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FREQUENCY DOMAIN IDENTIFICATION OF STRUCTURAL MODELS FROM EARTHQUAKE RECORDS

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FREQUENCY DOMAIN IDENTIFICATION OF STRUCTURAL MODELS FROM EARTHQUAKE RECORDS

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ABSTRACT

The usefulness of simple linear mathematical models for representing the behaviour of tall buildings during earthquake response is investigated for a variety of structures over a range of motions including the onset of structural damage. The linear models which best reproduce the measured response of the structures are determined from the recorded earthquake motions. In order to improve upon unsatisfactory results obtained by methods using transfer functions, a systematic frequency domain identification technique is developed to determine the optimal models. The periods, dampings and participation factors are estimated for the structural modes which are dominant in the measured response.

The identification is performed by finding the values of the modal parameters which produce a least-squares match over a specified frequency range between the unsmoothed, complex-valued, finite Fourier transform of the acceleration response recorded in the structure and that calculated for the model. It is possible to identify a single linear model appropriate for the entire response, or to approximate the nonlinear behavior exhibited by some structures with a series of models optimal for different segments of the response.

The investigation considered the earthquake records obtained in ten structures ranging in height from seven to forty-two stories. Most of the records were from the San Fernando earthquake. For two of these structures, smaller-amplitude records from more distant earthquakes were also analyzed. The maximum response amplitudes ranged from approximately 0.025 g to 0.40g.

The very small amplitude responses were reproduced well by linear models with fundamental periods similar to those measured in vibration tests. Most of the San Fernando responses in which no structural damage occurred (typically 0.2g-0.3g maximum accelerations) were also matched closely by linear models. However, the effective fundamental periods in these responses were characteristically 50 percent longer than in vibration tests. The average first mode damping identified from these records was about 5 percent of critical. Only those motions which produced structural damage could not be represented satisfactorily by time-invariant linear models. Segment-by-segment analysis of these records revealed effective periods of two to three times the vibration test values with fundamental mode dampings of 15 to 20 percent.

The systematic identification technique generally achieves better matches of the recorded responses than those produced by models derived by trial-and-error methods, and consequently more reliable estimates of the modal parameters. The close reproductions of the measured motions confirm the accuracy of linear models with only a few modes for representing the behaviour during earthquake response of tall buildings in which no structural damage occurs.

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I. INTRODUCTION

1.1 THE NEED FOR STRUCTURAL IDENTIFICATION FROM EARTHQUAKE RECORDS

Strong motion earthquake records provide one of the few sources of information on the response of large structures to potentially damaging excitations. It has been observed from these records that the dynamic properties of many structures are markedly different during response to strong ground motion than in small amplitude ambient and forced vibration tests. These differences are most evident as lengthened fundamental periods and higher dampings during earthquake response, and occur at levels of response approaching and including incipient damage. (Response of heavily damaged or collapsed structures has not yet been recorded.) The properties at large amplitudes are more relevant, of course, for earthquake resistant design, and since vibration tests at the amplitudes typical of strong earthquakes are not feasible, it is of considerable interest and importance to extract information about structural behavior from strong motion data.

Two of the factors which must be known better for the improvement of earthquake resistant design of structures are the nature of the earthquake ground motions which are likely to be encountered, and how a structure will respond to a given ground motion. Consequently, much effort has been devoted in seismic regions around the world to recording earthquake ground motions and structural responses. Prior to 1971, this had resulted in a limited number of strong ground motion records being obtained, together with a very few structural response records ([13], [16]).

The San Fernando earthquake of February 9, 1971 produced many more ground acceleration records and, more importantly for the present application, structural response records from approximately fifty buildings designed according to modern building codes and practices. This earthquake provided data that engineering researchers had long been seeking -- actual measurements of earthquake responses which could be compared with those calculated by subjecting structural models to the measured ground excitation. The importance with which these records are regarded is reflected in the dynamic analyses of many structures described in the NOAA report on the San Fernando earthquake [19].

Although general agreement between analysis and observation was obtained, these studies and others of the San Fernando data revealed deficiencies in the methods of analysis used to extract information about structural behavior from recorded earthquake responses. Many of the early studies used trial-and-error modifications of the parameters of a model synthesized from design data to improve the matches between the calculated and measured response histories. Transfer function approaches in the frequency domain were also common.

With the increased data provided by the San Fernando earthquake, there was now an opportunity for the development of systematic structural identification techniques to obtain optimal estimates of the parameters of models according to well-defined criteria. These methods have the capability of extracting much more information from the records than the methods previously used in earthquake engineering.

The aim of structural identification from earthquake records, in general terms, is to improve the understanding of the dynamic response of structures to strong ground motions. Improving structural models involves determining both the types of mechanisms which are important in the response, and the numerical values of the parameters of the selected models. Better models and more accurate parameter determination are required to allow advantageous use of sophisticated analytical tools, such as finite element methods and dynamic analysis techniques. These tools have been used for calculating the response of structural models to earthquakes, but their usefulness is limited by the reliability of the structural models (as well as uncertainties in the input).

1.2 THE SYNTHESIZED MODEL AND TRANSFER FUNCTION APPROACHES

Two common methods of analysis of recorded earthquake response data are the synthesized model approach with trial-and-error adjustment of the parameters, and the transfer function approach in the frequency domain. Often these methods are combined: the frequency domain data, which accentuate the approximate modal frequencies, may be used to guide the modifications in the parameters of the synthesized model; or an initial model derived from either design data or vibration test data may be used to interpret Fourier response spectra in terms of modal periods and dampings.

Use of the synthesized model involves formulating a model from the design data, calculating the response to the measured ground acceleration, and making trial-and-error adjustments to the model parameters to achieve a better match of the measured response. The principal usefulness of the method is in determining how well the response of the initial model matches the recorded response since this illustrates the accuracy of the design procedure. The weakness of the method for structural identification lies in the trial-and-error parameter adjustment: there is no quantitative criterion to define the goodness-of-fit, and there is no systematic way to compute the parameter adjustments required to improve the fits. The complicated interactions of changes in different parameters on the calculated response pose a difficult problem to solve by trial-and-error methods, and usually the approach ceases after two or three sets of parameter changes have been performed, with no guarantee of convergence to the optimal estimates. This failure to achieve the best match of the data, according to some well-defined criterion, may lead to incorrect conclusions about the ability of a class of models to represent the behavior of the structure. Several examples presented later in this thesis will illustrate that systematic identification techniques typically achieve much closer approximations to the observed behavior than trial-and-error methods using the same type of model.

There are many difficulties involved in the synthesis of a dynamic model. Most design analyses are based on a linear, time-invariant, planar model with a rigid base and classical normal modes. There are several assumptions involved in this formulation which are idealizations of the real behavior. Within the framework of this class of model, the usual first step in trial-and-error approaches is to calculate the mass, damping and stiffness matrices and then modally decompose the resulting equations of motion to calculate the earthquake response. In the parameter adjustment phase, it is usually the modal properties which are varied rather than the matrix elements. The effect of adjustments to the modal parameters on the calculated response is easier to visualize intuitively than the effect of changing elements of the stiffness matrix. In fact, accurate estimation of the matrix elements may not be possible even by systematic identification techniques, let alone by intuitive methods, because of the limited amount of data and the effect of noise.

The synthesis of the three matrices is not straightforward. Even within the given assumptions, the model may vary from a simple lumped-mass formulation, in which the mass and stiffness of all the building components at each story level are lumped into a single mass and spring, to a more complex finite-element formulation which attempts to represent the major building components individually. In actual buildings, both the stiffness and damping are amplitude dependent, and the engineer must select the values appropriate for the range of expected amplitudes. The difficulties inherent in this estimation are revealed by the discrepancies in period between the design model and that seen in earthquake response.

There is no commonly applied systematic method to synthesize the damping matrix from the damping of the individual components, although such approaches have been suggested (e.g., Raggett [22]). Indeed, the damping matrix itself is usually not calculated directly; rather, assumed values of modal dampings are employed. Typical values range from 2 to 10 percent of critical, depending on circumstances.

The second common method of analysis of earthquake response data, the transfer function approach, is based on the simple relation that exists for linear, time-invariant systems between the Fourier transform of the input, $Z(\omega)$, and the output, $Y(\omega)$, in the frequency domain.

These transforms are related through the complex-valued transfer function, $H(\omega)$:

$$Y(\omega) = H(\omega)Z(\omega)$$

This relationship assumes zero initial conditions for the input and output and implies that full time histories are used in calculating $Z(\omega)$ and $Y(\omega)$. In the analysis of earthquake data, where the input is usually the recorded base acceleration and the output the measured acceleration response at some position in the structure, both conditions are commonly violated. While the structure is initially at rest, the first part of the motion is often not recorded since a threshold level of motion is required to trigger the instrument. In addition, truncated time histories are usually used in the calculation of the transforms, with the small amplitude motions in the tails of the records being neglected. In time-window analyses, obviously the full time histories are not used, and non-zero conditions typically prevail at both the beginning and end of all segments.

In applications in earthquake engineering, the system is often both non-linear and time-variant. The ratio of the output transform to the input transform then provides an average characterization of the non-linear system over the duration of record used, and is interpreted as the transfer function of the equivalent time-invariant linear system. The more ambitious frequency domain studies attempt to trace the time variation of the system properties by a moving window Fourier analysis, considering the records segment by segment [31].

In practice, $Y(\omega)$ and $Z(\omega)$ are calculated at discrete frequencies from finite digitized records, using a fast Fourier transform (FFT) algorithm. Typically, the initial and final portions of the time segments are tapered to provide smooth window functions which convolve with the transforms of the time histories to produce the estimated transforms. To produce smoother functions for examination, a weighted average of the transforms over several neighboring frequency points is often calculated. In most cases the estimated transfer function is also smoothed, accentuating the major peaks in comparison to the minor ones, but reducing the amplitude and increasing the band-width of all peaks. In addition, it is common in earthquake engineering to concentrate attention on $|H(\omega)|$, disregarding the information contained in the phase spectrum.

Once the modulus of the transfer function, $|H(\omega)|$, has been determined, the parameters of the lower modes are estimated from the theoretical form of $|H(\omega)|$. These estimates typically involve the use of only a few of the values of $|H(\omega)|$. Points near the maxima of $|H(\omega)|$ are used to determine the modal frequencies, and the amplitude of the peaks and the band-width at the half-power points are used to estimate the participation factors and modal dampings.

Unfortunately, the calculated function $|H(\omega)|$ is usually very jagged, unlike its theoretical counterpart which is a smooth curve with well-defined peaks at the lower modal frequencies. The jaggedness is caused by the combined effects of time-variation and amplitude non-linearity of the system, finite length and discrete sampling of the records, the neglect of the effects of the initial conditions, and measurement noise in the data. Because of the irregularity of $|H(\omega)|$, it is often difficult to identify more than the first one or two modal frequencies with confidence. In addition, the half-power bandwidths are generally poorly defined, making accurate estimates of damping and participation factors difficult.

Two of the major disadvantages of the transfer function approach are that it is basically a nonparametric method with a parametric model imposed at the end of the calculation to interpret the results, and that most of the data areignored in estimating the parameters because only a few frequency points are used. Although the analyst may have a specific form of model in mind, this is essentially a "black box" approach, in that the form of the model is not used to constrain the estimates of the transfer function. Regarding the second point, many "parameters" are estimated in the transfer function method, namely the values of the transfer function at each frequency, but most of them are not used in calculating the modal frequencies, dampings and participation factors.

Intuitively, one suspects that a more successful approach would utilize more of the knowledge about the model from the outset of the analysis, and would include more of the frequency points directly in the estimation of the parameters by using some integrated measure-of-fit.

The limitations of the modified synthesized model and the transfer function approaches for accurately estimating structural properties from recorded earthquake data have recently led to the development of more systematic structural identification techniques. In the main these techniques have been adopted from other fields, but the nature of earthquake excitation and response poses some problems which require special attention.

The transient nature of the excitation is important in that it eliminates many identification techniques which have been successful in other applications but which require a specific form of input, for example band-limited Gaussian noise or sinusoidal excitation. Characteristically, earthquake excitation is nonstationary, with both the amplitude (r.m.s. value) and spectral character changing in time; transient, with a duration of typically forty seconds or less; and non-repeatable. The short duration of a record may pose resolution problems for the estimation of low natural frequencies. The nonstationarity and short duration of the records, together with non-linear and time-varying structural behavior in strong shaking, make smoothing of parameter estimates by averaging over several segments of a record difficult. The non-repeatability of the excitation removes the possibility of smoothing of estimates by ensemble averaging.

Generally the model being identified is a gross approximation to the actual system, as for example an equivalent time-invariant model for an obviously time-varying system. In structural identification, where the overall behavior of a building is determined by the combination of many components, the optimal model in a given class may give an imperfect representation of the structure to a greater extent than in other fields concerned with smaller and simpler systems. The identification technique must therefore be robust, in the sense that it should not only correctly estimate the parameters of a model closely approximating the measured behavior of the real system, but also produce the best fit in the presence of considerable model error or measurement noise.

The following section reviews some of the identification techniques which are useful in earthquake engineering.

