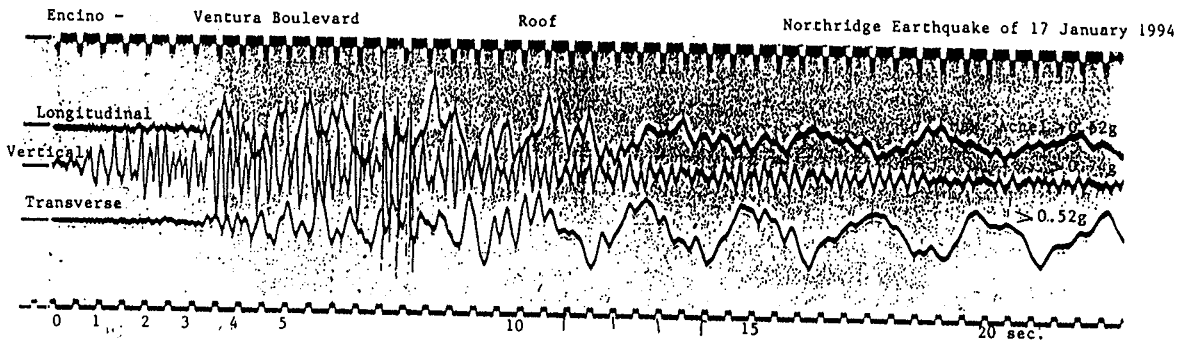
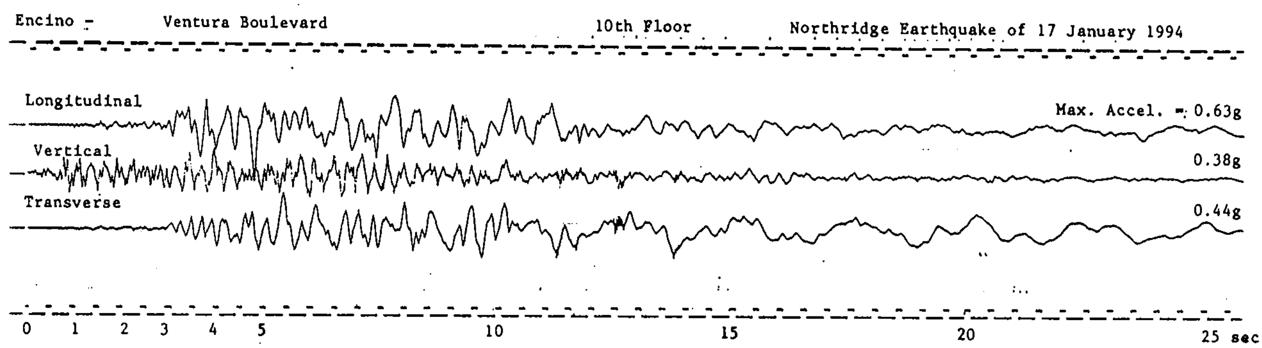


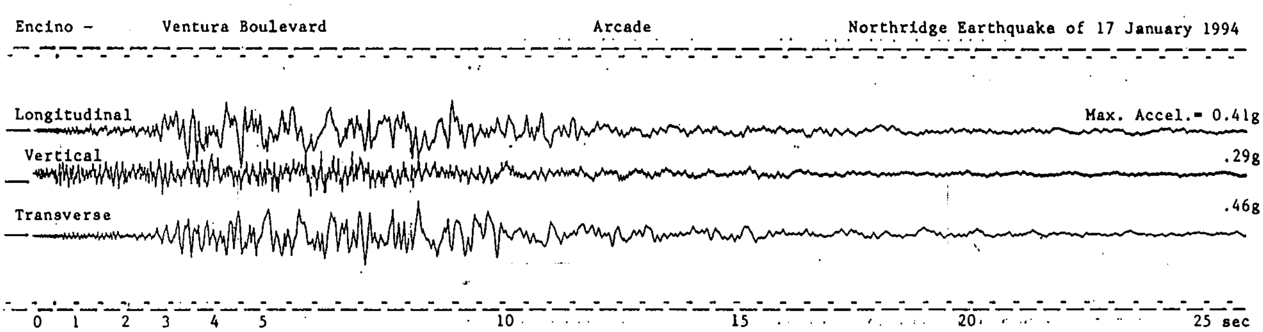

FIGURE 7




Note: Longitudinal direction is perpendicular to Ventura Boulevard.