1.3 STRUCTURAL IDENTIFICATION LITERATURE REVIEW

Systematic identification techniques have been applied to civil engineering structures only recently. Prior to the San Fernando earthquake of 1971, there was little incentive to develop sophisticated methods for estimating structural parameters from earthquake response records because of the scarcity of data available for analysis. In fact, only one significant set of data existed in the United States, for the Alexander Building in the 1957 San Francisco earthquake (Hudson [13]). The many records created by the San Fernando earthquake, and the limitations of trial-and-error and transfer function methods for satisfactorily extracting information from the data, have led to much work in the structural identification field in recent years. Concurrently with the interest in analyzing earthquake response records, the increasing use of dynamic testing of complicated structures, including nuclear power plants, dams and tall buildings, has also led to the development of more refined analytical techniques for the interpretation of test data. The efforts in these closely allied fields have produced many papers on structural identification in the last five years. Since this thesis is primarily concerned with structural identification from earthquake records using a frequency domain approach, the following literature review concentrates on those methods which either are applicable to earthquake response data or employ a related method of analysis.

There are several recent survey articles on structural identification including Collins, Young and Kiefling (1972) [4]; Sage (1972) [23]; Schiff (1972) [24]; and Hart and Yao (1976) [12].

The three 1972 papers formed part of an ASME volume entitled "System Identification of Vibrating Structures: Mathematical Models from Test Data" [20]. This booklet signalled the beginning of widespread interest in the application of system identification to civil engineering structures. As suggested by the title, the primary emphasis is on the analysis of test data, but there is some consideration of earthquake response. The volume discusses many of the techniques developed in other fields which have since been adapted to civil engineering applications.

Hart and Yao (1976) provide a very thorough survey discussing many of the recent developments in the field and updating the earlier article by Collins, Young and Kiefling. The paper was presented at an ASME/EMD conference at UCLA. The conference proceedings [1] constitute a large and diverse collection of papers on structural identification.

For the earthquake engineer, two of the more readable books on system identification in general are those by Eykhoff (1974) [8] and Beck and Arnold (1977) [3].

The trial-and-error synthesized model approach was used extensively in the analysis of the response of many buildings shaken by the San Fernando earthquake [19]. These papers contain much useful data about the buildings analyzed and serve as a starting point for further studies using more advanced identification techniques. For example, many of these buildings are re-examined in this thesis (Chapters IV and V).

Another notable use of the synthesized model approach is that of Wood in his analysis of the response of JPL Building 180 to the San Fernando earthquake [33]. Starting with a modal model for each direction derived from a simple lumped mass and stiffness model for each floor, the modal periods and dampings were altered to improve the visual match of the Fourier amplitude spectra of the recorded and model acceleration responses. This building exhibited substantial period lengthening during the course of the response which complicated the identification.

A moving window transfer function approach was used by Udwadia and Trifunac [31] to study the time-variation of the effective modal periods of Millikan Library and JPL Building 180 during their response to the Borrego Mountain, Lytle Creek and San Fernando earthquakes.

Hart and Vasudevan [11] and Hart, DiJulio and Lew [10] used transfer functions to determine the periods and dampings for up to three modes in each direction for about a dozen buildings from the San Fernando response data.

The problems of using jagged Fourier spectra and the associated transfer functions have led several researchers to estimate parameters based on various integrated measures-of-fit of the frequency domain data to overcome the variability of the individual frequency ordinates. In analyzing ambient vibration test data, Vanmarcke [32] selected the modal parameters to reproduce the first three moments of the measured amplitude response spectrum over a narrow frequency band around the resonant peak. Schiff, Feil and Bogdanoff [25] chose the modal parameters to minimize a measure of error in the unsmoothed spectral estimates. Ibáñez [14] applied a similar philosophy to forced vibration test data. He selected the modal parameters to provide a least-squares match of the complex-valued Fourier response spectrum over a narrow frequency band around each resonance. For steady-state tests, Ibáñez used the frequency response expressions given by equivalent linearization techniques to estimate the parameters of mildly nonlinear systems as well.

Torkamani and Hart [27] developed a novel non-parametric method to estimate an impulse response function. Rather than selecting the impulse response function to match the response data exactly, which involves the solution of ill-conditioned equations, a specified discrepancy was allowed between the model and measured response and a smoothed impulse response function was chosen to minimize a measure of jaggedness. The identification technique was applied to a number of simulated problems, not always with entirely satisfactory results, and finally the transverse response records from the roof of the Orion Avenue Holiday Inn were analyzed segment-by-segment.

Udwadia and Marmarelis [28,17] utilized the Wiener technique of non-parametric identification to study the ambient vibration and earthquake response of Millikan Library. The first paper was concerned with linear models, while the second paper considered the second order kernels and determined the nonlinear contribution to the roof response during the strong shaking of the San Fernando earthquake.

The time-varying and hysteretic nonlinear behavior of Millikan Library during its response to the east-west component of the San Fernando earthquake was also studied by Iemura and Jennings [15].

They estimated the hysteresis loops for each cycle of response from the data. From their results, it appeared that the library exhibited deteriorating hysteretic behavior up to the time of maximum response, losing not only some of its stiffness but also some energy dissipation capacity during the large amplitude response in the early part of the earthquake. It was also found that nonstationary linear or bilinear hysteretic models with four changes of properties during the earthquake gave much better fits of the measured response than stationary linear or bilinear hysteretic models, indicating the importance of the stiffness degradation in determining the response.

Raggett [21] developed a systematic time-domain technique for identifying the modal parameters of linear models by achieving least-squares fits of the response histories. Raggett's approach was to narrow band-pass filter the measured records around the initial estimate of each modal frequency in turn and then perform a single-mode identification of each set of filtered data. Interaction between modes caused some difficulties in that even at the modal frequencies the response is not purely that of a single mode. Beck [2] used a similar method, but rather than filtering the data he subtracted from the records the contributions calculated from the model for all but the mode of interest, identifying the parameters of each mode in turn and then iterating through the modes. This approach was less affected by modal interference, producing better results using Raggett's synthesized data, although the identification of closely spaced modes was not attempted. Both Raggett and Beck obtained excellent matches to recorded earthquake data.

Beck also investigated an optimal filter approach for identifying linear modal models. This method produced good results in test problems where the system could be represented exactly by a linear model, but was unreliable in that the estimated parameter values did not always produce the required minimum of the specified identification criterion in the presence of substantial model error. This is a major drawback when identifying linear models from earthquake data since the real system is often non-linear and time-varying. The filter method was also numerically less efficient than the modal minimization method.

Besides the development and investigation of the two identification methods, Beck performed an extensive analysis of the identifiability of linear structural models from earthquake data. The uniqueness of damping and stiffness estimates derived from earthquake response measurements was also studied by Udwadia, Sharma and Shah [26,29,30]. The results of these studies are discussed in Chapter II.

Several nonlinear models have been proposed for the identification of simple frame systems from shaking table experiments. For a single degree-of-freedom system, Distefano and Rath [6] considered a nonlinear model with cubic stiffness and damping terms. They performed a series of numerical experiments with this model, adding noise to the calculated response, and compared three common identification approaches: an equation error method, an imbedding filter technique, and a Gauss-Newton quasilinear method.

The equation error method has the advantage that it is non-iterative. However, it requires measurements of the response at every degree-of-freedom, which generally makes it impractical for multi-degree-of-freedom models. Even for the single degree-of-freedom system,it requires accurate records of the acceleration, velocity and displacement. When the displacement is derived from the double integration of a noisy acceleration record, the usual procedure in practice, the "measurement error" corrupts the estimate more using this procedure than with the other two approaches.

The filter and Gauss-Newton methods both produce a least-squares fit of some response quantity and hence are known as output error methods. The filter method sequentially updates the parameter estimates as more of the response record is utilized, while the Gauss-Newton method utilizes all the data in each iteration. The Gauss-Newton method was found superior to the filtering method in the accuracy with which the parameters were estimated.

Distefano and Rath also considered a bilinear hysteretic model [7]. This study revealed a feature which is common in the identification of nonlinear models with different branch curves. Unless the initial estimates of the parameters are very accurate, the identification algorithm may quickly become out-of-step with the system response in that it may attempt to fit an elastic branch to a yielding portion of the response and vice versa.

Distefano and Rath [6] and Distefano and Pena-Pardo [5] identified the parameters of the top floor of a three-story steel frame subjected to a shaking table test, using the second floor response as the input. A cubic nonlinear model provided a much better fit to the measured response than did a linear model.

Matzen and McNiven [18] considered a Ramberg-Osgood hysteretic model for a single story steel frame. They used a Gauss-Newton method to obtain a combined least squares fit of the acceleration and displacement response. The identification from the shaking table data was complicated by time-varying behavior of the structure. In particular, the hysteresis loop during the first cycle was different from the later loops. The model reproduced the system response very closely except for the virgin loading curve.

Most of the studies reported in the literature to date have concentrated on developing an identification technique, testing it with synthesized data, and perhaps illustrating its application with earthquake data from one or two buildings. There have been very few systematic studies of the response of multi-degree-of-freedom systems. The following work concentrates on the application of an identification method for multi-degree-of-freedom linear models to the actual earthquake records, considering ten structures.

1.4 OUTLINE OF THIS WORK

In this thesis, a desire to overcome limitations of transfer function approaches has led to the development and application of a systematic identification technique using the frequency domain data to obtain information about the dynamic properties of buildings during earthquake response.

Chapter two describes the technique for identifying the parameters of the lower modes of linear models of structures from their earthquake acceleration records. Some results on identifiable parameters of linear models are used to justify the selection of the modal properties rather than the elements of the damping and stiffness matrices as the appropriate parameters to estimate. The identification algorithm, a modified Gauss-Newton iterative method to achieve a least squares match of the acceleration response transform over a specified frequency band, is described in detail. Information on the reliability of the parameter estimates which becomes available during the identification is discussed. Some variations of the basic identification technique are presented. Finally an application of the method to a test problem with synthesized data is used to illustrate its accuracy and capabilities.

The main objective of this work is to apply the identification technique to actual earthquake records to determine the achievable accuracy of linear models and the effective values of their parameters in reproducing the measured response of a variety of structures at different strengths of excitation. To this end, the earthquake records obtained from ten structures (Table 1.1) are analyzed in chapters three to five. Both reinforced concrete and steel buildings are considered, ranging in height from the seven story Holiday Inn buildings to the forty-two story Union Bank. Most of the records are from the San Fernando earthquake, but three sets of records for smaller intensity shaking in the Borrego Mountain and Lytle Creek earthquakes are also considered. The maximum ground acceleration ranged from 0.007g in JPL Building 180 during the Borrego Mountain earthquake to 0.25g in the Orion Avenue Holiday Inn during the San Fernando earthquake. For most of the buildings no structural damage occurred, but the fundamental periods were known to have lengthened considerably during the earthquake from their vibration test values. The three buildings discussed in chapter four, the two Holiday Inns and the Bank of California, were the most heavily damaged of the instrumented buildings during the San Fernando earthquake; each building received minor structural damage.

The buildings studied and their levels of vibration are summarized in Table 1.1. The response levels are categorized into four classes. The results of the identification studies are summarized in Table 6.1. The locations of the structures are shown in the map of Figure 1.1.

Chapter six presents the general conclusions of the study; specific conclusions about the individual structures are included within the relevant chapters. The ability of the linear models to represent each of the four classes of response is commented upon, along with the typical extent of period lengthening and the values of effective damping. This concluding chapter also discusses the successes and some shortcomings of the identification technique. Finally, some suggestions for future research are presented.

TABLE 1.1: SUMMARY OF STRUCTURES AND RECORDS STUDIED

Structure	Material	Stories	Horizontal Dimensions (ft)	Earthquake	Δ^*	Max. Acc. (%G) Ground	Max. Acc. (%G) Response	Response Type
JPL Building 180	Steel	9	220 × 40	Borrego Mtn.		0.7	3	A
				Lytle Creek		2	4	A
				San Fernando	15	21	38	C
Millikan Library	RC	9	75 × 69	Lytle Creek		2	5	A
				San Fernando	19	20	34	C
1900 Avenue of the Stars	Steel	27	208 × 108	"	20	8	14	A-B
Union Bank, 445 S. Figueroa	Steel	39	196 × 98	"	21	15	20	B-C
KB Valley Center, 15910 Ventura	Steel	18	165 × 82	"	14	14	22	B
Kajima International, 250 E. First	Steel	15	96 × 66	"	21	12	19	B
Sheraton-Universal, 3838 Lankershim	RC	19	184 × 58	"	15	16	18	B
Holiday Inn, 1640 S. Marengo	RC	7	150 × 61	"	22	13	41	D
Holiday Inn, 8244 Orion	RC	7	150 × 61	"	8	25	38	D
Bank of California, 15250 Ventura	RC	12	161 × 60	"	14	22	28	D

RC - Reinforced concrete

Δ^* - Distance (in miles) from center of energy release in San Fernando earthquake (near Pacoima Dam)

Response types:
- A - Small amplitudes, close to vibration test periods.
- B - Moderate amplitudes, linear models good but periods changed from vibration tests.
- C - Moderate amplitudes, time varying properties, time invariant linear models match maximum response.
- D - Minor structural damage (repairable), limit of linear models.

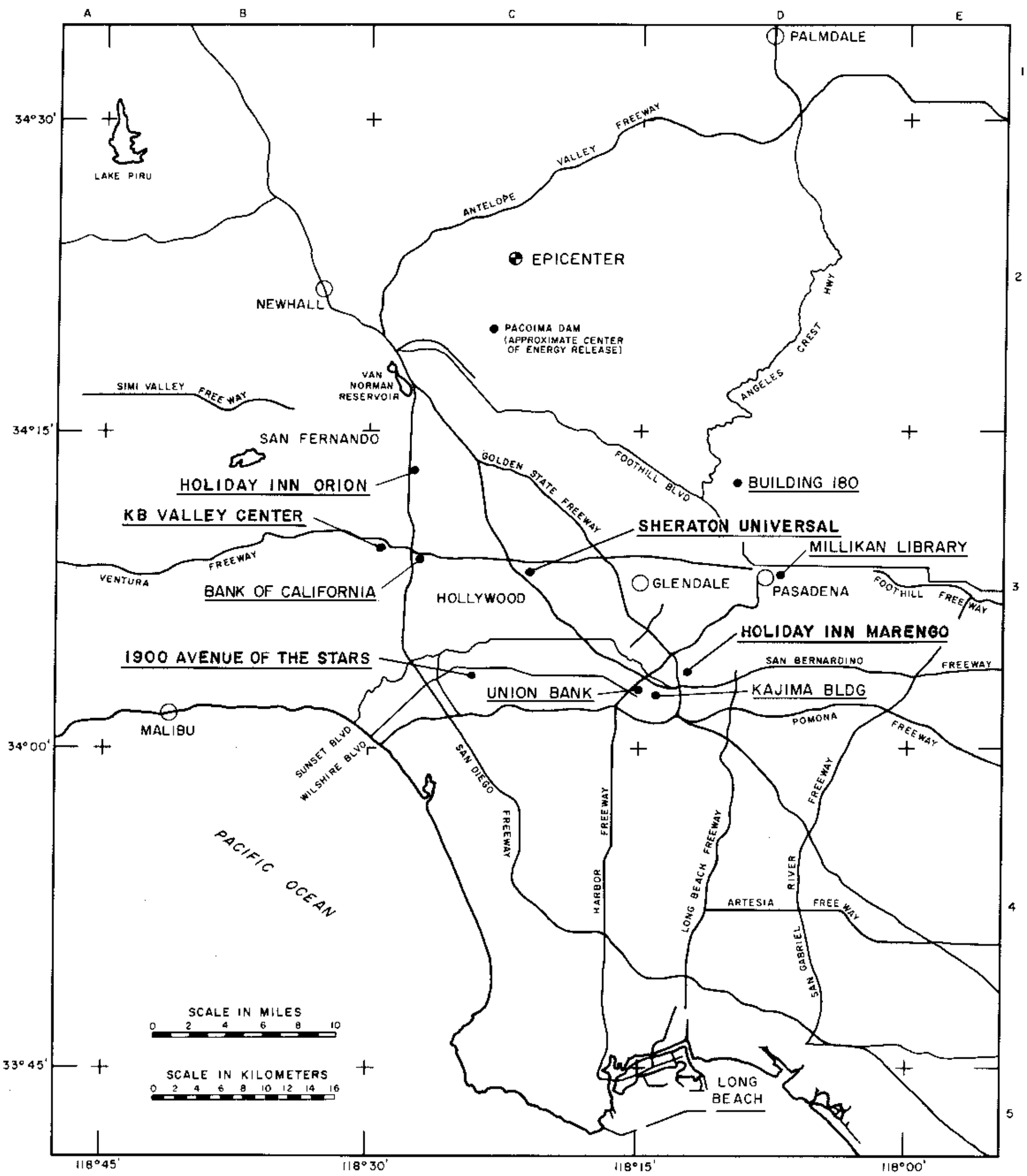


Figure 1.1 Map of the Los Angeles area showing the location of the buildings studied (adapted from Foutch, et al. [9]).

REFERENCES

1. ASCE/EMD Speciality Conference, 'Dynamic Response of Structures: Instrumentation, Testing Methods and Systems Identification,' UCLA March 30-31, 1976.

2. Beck, J.L., "Determining Models of Structures from Earthquake Records", Earthquake Engineering Research Laboratory, EERL 78-01, California Institute of Technology, Pasadena, California, June 1978.

3. Beck, J.V. and K.J. Arnold, 'Parameter Estimation in Engineering and Science', Wiley, New York 1977.

4. Collins, J.D., J.P. Young and L. Kiefling, "Methods and Applications of System Identification in Shock and Vibration," in 'System Identification of Vibrating Structures', W.D. Pilkey and R. Cohen (eds), ASME, New York, 1972.

5. Distefano, N. and B. Pena-Pardo, "System Identification of Frames under Seismic Loads," Journal Eng. Mech. Division, ASCE 102, EM2, 313-29, April 1976.

6. Distefano, N. and A. Rath, "Modeling and Identification in Nonlinear Structural Dynamics", Report No. EERC 74-15, University of California, Berkeley, 1974.

7. N. Distefano and A. Rath, "Sequential Identification of Hysteretic and Viscous Models in Structural Seismic Dynamics", Comp. Methods in Applied Mechanics and Engineering, 6, 219-32,1975.

8. Eykhoff, P., 'System Identification', Wiley and Sons, 1974.

9. Foutch, D.A., G.W. Housner and P.C. Jennings, "Dynamic Responses of Six Multistory Buildings during the San Fernando Earthquake", Report No. EERL 75-02, California Institute of Technology, Pasadena, California, October 1975.

10. Hart, G.C., R.M. DiJulio and M. Lew, "Torsional Response of High-Rise Buildings", Journal Structural Division, ASCE 101, ST2, 397-416, 1975.

11. Hart, G.C. and R. Vasudevan, "Earthquake Design of Buildings: Damping", Journal Structural Division, ASCE 101, ST1, 11-29, 1975.

12. Hart, G.C. and J.T.P. Yao, "System Identification in Structural Dynamics," Journal Engineering Mech. Division, ASCE 103, EM6, 1089-1104, 1976.

13. Hudson, D.E., "A Comparison of Theoretical and Experimental Determinations of Building Response to Earthquakes", Proc. 2nd World Conference on Earthquake Engineering, Vol. II, 1105-1120, Japan 1960.

14. Ibáñez, P., "Identification of Dynamic Structural Models from Experimental Data", Report No. UCLA-ENG-7225, UCLA, Los Angeles, California, 1972.

15. Iemura, H. and P.C. Jennings, "Hysteretic Response of a Nine-Story Reinforced Concrete Building," International Journal of Earthquake Engineering and Structural Dyn., 3, 183-241, 1974.

16. Japan National Committee on Earthquake Engineering, "Niigata Earthquake of 1964", Proc. 3rd World Conference on Earthquake Engineering Vol. III, S78-109, New Zealand 1965.

17. Marmarelis, P.Z. and F.E. Udwadia, "The Identification of Building Structural Systems II. The Nonlinear Case," Bulletin Seism. Soc. Am. 66, 153-71, February 1976.

18. Matzen, V.C. and H.D. McNiven, "Investigation of the Inelastic Characteristics of a Single Story Steel Structure Using System Identification and Shaking Table Experiments", Report No. EERC-76-20, University of California, Berkeley, 1976.

19. Murphy, L.M. (ed.), 'San Fernando, California, Earthquake of February 9, 1971', U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration (NOAA), Washington D.C., 1973.

20. Pilkey, W.D. and R. Cohen (eds.), 'System Identification of Vibrating Structures: Mathematical Models from Test Data', ASME, New York, 1972.

21. Raggett, J.D., "Time Domain Analysis of Structural Motions," Preprint 2209, ASCE National Structural Engineering Meeting, Cincinnati, Ohio, 1974.

22. Raggett, J.D., "Estimating Damping of Real Structures," Journal Structural Division, ASCE, 101, ST9, 1823-35, September 1975.

23. Sage, A.P., "System Identification History, Methodology, Future Prospects", in 'System Identification of Vibrating Structures', W.D. Pilkey and R. Cohen (ed.), ASME, New York, 1972.

24. Schiff, A.J., "Identification of Large Structures Using Data from Ambient and Low Level Excitations", in 'System Identification of Vibrating Structures', W.D. Pilkey and R. Cohen (eds.), ASME, New York, 1972.

25. Schiff, A.J., P.J. Feil and J.L. Bogdanoff, "Estimating Structural Parameters from Response Data", Proc. 5th World Conf. Earthquake Engineering, 2558-2567, Rome, 1973.

26. Shah, P.C. and F.E. Udwadia, "A Methodology for Optimal Sensor Locations for Identification of Dynamic Systems", Journal Appl. Mech., 45, 1, 188-96, March 1978.

27. Torkamani, M.A. and G.C. Hart, "Building System Identification Using Earthquake Data", Report No. UCLA-ENG-7507, UCLA, Los Angeles, California, 1975.

28. Udwadia, F.E. and P.Z. Marmarelis, "The Identification of Building Structural Systems I. The Linear Case", Bulletin Seism. Soc. Am., 66, 125-51, 1976.

29. Udwadia, F.E. and P.C. Shah, "Identification of Structures Through Records Obtained during Strong Earthquake Ground Motion", Journal Engineering for Industry, ASME, 98, 1347-62, 1975.

30. Udwadia, F.E., D.K. Sharma and P.C. Shah, "Uniqueness of Damping and Stiffness Distributions in the Identification of Soil and Structural Systems," Journal Appl. Mech. 45, 1, 181-7, March 1978.

31. Udwadia, F.E. and M.D. Trifunac, "Time and Amplitude Dependent Response of Structures", International Journal Earthquake Engineering and Structural Dyn., 2, 359-78, 1974.

32. Vanmarcke, E.H., "Method of Spectral Moments to Estimate Structural Damping", in 'Stochastic Problems in Dynamics', B.L. Clarkson (ed.), Proc. Symposium held at University of Southampton, Pitman Press, London, 1977.

33. Wood, J.H., "Analysis of the Earthquake Response of a Nine-Story Steel Frame Building during the San Fernando Earthquake", Report No. EERL 72-04, California Institute of Technology, Pasadena, California, 1972.

II. A FREQUENCY DOMAIN IDENTIFICATION TECHNIQUE FOR LINEAR STRUCTURAL MODELS

2.1 LINEAR STRUCTURAL MODELS

A frequency domain technique for the identification of the modal parameters of linear structural models from recorded earthquake response is developed in this chapter.

Such models are certainly adequate for weaker, non-damaging excitations, and for response to stronger shaking if the nonlinearities are not too severe. For strong earthquake response, structures respond nonlinearly to a significant degree, but the observed nonlinear and time-varying behavior can be approximated by considering a series of linear models appropriate for different segments of the response, thereby tracing the changes in the effective linear parameters. This approach is similar to the equivalent linearization techniques employed in some non-linear analyses (e.g. Iwan and Gates, [5]).

For the most part the models are also assumed planar, that is the response in a given direction is caused only by the component of the input in that direction. Occasionally the planar assumption is relaxed to allow response in a given direction caused by both components of the horizontal ground motion. This permits investigation of the possibility of horizontal coupling.

The parameters estimated from the earthquake data are those of the dominant modes of the response rather than the elements of the stiffness and damping matrices. Considerations of identifiability, that is, the ability to uniquely determine the parameters from the input and output records, and accuracy of estimation show that the modal properties are the appropriate parameters to identify from response records at a limited number of locations in the structure (Beck, [1]).

The identification of the modal parameters is performed using frequency domain data, specifically the complex-valued finite Fourier transforms of the recorded ground acceleration and absolute acceleration response of the structure. The transforms are calculated using a fast Fourier transform algorithm.

Linear models are considered in this study primarily because of their simplicity and because of their widespread use in response calculations, particularly for design. Linear models also serve as a natural starting point for nonlinear studies. For example, the nonlinear response of some structures at large amplitudes has been calculated successfully using the concept of equivalent linearization, in which the structure is modeled by a linear system with properties that vary with amplitude or time.

In view of these facts, and the difficulty of the problem, it appears justifiable to adopt an equivalent linear model for the inverse problem of establishing the model from the recorded input and response data. In fact, the degree of success of matching the recorded response using linear models will provide an evaluation of the usefulness of this approach for earthquake response calculations. In addition, it is hoped that a consideration of the variation of the effective linear parameters in conjunction with the recorded nonlinear response will indicate which nonlinear mechanisms are important. For example, a dependence of the effective fundamental period mainly on the present amplitude of response would indicate amplitude-dependent but non-degrading stiffness, while an increase of the period when the preceding maximum amplitude is exceeded with little recovery for subsequent smaller amplitude oscillations would suggest a degradation of stiffness dictated by the maximum amplitude of response.

2.2 THE EQUATIONS OF MOTION FOR LINEAR MODELS

The equations of motion for linear, time-invariant, planar structures with classical normal modes will now be presented.

The equations of motion at each of N_f degrees of freedom relating the displacement, $\underset{\sim}{x}$, velocities $\dot{\underset{\sim}{x}}$, and accelerations $\ddot{\underset{\sim}{x}}$, relative to the ground, to the ground acceleration $\ddot{z}_g$ may be written:

$$[M]\ddot{\underset{\sim}{x}}(t) + [C]\dot{\underset{\sim}{x}}(t) + [K]\underset{\sim}{x}(t) = -[M]\underset{\sim}{1}\,\ddot{z}_g(t) \qquad (2\text{-}1)$$

The mass matrix [M] and the stiffness matrix [K] must be symmetric and positive definite, while the damping matrix [C] is symmetric and positive semi-definite. It is further assumed that the form of [C] is such that classical oscillatory modes exist. All components of the vector $\underset{\sim}{1}$ are unity. Equation (2-1) is the matrix form of the equations of motion for this system.