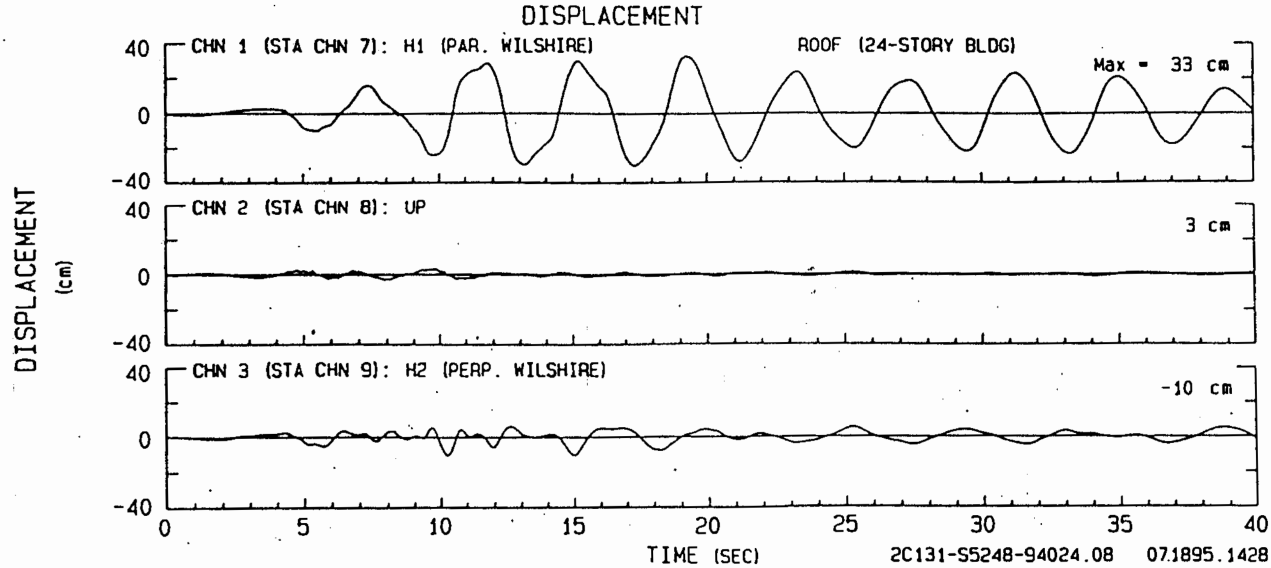


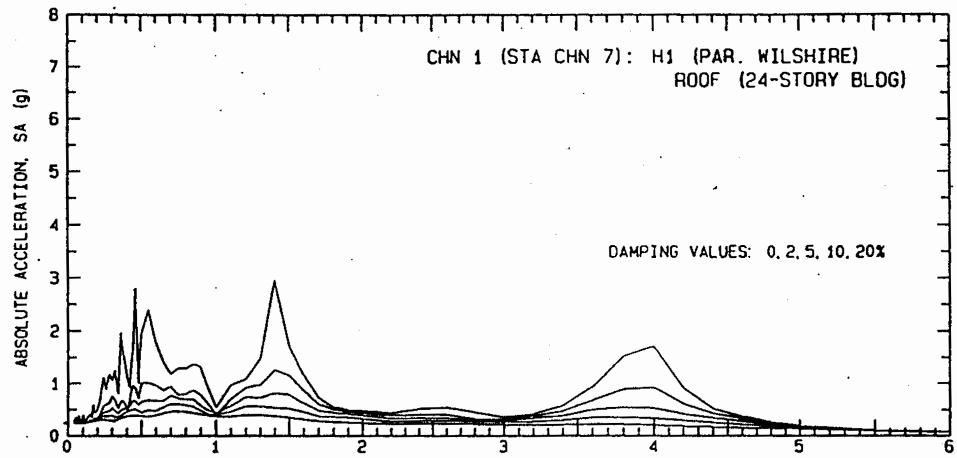

Note: Longitudinal direction is perpendicular to Ventura Boulevard




Note: Longitudinal direction is perpendicular to Ventura Boulevard.

FIGURE 8


DISPLACEMENT
CHN 1 (STA CHN 7): H1 (PAR. WILSHIRE)
ROOF (24-STORY BLDG)
Max = 33 cm
CHN 2 (STA CHN 8): UP
3 cm
CHN 3 (STA CHN 9): H2 (PERP. WILSHIRE)
-10 cm
DISPLACEMENT (cm)
TIME (SEC)
2C131-S5248-94024.08 07.1895.1428


CHN 1 (STA CHN 7): H1 (PAR. WILSHIRE)
ROOF (24-STORY BLDG)
DAMPING VALUES: 0, 2, 5, 10, 20%
ABSOLUTE ACCELERATION, SA (g)

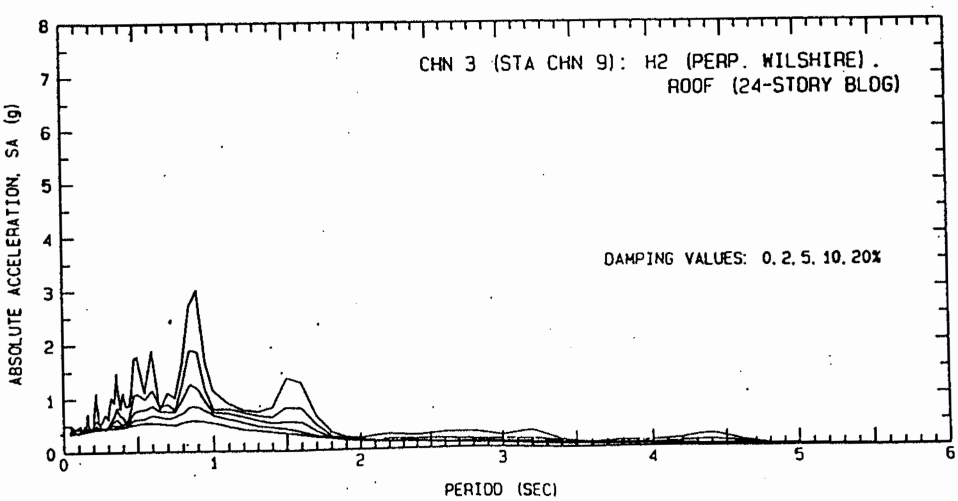

CHN 3 (STA CHN 9): H2 (PERP. WILSHIRE).
ROOF (24-STORY BLDG)
DAMPING VALUES: 0, 2, 5, 10, 20%
ABSOLUTE ACCELERATION, SA (g)
PERIOD (SEC)

FIGURE 9