The equations of motion may be modally decomposed. Let the mode shape vectors $\underset{\sim}{\phi}_r$ satisfy the eigenvalue equations

$$[K]\underset{\sim}{\phi}_r = \omega_r^2 [M]\underset{\sim}{\phi}_r \qquad r=1,\ldots N_f \qquad (2\text{-}2)$$

Define the matrix of mode shape vectors, whose component Φ_{pr} is the rth mode shape at position p:

$$[\Phi] = \Phi_{pr} = [\underset{\sim}{\phi}_1 \; \underset{\sim}{\phi}_2 \cdots \underset{\sim}{\phi}_r \cdots \underset{\sim}{\phi}_{N_f}] \qquad (2\text{-}3)$$

The mode shape vectors satisfy orthogonality conditions with respect to the mass matrix [M]:

$$\left.\begin{aligned} \underset{\sim}{\phi}_r^T[M]\underset{\sim}{\phi}_s &= 0 \qquad & r \neq s \\ &\neq 0 \qquad & r = s \end{aligned}\right\} \qquad (2\text{-}4)$$

Also, the damping matrix satisfies

$$[C]\underset{\sim}{\phi}_r = 2\zeta_r\omega_r[M]\underset{\sim}{\phi}_r \qquad 0 \leq \zeta_r < 1 \qquad (2\text{-}5)$$

The relative displacement vector can be expressed as the sum of the modal components

$$\underset{\sim}{x}(t) = [\Phi]\underset{\sim}{\xi}(t) \qquad (2\text{-}6)$$

or

$$x_p(t) = \sum_{r=1}^{N_f} \Phi_{pr}\xi_r(t) = \sum_{r=1}^{N_f} x_{pr}(t) \qquad (2\text{-}7)$$

Substituting (2-6) into the equation of motion (2-1), multiplying by $[\Phi]^T$, and dividing the rth mode equation by $\underset{\sim}{\phi}_r^T[M]\underset{\sim}{\phi}_r$ produces

$$\ddot{\xi}_r(t) + 2\zeta_r\omega_r\dot{\xi}_r(t) + \omega_r^2\xi_r(t) = -\frac{\underset{\sim}{\phi}_r^T[M]\underset{\sim}{1}}{\underset{\sim}{\phi}_r^T[M]\underset{\sim}{\phi}_r}\ddot{z}_g(t) \qquad (2\text{-}8)$$

Multiplying by Φ_{pr} gives the equation for the rth mode displacement at p:

$$\ddot{x}_{pr}(t) + 2\zeta_r\omega_r\dot{x}_{pr}(t) + \omega_r^2 x_{pr}(t) = -\Phi_{pr}\frac{\underset{\sim}{\phi}_r^T[M]\underset{\sim}{1}}{\underset{\sim}{\phi}_r^T[M]\underset{\sim}{\phi}_r}\ddot{z}_g(t) \qquad (2\text{-}9)$$

or

$$\ddot{x}_{pr}(t) + a_r\dot{x}_{pr}(t) + b_r x_{pr}(t) = -c_{pr}\ddot{z}_g(t) \tag{2-10}$$

where

$$a_r = 2\zeta_r\omega_r \quad , \quad b_r = \omega_r^2 \quad , \quad c_{pr} = \Phi_{pr}\frac{\underset{\sim}{\phi}_r^T[M]\underset{\sim}{1}}{\underset{\sim}{\phi}_r^T[M]\underset{\sim}{\phi}_r} \tag{2-11}$$

Equations (2-9) or (2-10) together with (2-7) are the modal form of the equations of motion. The parameters are ζ_r, the fraction of critical viscous damping in mode r, the rth mode natural frequency (in radians/sec) ω_r, and the effective rth mode participation factor at p, c_{pr}, or alternatively, a_r, b_r and c_{pr}.

It should be noted that the measured quantities are usually the ground acceleration $\ddot{z}_g(t)$ and the absolute acceleration response $\ddot{x}_p(t) + \ddot{z}_g(t)$ at one or more positions in the structure. Relative velocities and displacements, $\dot{x}_p(t)$ and $x_p(t)$, can be derived from the records by subtraction and integration. However, the integration process generally produces long period errors, usually most obvious in the displacement histories.

The modal equations can be transformed to the frequency domain by taking Fourier transforms.

The finite Fourier transform $F_T(\omega)$ of f(t) over a record length T is defined as

$$F_T(\omega) = \int_0^T f(t)e^{-i\omega t}dt \tag{2-12}$$

Let $A_{pT}(\omega)$ = the finite Fourier transform of the absolute acceleration $\ddot{x}_p(t) + \ddot{z}_g(t)$

$Z_T(\omega)$ = the finite Fourier transform of the ground acceleration $\ddot{z}_g(t)$

$X_{pT}^{(r)}$ = the finite Fourier transform of $x_{pr}(t)$

$V_{pT}(\omega)$ = the finite Fourier transform of the relative velocity $\dot{x}_p(t)$

$X_{pT}(\omega)$ = the finite Fourier transform of the relative displacement $x_p(t)$.

In practice, these transforms will also be discrete, calculated by the use of a fast Fourier transform (FFT) algorithm which produces the complex-valued transforms at N equally spaced frequencies ω_n from 2N equally spaced samples of a record of length T where

$$\omega_n = n\Delta\omega = \frac{2\pi n}{T} \qquad n = 0,1,\ldots,N-1 \tag{2-13}$$

The transforms of the modal velocities and modal accelerations, for the sampled frequencies, can be expressed in terms of $X_{pT}^{(r)}(\omega)$ as:

$$\int_0^T \dot{x}_{pr}(t)e^{-i\omega t}dt = x_{pr}(T)e^{-i\omega T} - x_{pr}(0) + i\omega X_{pT}^{(r)}(\omega)$$

$$= x_{pr}(T) - x_{pr}(0) + i\omega X_{pT}^{(r)}(\omega) \tag{2-14}$$

$$\int_0^T \ddot{x}_{pr}(t)e^{-i\omega t}dt = \dot{x}_{pr}(T) - \dot{x}_{pr}(0) + i\omega[x_{pr}(T) - x_{pr}(0)] - \omega^2 X_{pT}^{(r)}(\omega) \tag{2.15}$$

Transforming the rth mode equation (2.10) produces, for the sampled frequencies,

$$X_{pT}^{(r)}(\omega) = \frac{-c_{pr}[b_r - \omega^2 - i\omega a_r]}{\left(b_r - \omega^2\right)^2 + \omega^2 a_r^{\,2}} Z_T(\omega)$$

$$- \frac{[b_r - \omega^2 - i\omega a_r]}{\left(b_r - \omega^2\right)^2 + \omega^2 a_r^{\,2}} [\dot{x}_{pr}(T) - \dot{x}_{pr}(0)] \tag{2.16}$$

$$- \frac{[a_r b_r + i\omega(b_r - \omega^2 - a_r^{\,2})]}{\left(b_r - \omega^2\right)^2 + \omega^2 a_r^{\,2}} [x_{pr}(T) - x_{pr}(0)]$$

Combining the modal contributions gives the following expressions for the transformed displacement, velocity and acceleration

$$X_{pT}(\omega) = \left[\sum_{r=1}^{N_f} \frac{-\left(b_r - \omega^2\right) + i\omega a_r}{\left(b_r - \omega^2\right)^2 + \omega^2 a_r^{\,2}} c_{pr} \right] Z_T(\omega)$$

$$- \sum_{r=1}^{N_f} \frac{\left(b_r - \omega^2\right) - i\omega a_r}{\left(b_r - \omega^2\right)^2 + \omega^2 a_r^{\,2}} \left[\dot{x}_{pr}(T) - \dot{x}_{pr}(0)\right] \tag{2-17}$$

$$\sum_{r=1}^{N_f} \frac{a_r b_r + i\omega\left(b_r - \omega^2 - a_r^{\,2}\right)}{\left(b_r - \omega^2\right)^2 + \omega^2 a_r^{\,2}} \left[x_{pr}(T) - x_{pr}(0)\right]$$

$$V_{pT}(\omega) = \left[\sum_{r=1}^{N_f} \frac{-\omega^2 a_r - i\omega \left(b_r - \omega^2\right)}{\left(b_r - \omega^2\right)^2 + \omega^2 a_r^2} c_{pr} \right] Z_T(\omega)$$

$$- \sum_{r=1}^{N_f} \frac{\omega^2 a_r + i\omega \left(b_r - \omega^2\right)}{\left(b_r - \omega^2\right)^2 + \omega^2 a_r^2} v_{pr} \qquad (2\text{-}18)$$

$$\sum_{r=1}^{N_f} \frac{\left(b_r - \omega^2\right) b_r - i\omega a_r b_r}{\left(b_r - \omega^2\right)^2 + \omega^2 a_r^2} d_{pr}$$

$$A_{pT}(\omega) = \left[1 + \sum_{r=1}^{N_f} \frac{\omega^2 \left(b_r - \omega^2\right) - i\omega^3 a_r}{\left(b_r - \omega^2\right)^2 + \omega^2 a_r^2} c_{pr} \right] Z_T(\omega)$$

$$+ \sum_{r=1}^{N_f} \frac{b_r \left(b_r - \omega^2\right) + \omega^2 a_r^2 - i\omega^3 a_r}{\left(b_r - \omega^2\right)^2 + \omega^2 a_r^2} v_{pr} \qquad (2\text{-}19)$$

$$+ \sum_{r=1}^{N_f} \frac{a_r b_r \omega^2 + i\omega b_r \left(b_r - \omega^2\right)}{\left(b_r - \omega^2\right)^2 + \omega^2 a_r^2} d_{pr}$$

The parameters v_{pr} and d_{pr} are the differences in the rth mode velocities and displacements at position p between the beginning and the end of the record segment of duration T.

$$v_{pr} = \dot{x}_{pr}(T) - \dot{x}_{pr}(0) \qquad (2\text{-}20)$$

$$d_{pr} = x_{pr}(T) - x_{pr}(0) \qquad (2\text{-}21)$$

When the parameters a_r, b_r, c_{pr}, d_{pr} and v_{pr} and the input $\ddot{z}_g(t)$ from $t = 0$ to $t = T$ are known, the initial modal displacements and velocities, $x_{pr}(0)$ and $\dot{x}_{pr}(0)$, of the model can be simply calculated. First the forced response with zero initial conditions is calculated by solving equation (2.10). The forced displacements and velocities at time T are subtracted from d_{pr} and v_{pr} to give the free vibration response d_{pr}^* and v_{pr}^*. The following analytical expressions (2-22) and (2-23) for the free vibration response at time T to initial conditions at $t = 0$ can then be inverted to produce the initial modal displacements and velocities $x_{pr}(0)$ and $\dot{x}_{pr}(0)$.

$$d_{pr}^* = e^{-\frac{1}{2}a_r T}\left[x_{pr}(0)\cos\sqrt{b_r - \frac{a_r^2}{4}}\,T + \frac{[\dot{x}_{pr}(0) + \frac{1}{2}a_r x_{pr}(0)]}{\sqrt{b_r - \frac{a_r^2}{4}}}\sin\sqrt{b_r - \frac{a_r^2}{4}}\,T\right] \quad (2.22)$$

$$v_{pr}^* = e^{-\frac{1}{2}a_r T}\left[\dot{x}_{pr}(0)\cos\sqrt{b_r - \frac{a_r^2}{4}}\,T - \frac{[\frac{1}{2}a_r\dot{x}_{pr}(0) + b_r x_{pr}(0)]}{\sqrt{b_r - \frac{a_r^2}{4}}}\sin\sqrt{b_r - \frac{a_r^2}{4}}\,T\right] \quad (2.23)$$

The expression for the transform of the acceleration response, (2-19), is the model equation for the identification technique developed in this work. The displacement and velocity transform expressions (2-17) and (2-18) could also be used, but these quantities are poorer in high frequency content so may not yield information about as many modes. As indicated during the development of these equations, the matrix form (2-1) and the time domain modal equations (2-7) and (2-10) are other possible starting points. The reasons for the selection of the frequency domain modal equations in this work will now be discussed.

2.3 IDENTIFIABILITY OF LINEAR MODELS

Two important considerations in the selection of the form of the model to be identified are whether the measured input and output records of the system allow the parameters of the model to be determined uniquely, and the effect of measurement noise and model error on the accuracy of the estimates of the model parameters. Some aspects of these questions can be answered prior to the identification, while quantities calculated in the course of the identification provide information on other aspects. Beck [1] has considered these problems in detail, and his results are discussed below.

An obvious reason for considering the uniqueness of the parameter estimates in earthquake engineering is that generally the response is measured at only a very few locations in the structure. Usually, the data available are the basement acceleration and the response acceleration at the roof, and possibly another response record from a location near mid-height of the structure. It is possible to imagine different models which produce the same response to the excitation at the measurement locations but different responses elsewhere in the structure. In fact, for simple linear models such sets of companion models have been reported as examples [1,6,7].

The question of the uniqueness of the parameter estimates which can be determined from measurements of the excitation and response of the system can be approached by consideration of the identifiability of the model class. Identifiability, or strictly global identifiability, means that knowledge of the noise-free input and output of a model at specified locations allows the parameters to be determined uniquely. In practice, it is desired to determine the parameters of a model uniquely from measurements of the input and output of a real system, in which the records are contaminated by noise. Noisy data can corrupt the parameter estimates even if the system can be represented exactly by the given type of model. The identification is also complicated by model error, in that the model within a specified class which best fits the data is not a completely accurate representation of the structure. However, identifiability of the model class from noise-free data is a necessary condition for uniqueness of the model determined from noise-corrupted records of a non-ideal system.

Identifiability is also a function of the response location, since a given model may not be identifiable from a response record at one location, but may be identifiable from a set of response records at several locations, or a single record from some other location.

There is a less stringent property called local identifiability in which the input-output records specify a finite number of distinct possibilities for the model. This degree of identifiability may be sufficient to determine uniquely the true model if prior information is available to discriminate among the choices.

First Beck studied the general case where the only restrictions were that the stiffness matrix [K] be symmetric and positive definite and the damping matrix [C] be symmetric and positive semi-definite and be such that classical oscillatory modes exist. It was assumed that the mass matrix [M] was diagonal and known. The following properties were proved:

1) A knowledge of ω_r, ζ_r and $\underset{\sim}{\phi}_r$ for all modes uniquely defines [C] and [K], with [M] known, and vice versa.

2) If the base motion $\ddot{z}_g(t)$ and the response at some position p (either the displacement $x_p(t)$, the velocity $\dot{x}_p(t)$, or the acceleration $\ddot{x}_p(t)$) are known then

   (i) c_{pr}, $x_{pr}(0)$ and $\dot{x}_{pr}(0)$ are determined uniquely for all modes

   (ii) ω_r and ζ_r are determined uniquely if the rth mode contributes to the response at p, i.e., $c_{pr} \neq 0$, or $x_{pr}(0) \neq 0$ or $\dot{x}_{pr}(0) \neq 0$.

The important part of this result, as far as the identifiability of the stiffness and damping matrices from response records at a limited number of locations is concerned, is that the only mode shape information directly estimable is the effective participation factors c_{pr}. These can be found for all modes, but only at the locations where the response is measured. It is knowledge of the full mode shape matrix $[\Phi]$ for all modes at all positions that is required to define [K] and [C] uniquely. This limited information leads to the results:

3) In general, all the elements of [K] and [C] can be determined uniquely (i.e., [K] and [C] are globally identifiable) only if the response is measured at all degrees of freedom.

4) Local identifiability of [K] and [C] requires measurement of the response at half or more of the degrees of freedom.

Results (3) and (4) impose severe restrictions on the estimation of the damping and stiffness matrices. Usually there are response records at no more than two locations in the structure, allowing a locally identifiable model with a maximum of four degrees of freedom. This provides unsatisfactory resolution of the stiffness and damping distributions for most applications in earthquake engineering. For example, it is usually desired to have at least one degree-of-freedom per floor for a building, except possibly for very tall structures where fewer degrees-of-freedom may be acceptable.

Fortunately, the situation regarding the identifiability of the stiffness and damping matrices improves somewhat when the class of models is restricted to linear chain systems, which greatly reduces the number of independent matrix elements for the same number of degrees-of-freedom. Chain models retain sufficient resolution to be of practical use, and are commonly used where it is appropriate for each floor to be represented as a lumped mass linked by horizontally-acting spring elements to the masses above and below. Such models are not suitable for very tall buildings for example, where column-shortening "bending" behavior rather than "shear" is significant.

Repeating an earlier result of Udwadia, Shah and Sharma [6 , 7], Beck showed:

5) For a linear chain system, [K] and [C] are locally identifiable from a knowledge of the input ground motion and the response at one location, and are globally identifiable if that location is the first mass above the ground.

This result makes the outlook for the use of a matrix approach in the identification appear more promising for a chain model than for a general linear model. However, there are still problems for practical applications. The local identifiability result is not always useful as there are N_f! possible models for an N_f degree-of-freedom system (i.e., an N_f story structure for the usually desired spatial resolution) whose response is measured at the roof.

Similarly the globally identifiable result for response records from the first floor is not applicable for most presently available records -- these were not obtained at the first floor. Moreover, a first floor location is not necessarily the best for future instrumentation systems, since the effect of measurement noise on the accuracy of the parameter estimates must be considered. The amplitude of the response at the first floor is generally the smallest in the structure, making its measurement the most sensitive to noise. Furthermore, Beck has shown that the stiffnesses are determined from the ratio of the limits of the Fourier transforms of the displacements on adjacent floors as the frequency tends to infinity. This severely limits the applicability of this approach since the high frequency response is small and hence is sensitive to corruption by noise. The calculation of the ratio of the high frequency components from different locations is ill-conditioned, as can be seen by examination of calculated transforms.

These considerations of identifiability and accuracy suggest that the appropriate properties to estimate from seismic response data are the modal parameters rather than the damping and stiffness matrices. Theoretically, in the noise-free case the frequencies, dampings and effective participation factors at the locations of the measured responses can be estimated for all modes contributing to the response. In practice, the signal at high frequencies will be small and affected by noise, and most of the response will be contributed by a few of the lower modes, so only the parameters of the dominant modes of the response can be estimated.

For the modal model, the time and frequency domain versions of the equation of motion are equivalent, and either may be used for the identification process. There appears to be no persuasive reason for preferring one approach to the other. One of the motivating factors in this work was the desire to extract useful information from the frequency domain data following some initial experience with the difficulties of the transfer function approach. Consequently, in this thesis a frequency domain identification technique is developed for linear structural models.

2.4 THE ERROR CRITERION

The identification is performed by selecting the parameters to obtain a least squares fit of the transform of the model response (equation 2-19) to the transform of the measured response acceleration over a specified frequency band. That is, we seek to minimize

$$J = \sum_{\ell=\ell_{min}}^{\ell_{max}} \left| A(\ell\Delta\omega) - A_{pT}(\ell\Delta\omega) \right|^2 \qquad (2\text{-}24)$$

with respect to the parameters a_r, b_r, c_{pr}, d_{pr} and v_{pr}, $(r=1,\ldots,N_m)$, of the N_m modes of the model. In equation (2.24)

A = the discrete finite Fourier transform of the measured response acceleration at position p in the structure calculated by a fast Fourier transform algorithm from 2N equally spaced data points of a segment of record of duration T

A_{pT} = the model response transform given by equation (2-19)

$\Delta\omega = \frac{2\pi}{T}$

$0 < \ell_{min} < \ell_{max} \leq N-1$, ℓ an integer

In the presentation of results later, a normalized error E will be reported. E is defined as the mean square error divided by the mean square response, taken over the same frequency band and for the same segment of record.

$$E = \frac{\sum_{\ell_{min}}^{\ell_{max}} |A(\ell\Delta\omega) - A_{pT}(\ell\Delta\omega)|^2}{\sum_{\ell_{min}}^{\ell_{max}} |A(\ell\Delta\omega)|^2} \qquad (2-25)$$

From Parseval's theorem, the error criterion is identical to a time domain least squares fit of the model response acceleration to the measured acceleration if all the FFT frequency points are used in the identification. If the frequency band is chosen to include all the significant response, the estimated parameters are essentially equal to those obtained from a fit in the time domain. A similar frequency domain error criterion has been used by Ibáñez [3] in applications to vibration test data.

2.5 THE IDENTIFICATION ALGORITHM

The least-squares minimization of J in equation (2-24) is performed using an iterative Gauss-Newton type approach to solve the nonlinear algebraic equations which result from setting to zero the partial derivatives of J with respect to the parameters. The algorithm takes advantage of the linearity of the equations with respect to the modal participation factors and the modal displacement and velocity differences. The technique ensures that the error is reduced at each iteration.

In the following the subscript p denoting the dependence on position is dropped.

Denote the parameters occurring nonlinearly in the expression for A_T by

$$\underset{\sim}{\alpha}^T = (a_1,\ldots a_{N_m}, b_1,\ldots b_{N_m}) \qquad (2\text{-}26)$$

and the linearly occurring parameters by

$$\underset{\sim}{\beta}^T = (c_1,\ldots c_{N_m}, d_1,\ldots d_{N_m}, v_1\ldots v_{N_m}) \qquad (2\text{-}27)$$

The complete vector of parameters is denoted by

$$\underset{\sim}{\gamma}^T = \left(\underset{\sim}{\alpha}^T, \underset{\sim}{\beta}^T\right) \qquad (2\text{-}28)$$

For a local minimum of J with respect to the parameters of $\underset{\sim}{\gamma}$:

$$\frac{\partial J}{\partial \underset{\sim}{\gamma}} = \underset{\sim}{0} \qquad (2\text{-}29)$$

and the second derivative matrix

$$[S] = \nabla_{\underset{\sim}{\gamma}}(\nabla_{\underset{\sim}{\gamma}} J) = \left[\frac{\partial^2 J}{\partial \gamma_i \, \partial \gamma_j}\right] \qquad i=1,\ldots 5N_m, j=1,\ldots 5N_m \tag{2-30}$$

must be positive definite (at the minimum of J)

The individual equations of (2-29) have the form

$$\frac{\partial J}{\partial \gamma_i} = 0 = -2 \sum_{\ell=\ell_{min}}^{\ell_{max}} \left\{ R_e[A(\ell\Delta\omega) - A_T(\ell\Delta\omega)]R_e\left[\frac{\partial A_T(\ell\Delta\omega)}{\partial \gamma_i}\right] + I_m[A(\ell\Delta\omega) - A_T(\ell\Delta\omega)]I_m\left[\frac{\partial A_T(\ell\Delta\omega)}{\partial \gamma_i}\right] \right\} \quad i=1,\ldots 5N_m \tag{2-31}$$

The sets of equations

$$\frac{\partial J}{\partial \underset{\sim}{a}} = \underset{\sim}{0} \quad , \quad \frac{\partial J}{\partial \underset{\sim}{b}} = \underset{\sim}{0} \tag{2-32}$$

are nonlinear with respect to all the parameters in $\underset{\sim}{\gamma}$. However, the equations

$$\frac{\partial J}{\partial \underset{\sim}{c}} = \underset{\sim}{0} \quad , \quad \frac{\partial J}{\partial \underset{\sim}{d}} = \underset{\sim}{0} \quad , \quad \frac{\partial J}{\partial \underset{\sim}{v}} = \underset{\sim}{0} \tag{2-33}$$

are linear in the parameters $\underset{\sim}{c}$, $\underset{\sim}{d}$, and $\underset{\sim}{v}$, although nonlinear in $\underset{\sim}{a}$ and $\underset{\sim}{b}$.

This suggests a two-part iterative algorithm to take advantage of the linearity of equations (2-33) with respect to $\underset{\sim}{c}$, $\underset{\sim}{d}$ and $\underset{\sim}{v}$. First, initial estimates are chosen for all parameters. Then the nonlinear equations (2-32) are solved approximately by a modified Gauss-Newton method to produce new values of $\underset{\sim}{a}$ and $\underset{\sim}{b}$. The linear equations (2-33) are then solved exactly for the $\underset{\sim}{c}$, $\underset{\sim}{d}$ and $\underset{\sim}{v}$ corresponding to the latest values of $\underset{\sim}{a}$ and $\underset{\sim}{b}$. The process is repeated until a selected convergence criterion is satisfied. The algorithm is now described in detail.

Define the partial Hessian matrix of J with respect to the parameters $\underset{\sim}{\alpha}$ as

$$(PH)_{ij} = 2 \sum_{\ell=\ell_{min}}^{\ell_{max}} \left\{ R_e\left(\frac{\partial A_T(\ell\Delta\omega)}{\partial\alpha_i}\right) R_e\left(\frac{\partial A_T(\ell\Delta\omega)}{\partial\alpha_j}\right) + I_m\left(\frac{\partial A_T(\ell\Delta\omega)}{\partial\alpha_i}\right) I_m\left(\frac{\partial A_T(\ell\Delta\omega)}{\partial\alpha_j}\right) \right\} \quad i,j=1,..2 \times N_m \tag{2-34}$$

The algorithm for stepping from iteration K to iteration K + 1 is as follows:

1) For the first iteration make initial estimates for all parameters; for other iterations take the latest estimates.

2) Solve

$$\left[PH(\underset{\sim}{\chi}^K)\right]\Delta\underset{\sim}{\alpha}^{K+1} = -\frac{\partial J(\underset{\sim}{\chi}^K)}{\partial\underset{\sim}{\alpha}} \tag{2-35}$$

to obtain $\Delta\underset{\sim}{\alpha}^{K+1}$. This is the Gauss-Newton formula for solving $\frac{\partial J}{\partial\underset{\sim}{\alpha}} = 0$. It is based on a first order Taylor series expansion of $A_T(\underset{\sim}{\alpha}, \underset{\sim}{\beta}^K, \omega)$ about $\underset{\sim}{\alpha}^K$ in the expression for J.

3) Perform a line search for the minimum of J in the direction of $\Delta\underset{\sim}{\alpha}^{K+1}$, i.e., find the s, and corresponding $\underset{\sim}{\alpha}_s^{K+1}$, which minimizes J

$$\underset{\sim}{\alpha}_s^{K+1} = \underset{\sim}{\alpha}^K + s\Delta\underset{\sim}{\alpha}^{K+1} \tag{2-36}$$

The line search is implemented by taking successive increments of s (usually ½), finding $\underset{\sim}{\alpha}_{s_i}^{K+1}$ corresponding to $s=s_i$ and solving for the corresponding $\underset{\sim}{\beta}_{s_i}^{K+1}$ from

$$\frac{\partial J\left(\underset{\sim}{\alpha}_{s_i}^{K+1}\right)}{\partial \underset{\sim}{\beta}} = 0 \qquad (2\text{-}37)$$

J_{s_i} is evaluated for $\left(\underset{\sim}{\alpha}_{s_i}^{K+1}, \underset{\sim}{\beta}_{s_i}^{K+1}\right)$, and the process continued until $J_{s_{i+1}}$ increases from J_{s_i}. At the beginning of the line search J_{s0}, for $s_0 = 0$, is known from the previous iteration parameters $(\underset{\sim}{\alpha}^K, \underset{\sim}{\beta}^K)$. The line search terminates with three successive values such that $J_{s_i} < J_{s_{i-1}}$ and $J_{s_i} < J_{s_{i+1}}$ for $s_{i-1} < s_i < s_{i+1}$, so a minimum of J in the direction of $\Delta\alpha^{K+1}$ lies between s_{i-1} and s_{i+1}. A parabolic fit in s is then made through the three points and $s = s_{min}$ corresponding to the minimum of the parabola evaluated. The final values of $\left(\underset{\sim}{\alpha}^{K+1}, \underset{\sim}{\beta}^{K+1}\right)$ are found and J^{K+1} calculated.

If J_{s1} is larger than J_{s0}, the search is performed by halving rather than incrementing s, finishing again with a parabolic fit through three points.

4) Check the convergence criteria. If the fractional changes between iteration K and K+1 of all a_r, b_r and c_r are less than a preset tolerance, convergence is assumed to have occurred and iteration is stopped; otherwise the iterative process is repeated. There is a limit set to the number of iterations performed.

To utilize the identification algorithm the number of modes to be included in the model and initial estimates of the parameters must be chosen, and the time segment and frequency band for which the identification is to be performed must be selected.

The usual approach is first to match the response data with a single time-invariant model. The fast Fourier transform algorithm imposes some restrictions in that the length of the time segment T which can be used must be $2N\Delta t$, where N is a power of 2 and Δt is the sample interval, whose standard value is 0.02 seconds. This produces typical record lengths of 20.48 seconds ($N = 512$) or 40.96 seconds ($N = 1024$). This restriction could be overcome, if necessary, by redefining the data at other intervals.

When a time-invariant model provides a poor fit of the data, or when the estimated parameter values are considerably different from vibration test values, shorter time segments can be used to study the time variation of the equivalent linear parameters. In this type of study there is a basic conflict between the desire to use short time segments to obtain estimates of the "instantaneous" effective values of the parameters, and the need to have a segment long enough to obtain adequate frequency resolution and to reduce the error in the estimates introduced by measurement noise in the data. Experience has shown that segment lengths of approximately four times the fundamental period provide adequate resolution, although occasionally a duration as small as two periods can provide a good match of the response.

The number of modes required in the model to achieve a good match of the response data is determined by starting with one or two modes, and then performing successive identifications with another mode added until the optimal measure-of-fit is virtually unchanged by the addition of an extra mode. This approach allows the data to guide the number of modes required to adequately reproduce the recorded response. Experience with test problems (section 2.8) and examinations of the results obtained from real data suggest that the estimates of the parameters of the highest mode in the model, particularly the damping, may be in error. This happens because the bandwidth of the highest mode of the model response can be broadened in an attempt to reproduce the high frequency response contributed by the modes neglected in the model. For this reason, it is sometimes best to include one more mode in the model than is required to represent adequately the response so that realistic estimates can be achieved for the important modes in the response, while the estimates for the highest mode, which can be highly affected by noise, are ignored. However, this approach is not always possible because the estimation of the parameters of the "extra" mode may cause nonconvergence of the algorithm, particularly if the "error signal" being matched is small.

The frequency band for the identification is generally chosen broad enough to include all the significant response. However, a judicious choice of the high frequency limit can greatly increase the computational efficiency by ignoring the unimportant small amplitude high frequency data. For example, the measured earthquake response of buildings typically contains little response beyond 10 or 15Hz, while the fast Fourier transform produces data up to 25Hz (for $\Delta t = 0.02$ seconds). The low frequency limit is chosen to avoid the long period errors that often show up in displacement plots by excluding the lowest frequency points from the analysis.

When a one or two mode model is being considered for a structure obviously containing more responding modes, sometimes the frequency band is chosen so only the modal peaks under consideration are included in the identification. This is equivalent to band-pass filtering the time histories, an approach which has been used to study the fundamental mode behavior in previous studies (e.g., Iemura and Jennings [4]).

The initial period estimates can be determined by an inspection of the Fourier response spectrum and the transfer function, or from an examination of the acceleration, velocity or displacement histories. The participation factors may be estimated from measured mode shapes or design models where these are available, or may be chosen, for framed structures for example, as the values appropriate for a uniform shear beam. The initial estimates of the dampings may be either derived from the resonant amplifications, or given a "standard" value such as 5 percent. The most critical initial parameter estimates are the periods, particularly of the higher modes. The identification algorithm may converge to the modal periods closest to the initial estimates, so it is possible for some modes to be missed. This is chiefly a difficulty for the higher modes of time-varying structures, where there may be multiple peaks for each mode caused by the variation of the effective modal period. Moreover, it is often difficult to determine whether multiple peaks in the vicinity of a modal frequency are caused by closely spaced translational modes, the time variation of a single mode, or perhaps by a torsional response.

2.6 THE ROLE OF THE SECOND DERIVATIVE AND RELATED MATRICES: SENSITIVITY ANALYSIS

The matrix formed by the second derivatives of J with respect to the parameters $\underset{\sim}{\gamma}$ and two closely related matrices play important roles in the identification process.

The components of the second derivative matrix S are

$$S_{ij} = \frac{\partial^2 J}{\partial\gamma_i \partial\gamma_j} = 2 \sum_{\ell=\ell_{min}}^{\ell_{max}} \Bigg\{ R_e\left(\frac{\partial A_T(\ell\Delta\omega)}{\partial\gamma_i}\right) R_e\left(\frac{\partial A_T(\ell\Delta\omega)}{\partial\gamma_j}\right) + I_m\left(\frac{\partial A_T(\ell\Delta\omega)}{\partial\gamma_i}\right) I_m\left(\frac{\partial A_T(\ell\Delta\omega)}{\partial\gamma_j}\right) - R_e[A(\ell\Delta\omega) - A_T(\ell\Delta\omega)]\, R_e\left(\frac{\partial^2 A_T(\ell\Delta\omega)}{\partial\gamma_i\, \partial\gamma_j}\right) - I_m[A(\ell\Delta\omega) - A_T(\ell\Delta\omega)]\, I_m\left(\frac{\partial^2 A_T(\ell\Delta\omega)}{\partial\gamma_i\, \partial\gamma_j}\right) \Bigg\}$$

$$i=1,\ldots 5N_m, j=1,\ldots 5N_m \qquad (2\text{-}38)$$

The reduced second derivative matrix $\overline{S}_{ij}$ is defined as

$$\overline{S}_{ij} = 2 \sum_{\ell=\ell_{min}}^{\ell_{max}} \left\{ R_e\left(\frac{\partial A_T(\ell\Delta\omega)}{\partial \gamma_i}\right) R_e\left(\frac{\partial A_T(\ell\Delta\omega)}{\partial \gamma_j}\right) + I_m\left(\frac{\partial A_T(\ell\Delta\omega)}{\partial \gamma_i}\right) I_m\left(\frac{\partial A_T(\ell\Delta\omega)}{\partial \gamma_j}\right) \right\}$$

$$i=1,\ldots 5N_m, j=1,\ldots 5N_m \tag{2-39}$$

The partial Hessian matrix, PH, with respect to the non-linearly occurring parameters $\underset{\sim}{\alpha}$, which has the same form as $\overline{S}$, has been defined previously in equation (2-34).

The matrices S and $\overline{S}$ contain information on the relative accuracy of the estimates of the various parameters and the identifiability of the model. The matrix PH occurs directly in the identification algorithm. The first derivatives of the model response, $\frac{\partial A_T}{\partial \gamma_i}$, which occur repeatedly in the definitions of these matrices, are also important in the consideration of the identifiability of the model.

The roles of these matrices, the relationships between them, and the origin of the Gauss-Newton formula (2-35) can be illustrated by considering a second order Taylor series expansion of $J(\underset{\sim}{\gamma})$ about some parameter estimates $\underset{\sim}{\gamma}_0$.

$$J(\underset{\sim}{\gamma}) = J(\underset{\sim}{\gamma}_0) + \left\{\frac{\partial J(\underset{\sim}{\gamma}_0)}{\partial \underset{\sim}{\gamma}}\right\}^T \{\underset{\sim}{\gamma} - \underset{\sim}{\gamma}_0\} + \tfrac{1}{2}\{\underset{\sim}{\gamma} - \underset{\sim}{\gamma}_0\}^T \left[\frac{\partial^2 J(\underset{\sim}{\gamma}_0)}{\partial \gamma^2}\right] \{\underset{\sim}{\gamma} - \underset{\sim}{\gamma}_0\} + 0\|\underset{\sim}{\gamma} - \underset{\sim}{\gamma}_0\|^3 \tag{2-40}$$

Differentiating this expression with respect to $\underset{\sim}{\gamma}$ produces

$$\frac{\partial J(\underset{\sim}{\gamma})}{\partial \underset{\sim}{\gamma}} = \frac{\partial J(\underset{\sim}{\gamma}_0)}{\partial \underset{\sim}{\gamma}} + \left[\frac{\partial^2 J(\underset{\sim}{\gamma}_0)}{\partial \underset{\sim}{\gamma}^2}\right]\{\underset{\sim}{\gamma} - \underset{\sim}{\gamma}_0\} + 0\|\underset{\sim}{\gamma} - \underset{\sim}{\gamma}_0\|^2 \qquad (2\text{-}41)$$

By setting $\frac{\partial J(\underset{\sim}{\gamma})}{\partial \underset{\sim}{\gamma}}$ to zero, this expression gives a possible formula for the change in the parameter estimates, $\Delta\underset{\sim}{\gamma} = \underset{\sim}{\gamma} - \underset{\sim}{\gamma}_0$, required to achieve the minimum of J.

$$\left[\frac{\partial^2 J(\underset{\sim}{\gamma}_0)}{\partial \underset{\sim}{\gamma}^2}\right]\Delta\underset{\sim}{\gamma} = -\frac{\partial J(\underset{\sim}{\gamma}_0)}{\partial \underset{\sim}{\gamma}} \qquad (2\text{-}42)$$

This expression is very similar to the partial Hessian formula (2-35) utilized in the algorithm. The algorithm separates the calculation of the nonlinearly and linearly occurring parameters, $\underset{\sim}{\alpha}$ and $\underset{\sim}{\beta}$, to reduce the number of calculations required at each iteration. This produces a modification of (2-42) to involve changes in the $\underset{\sim}{\alpha}$ only:

$$\left[\frac{\partial^2 J(\underset{\sim}{\gamma}_0)}{\partial \underset{\sim}{\alpha}^2}\right]\Delta\underset{\sim}{\alpha} = -\frac{\partial J(\underset{\sim}{\gamma}_0)}{\partial \underset{\sim}{\alpha}} \qquad (2.43)$$

In the expression used in the algorithm, $[\partial^2 J/\partial \underset{\sim}{\alpha}^2]$ is replaced by [PH]. The reason for this replacement can be seen by considering the change in J according to the second order expansion produced by the $\Delta\underset{\sim}{\alpha}$ of (2-43). Substituting for $\frac{\partial J(\underset{\sim}{\gamma}_0)}{\partial \underset{\sim}{\alpha}}$:

$$J(\underset{\sim}{\alpha},\underset{\sim}{\beta}_0) - J(\underset{\sim}{\gamma}_0) = -\tfrac{1}{2}\{\underset{\sim}{\alpha} - \underset{\sim}{\alpha}_0\}^T\left[\frac{\partial^2 J(\underset{\sim}{\gamma}_0)}{\partial \underset{\sim}{\alpha}^2}\right]\{\underset{\sim}{\alpha} - \underset{\sim}{\alpha}_0\} + 0\|\underset{\sim}{\alpha} - \underset{\sim}{\alpha}_0\|^3$$

$$(2.44)$$

While it may not be necessary to achieve a reduction in J at each iteration to reach convergence eventually, such a condition at least prevents divergence. An obvious condition for the reduction of J is for the right-hand side of (2-44) to be negative. This will be guaranteed if the matrix

$$\left[\frac{\partial^2 J(\underset{\sim}{\gamma_0})}{\partial \underset{\sim}{\alpha}^2}\right]$$

is positive definite. At the minimum of J, this is a necessary condition, but for arbitrary values of the parameters, there is no guarantee that the matrix is positive definite. This is the reason for using the partial Hessian matrix in the algorithm. This matrix is positive definite for all parameter values except for one special case where it is positive semi-definite.

From the definition of PH:

$$\Delta\underset{\sim}{\alpha}^T [PH] \Delta\underset{\sim}{\alpha} = 2\left[\sum_{\ell} \left(\Delta\underset{\sim}{\alpha}^T \frac{\partial A_T(\ell\Delta\omega)}{\partial \underset{\sim}{\alpha}}\right)\left(\frac{\partial \bar{A}_T(\ell\Delta\omega)^T}{\partial \underset{\sim}{\alpha}} \Delta\underset{\sim}{\alpha}\right)\right] \geq 0 \tag{2-45}$$

The case where the partial Hessian matrix is singular is important in that it corresponds to a non-identifiable model. The non-identifiability is manifested during the identification process in that the expression (2-35), the Gauss-Newton formula for the change in the non-linearly occurring parameters, cannot be inverted.

An identifiable model requires that the reduced second derivative matrix with respect to all parameters, $\bar{S}$, and not just PH, is non-singular. The correspondence between a singular $\overline{S}$ and a non-identifiable model is easily illustrated.

If $\overline{S}$ is singular, one of its columns, say column i, must be a linear combination of the others. This requires

$$\sum_{\ell} R_e \left\{ \frac{\partial \overline{A}_T(\ell\Delta\omega)}{\partial \gamma_i} \frac{\partial A_T(\ell\Delta\omega)}{\partial \underset{\sim}{\gamma}} \right\} = \sum_{\substack{j=1 \\ j\neq i}}^{5xN \,x\, m} \left[K_j \sum_{\ell} R_e \left\{ \frac{\partial \overline{A}_T(\ell\Delta\omega)}{\partial \gamma_j} \frac{\partial A_T(\ell\Delta\omega)}{\partial \underset{\sim}{\gamma}} \right\} \right] \qquad (2\text{-}46)$$

The constants K_j are independent of ℓ (the frequency).

Equation (2-46) implies that the sensitivity coefficients are linearly dependent:

$$\frac{\partial A_T(\ell\Delta\omega)}{\partial \gamma_i} = \sum_{\substack{j \\ j\neq i}} K_j \frac{\partial A_T(\ell\Delta\omega)}{\partial \gamma_j} \quad \text{for all } \ell \qquad (2\text{-}47)$$

It is a well known result that linearly dependent sensitivity coefficients, $\frac{\partial A_T}{\partial \underset{\sim}{\gamma}}$, lead to a non-identifiable model by weighted least-squares fitting (Beck and Arnold, [2]) since there are insufficient independent equations obtained by setting to zero the derivatives of the measure-of-fit J with respect to the parameters.

In practice, the parameter estimates will be poor if the $\overline{S}$ matrix is ill-conditioned. There are three ways of determining the possibility of this occurrence without actually calculating the eigenvalues of $\overline{S}$. The first is by plotting the sensitivity coefficients $\frac{\partial A_T}{\partial \underset{\sim}{\gamma}}$ as a function of frequency. If they are approximately linearly dependent over the frequency range for which they are large, the $\overline{S}$ matrix will be almost singular. The other faster checks for a possible singularity of $\overline{S}$ are to calculate the ratios η_i^2 of the diagonal terms to the largest diagonal term $\overline{S}_{kk}$, and the ratios ρ_{ij} of the off-diagonal to diagonal terms:

$$\eta_i^2 = \overline{S}_{ii}/\overline{S}_{kk} \quad , \quad \overline{S}_{kk} = \text{maximum diagonal term of } \overline{S} \tag{2-48}$$

$$\rho_{ij} = -\overline{S}_{ij} \Big/ \left[\overline{S}_{ii}\overline{S}_{jj} \right]^{\frac{1}{2}} \tag{2-49}$$

If some η_i is zero or some ρ_{ij} is unity, $\overline{S}$ is singular. The case where η_i is zero means J is not dependent on the ith parameter, while when ρ_{ij} is unity the ith and jth sensitivity coefficients are linearly dependent, and there is coupling between the estimates of the parameters.

When the ratio of two of the eigenvalues of the $\overline{S}$ matrix is large, there is a direction along which J is insensitive to changes in the parameters. As a rough guide, this occurs when the eigenvalue ratio exceeds about twenty-five, corresponding approximately to an η_i less than 0.2 (J insensitive to the corresponding parameter) or a ρ_{ij} greater than 0.8 (coupling between parameters).

The Gauss-Newton formula can be interpreted as the equation of "sensitivity ellipses", which are surfaces of constant J. When the coupling between parameters is small, the sensitivity ellipses about the optimal estimates $\underset{\sim}{\gamma}_{opt}$ can be approximated by

$$J \doteq J_{opt} + \tfrac{1}{2} \sum_i \overline{S}_{ii} \Delta\gamma_i^2 \tag{2-50}$$

It is convenient to consider a normalized form of this equation

$$\frac{J-J_{opt}}{J_{opt}} \doteq \frac{1}{2J_{opt}} \sum_i \left(\gamma_{i_{opt}}^2 \; \overline{S}_{ii}\right) \left(\frac{\Delta\gamma_i}{\gamma_{i_{opt}}}\right)^2 \qquad (2\text{-}51)$$

This equation can be used to determine relative error bounds on the different parameters. If each parameter is varied individually, the bound on the fractional variation in γ_i for a fractional variation of r in J_{opt} is

$$\frac{\Delta\gamma}{\gamma_{i_{opt}}} = \pm \sqrt{2r} \Bigg/ \left[\frac{\gamma_{i_{opt}}^2}{J_{opt}} \overline{S}_{ii}\right]^{\frac{1}{2}} \qquad (2\text{-}52)$$

In sensitivity analyses later, the normalized sensitivity matrix $\frac{\gamma_{i_{opt}} \gamma_{j_{opt}}}{J_{opt}} \overline{S}_{ij}$ is determined along with the coupling coefficient ρ_{ij}. The fractional changes in the parameters for r = 0.1 are determined from (2-52), which ignores the effect of coupling between the estimates but is sufficiently accurate for ρ_{ij} less than 0.8.

The Taylor series expansion of J about the optimal estimates indicates that the full second derivative matrix S rather than the reduced matrix $\overline{S}$ should be used for sensitivity analysis. However, $\overline{S}$ is easier to calculate, since it does not involve the second derivatives of the model response with respect to the parameters. The two matrices are sufficiently closely related, and indeed are identical at the optimal estimates for the unlikely case of zero optimal error, that it is felt that the use of $\overline{S}$ should not change the conclusions about the sensitivities which would be drawn from S. Checking for the singularity of $\overline{S}$ is all that is required to determine whether the parameters are identifiable.

2.7 VARIATIONS OF THE IDENTIFICATION METHOD

During the analysis of the earthquake response data, several variations of the standard identification method described above have at times been found useful.

One modification was an extension of the model to allow response in a given direction to be caused by both horizontal components of the ground motion. This extension has been used to investigate the possibility of horizontal coupling of the modes.

The model was generalized by allowing contributions to the forcing function in the equation of motion (2-1) from both horizontal components of the ground motion. The response vector $\underset{\sim}{x}$ now becomes a generalized displacement vector, containing both horizontal components of the translational response and possibly a rotational component about a vertical axis for each response location, giving a total of N_f degrees of freedom. Numerical indices 1 and 2 and the letter i refer to the two horizontal directions. The location is denoted by p and the mode by r as before. The vectors $\underset{\sim}{\Gamma}_1$ and $\underset{\sim}{\Gamma}_2$ specify the contributions from the two horizontal ground components. Equation (2-1) is modified to:

$$[M]\ddot{\underset{\sim}{x}}(t) + [C]\dot{\underset{\sim}{x}}(t) + [K]\underset{\sim}{x}(t) = -[M]\left(\underset{\sim}{\Gamma}_1\ddot{z}_1(t) + \underset{\sim}{\Gamma}_2\ddot{z}_2(t)\right) \qquad (2\text{-}53)$$

Following the modal decomposition as before:

$$x_p^{(i)}(t) = \sum_{r=1}^{N_f} \phi_{pr}^{(i)} \xi_r(t) = \sum_{r=1}^{N_f} x_{pr}^{(i)}(t) \qquad (2\text{-}54)$$

The equation of motion for $\xi_r(t)$ becomes:

$$\ddot{\xi}_r(t) + a_r\dot{\xi}_r(t) + b_r\xi_r(t) = -\frac{\underset{\sim}{\phi}_r^T[M](\underset{\sim}{\Gamma}_1\ddot{z}_1 + \underset{\sim}{\Gamma}_2\ddot{z}_2)}{\underset{\sim}{\phi}_r^T[M]\,\underset{\sim}{\phi}_r} \qquad (2\text{-}55a)$$

$$= -(\alpha_{r_1}\ddot{z}_1 + \alpha_{r_2}\ddot{z}_2) \qquad (2\text{-}55b)$$

The equations for the two horizontal components of the modal displacement are

$$\ddot{x}_{pr}^{(i)}(t) + a_r\dot{x}_{pr}^{(i)}(t) + b_r x_{pr}^{(i)}(t) = -\phi_{pr}^{(i)}(\alpha_{r_1}\ddot{z}_1 + \alpha_{r_2}\ddot{z}_2) \qquad (2\text{-}56a)$$

$$= -c_{i1pr}\ddot{z}_1 - c_{i2pr}\ddot{z}_2 \qquad (2\text{-}56b)$$

The transformation to the frequency domain follows through exactly as before. The equations take the same form, with effective participation factors for both directions of the ground acceleration for each mode at each position. There are only three independent participation factors for the two directions of input and response, since

$$\frac{c_{21pr}}{c_{11pr}} = \frac{c_{22pr}}{c_{12pr}} \qquad (2\text{-}57)$$

In practice, this constraint wasn't applied. The four participation factors were considered independent, with two estimated from each component of the response. The agreement of the two ratios of equation (2-57) was used together with the independent estimates of the modal dampings and frequencies from the two records as a check of the consistency of the models derived from the two records.

Investigations of possible horizontal coupling with this extended model were performed for only a few of the response records.

Several variations of the standard identification technique have been used to overcome problems of interaction between the estimates of the participation factors and dampings. This interaction occurred in the analysis of the data from many of the buildings studied. The reason for the interaction is that the dominant frequency components of the response occur near the modal frequencies where the amplification of the excitation depends on the ratio of the participation factor to the dampings. Consequently, the response is sensitive to this ratio, but much less sensitive to the individual values of the two parameters. This interaction is unfortunate, since the value of the damping is one of the results of most interest.

The strategy used in attempts to overcome this problem when there are response records from only one location in a structure is to constrain the participation factors to values derived from other data, such as vibration test measurements of the mode shapes or synthesized models. The minimization of J is performed only with respect to the other modal parameters, with the participation factors treated as known constants. The rationale behind this approach is that vibration tests and mode shape calculations from synthesized models have shown that the participation factors are much less sensitive than the periods to changes in the stiffness distribution. This approach, of course, prevents the extraction of information about changes in the mode shapes which may be useful in determining the location of any yielding or softening behavior in the structure associated with the lengthening of the modal periods.

Another approach can be used to circumvent the interaction problem when response records are available from two locations in a structure. The two records, generally from near midheight and at the roof, can be used simultaneously to estimate the values of the parameters. The reason for trying this approach is as follows. The coupling between the estimates of the damping ζ and participation factor c means that there is a line in the c-ζ plane along which the measure-of-fit J varies little. However, the direction of this line is likely to be different for the two records. Thus the identification of the two participation factors and one damping for each mode simultaneously from the two records may overcome the interaction effect and improve the estimates.

The identification criterion becomes:

$$\text{Minimize } \tilde{J}_2(\underset{\sim}{a},\underset{\sim}{b},\underset{\sim}{c}_1,\underset{\sim}{c}_2) = \sum_{\ell=\ell_{min}}^{\ell_{max}} \Big\{ g_1 |A_1(\ell\Delta\omega) - A_{T_1}(\ell\Delta\omega;\underset{\sim}{a},\underset{\sim}{b},\underset{\sim}{c}_1)|^2 + g_2 |A_2(\ell\Delta\omega) - A_{T_2}(\ell\Delta\omega;\underset{\sim}{a},\underset{\sim}{b},\underset{\sim}{c}_2)|^2 \Big\} \quad (2\text{-}58)$$

The indices 1 and 2 denote the two response locations, and g_1 and g_2 are weighting factors for the two records.

The normalized error J_2 is defined as

$$J_2 = \tilde{J}_2 \Bigg/ \left[\sum_{\ell=\ell_{min}}^{\ell_{max}} \{ g_1 |A_1(\ell\Delta\omega)|^2 + g_2 |A_2(\ell\Delta\omega)|^2 \} \right] \qquad (2\text{-}59)$$

When response records are available from two locations, this simultaneous identification approach has the advantage over the constrained participation factor approach that the variation with time of both the damping and participation factors can be traced.

2.8 AN APPLICATION OF THE IDENTIFICATION ALGORITHM WITH GENERATED TEST DATA

The performance of the standard identification method is now illustrated by reporting the results of identifications performed for a test case with generated data.

The parameters of a uniform ten-story shear structure were estimated from simulated data generated for its response to the first ten seconds of the 1940 El Centro north-south component, as shown schematically in Figure 2.1. The modal properties of the structure are listed in Table 2.1, together with the results of fits using models with four and six modes.

A six mode match of the Fourier transform of the top mass acceleration response was performed over the frequency band from 0.3 Hz to 13.6 Hz, covering the range of significant dynamic response. Initial values of the parameters for the analysis were estimated from an examination of the modulus of the calculated transfer function. The initial estimates of the natural periods were close to the true values, but some of the damping and participation factor estimates were taken 100 percent in error. During the course of the identification, the normalized error was reduced from 0.108 for the initial estimates to a final value of 0.008, an excellent match. Virtually no error was discernible in plots of the acceleration time histories (Figure 2.2). The estimates of the parameters for all but the sixth mode were very close to the true values. One, two and four mode identifications were also performed. The results of the four-mode match, with the initial estimates deliberately chosen in error, are also listed in Table 2.1.

This example illustrated the dominance of the lower modes in the earthquake response of this type of structure. A plot of the top mass displacement of the optimal two mode model was practically indistinguishable from that of the ten mass system. The velocity is more sensitive to higher modes than the displacement, but the optimal four mode model gave an essentially perfect velocity match.

Attempts to perform identifications with more than six modes were unsuccessful. Because of the limited frequency content of the input, only six modal peaks were discernible in the transfer function of the simulated response (Figure 2.3a), and models with more than six modes invariably led to unrealistic and nonconvergent estimates of the parameters of the higher modes.

The tests with simulated data showed that for nearly ideal situations the identification algorithm produced accurate estimates of the parameters and excellent fits in the time and frequency domains. The remainder of this dissertation is concerned with the analysis of real earthquake data.

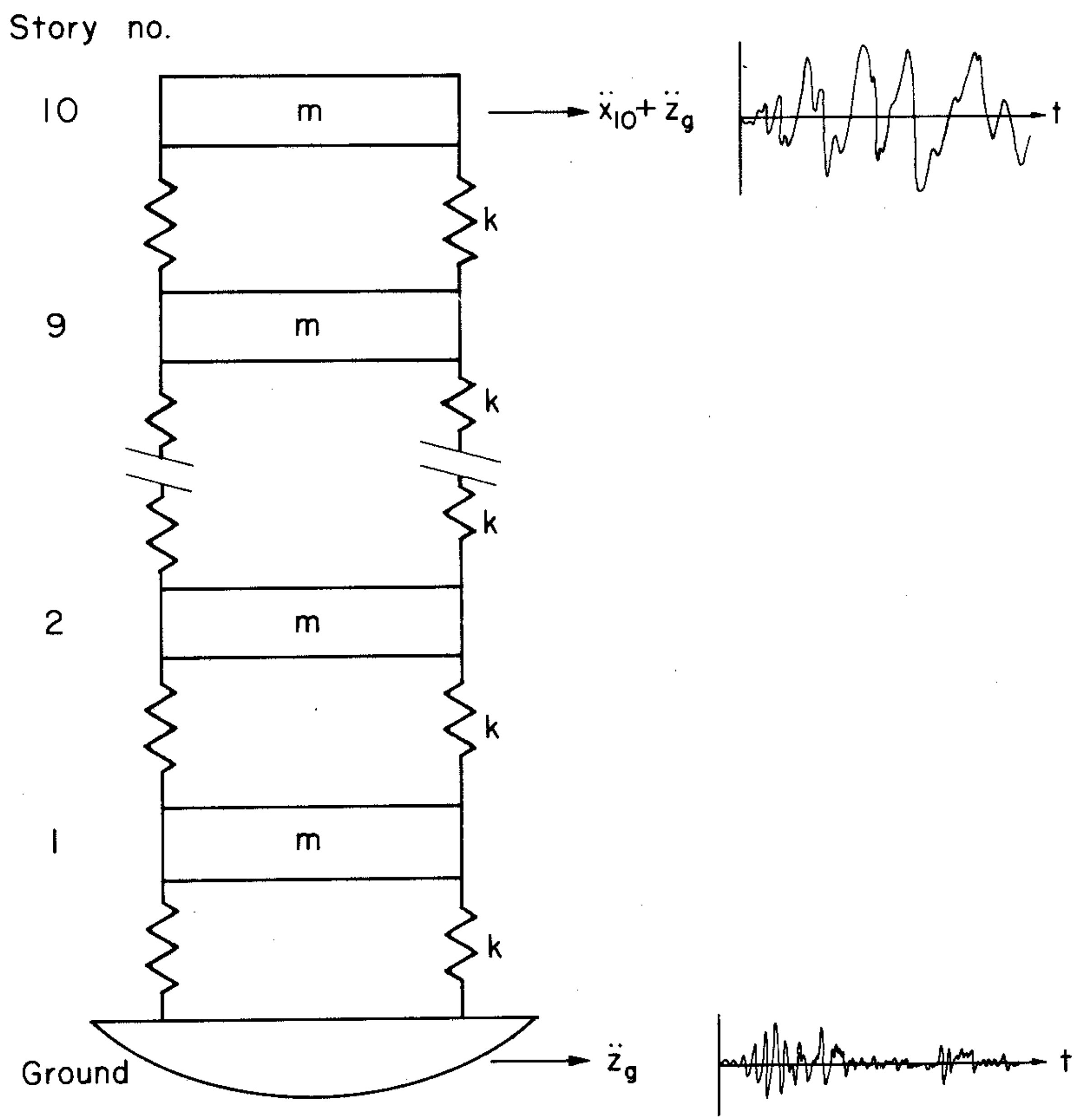


Figure 2.1 The uniform ten story shear structure used for testing the identification technique with the El Centro 1940 north-south component.

TABLE 2.1

PARAMETERS OF THE TEN STORY UNIFORM SHEAR STRUCTURE USED FOR A SIMULATION TEST OF THE IDENTIFICATION TECHNIQUE

True Parameter Values				
Mode	Period (sec)	Frequency (Hz)	Damping	Effective Participation Factor 10th Story
1	1.0000	1.0000	0.05	1.2673
2	0.3358	2.9780	0.05	-0.4068
3	0.2045	4.8900	0.05	0.2259
4	0.1495	6.6890	0.05	-0.1429
5	0.1199	8.3403	0.05	0.0934
6	0.1019	9.8135	0.05	-0.0601
7	0.0904	11.0619	0.05	0.0366
8	0.0829	12.0627	0.05	-0.0199
9	0.0782	12.7877	0.05	0.0087
10	0.0756	13.2275	0.05	-0.0021

Six Mode Match (Frequency band = 0.29 - 13.6 Hz)						
Initial Estimates (from transfer function)				Final Estimates		
Mode	Period (sec)	Damping	P.F.	Period	Damping	P.F.
1	1.00	0.06	1.25	1.0007	0.0501	1.2749
2	0.31	0.06	-0.45	0.3364	0.0565	-0.3865
3	0.20	0.07	0.35	0.2047	0.0521	0.2229
4	0.14	0.09	-0.25	0.1496	0.0535	-0.1344
5	0.12	0.10	0.15	0.1201	0.0508	0.0860
6	0.10	0.07	-0.07	0.1026	0.0317	-0.0308
Normalized Error = 0.108				Normalized Error = 0.008		

Four Mode Match (Frequency band = 0.29 - 6.74 Hz)					
Initial Estimates			Final Estimates		
Period (sec)	Damping	P.F.	Period	Damping	P.F.
1.120	0.038	1.25	1.0007	0.0502	1.2759
0.373	0.038	-0.35	0.3364	0.0564	-0.3854
0.224	0.038	0.18	0.2048	0.0517	0.2218
0.160	0.038	-0.14	0.1507	0.0394	-0.0959

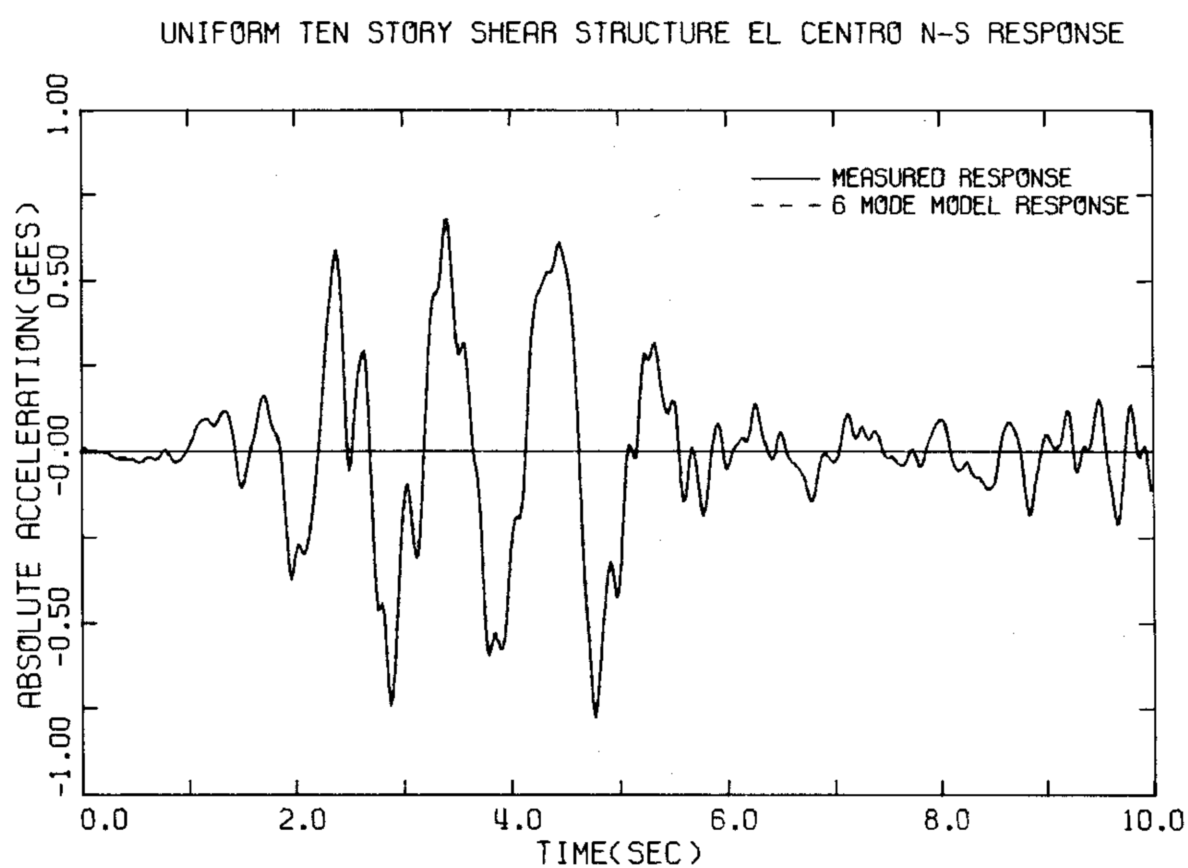


Figure 2.2 The measured and optimal six-mode model acceleration response of the uniform ten-story shear structure. The curves are virtually indistinguishable.

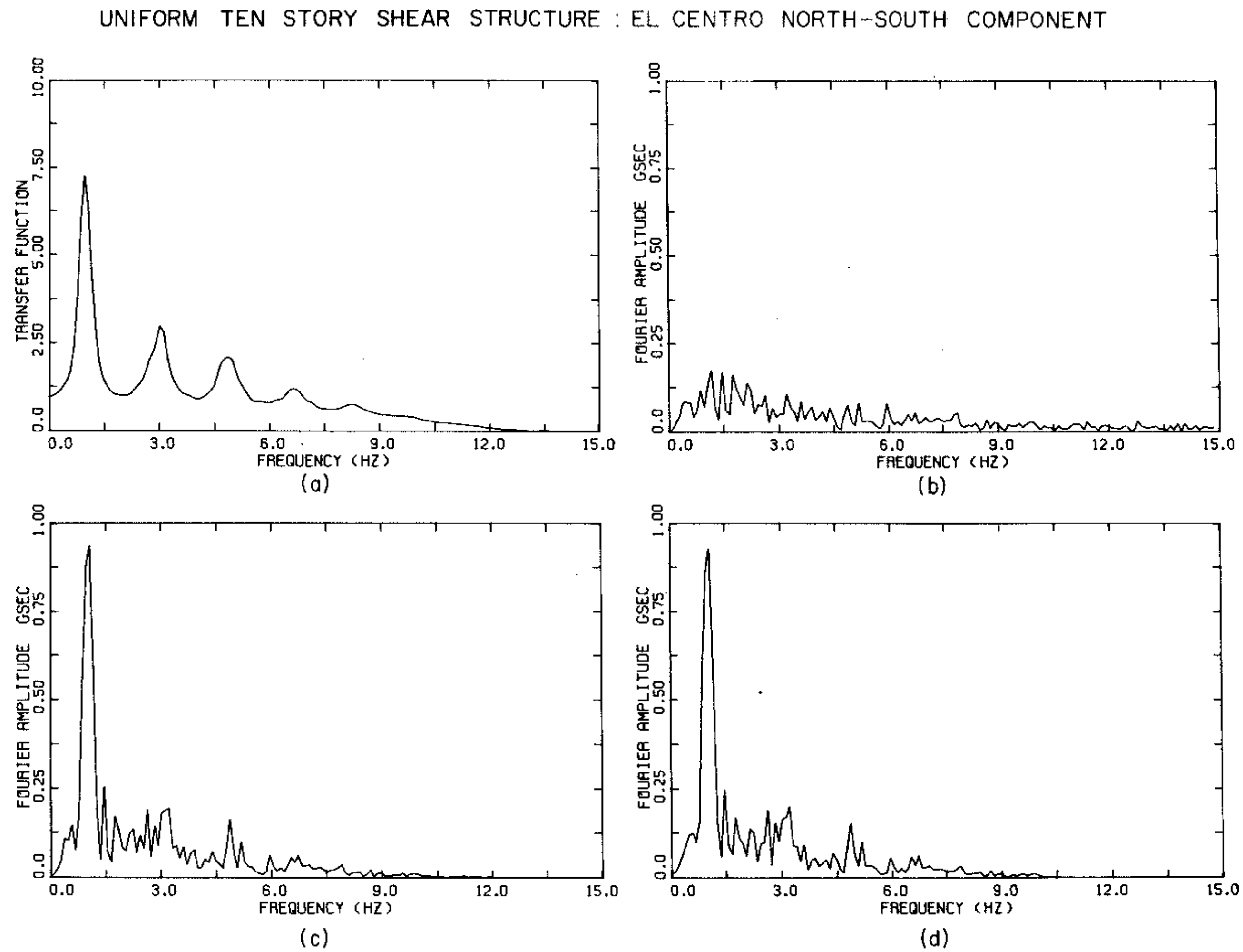


Figure 2.3 Frequency domain plots for the uniform ten story shear structure. The modulus of (a) the transfer function; (b) the unsmoothed ground acceleration transform; (c) the response acceleration transform; (d) the optimal six-mode response acceleration transform for comparison with (c).

REFERENCES

1. Beck, J.L., "Determining Models of Structures from Earthquake Records," Earthquake Engineering Research Laboratory, EERL 78-01, California Institute of Technology, Pasadena, California, June 1978.

2. Beck, J.V. and K.J. Arnold, 'Parameter Estimation in Engineering and Science', Wiley, New York 1977.

3. Ibáñez, P., "Identification of Dynamic Structural Models from Experimental Data", Report No. UCLA-ENG-7225, UCLA, Los Angeles, California, 1972.

4. Iemura, H. and P.C. Jennings, "Hysteretic Response of a Nine-Story Reinforced Concrete Building," Int. Journal of Earthquake Eng. and Struct. Dyn., 3, 183-201, 1974.

5. Iwan, W.D. and N.C. Gates, "Estimating Earthquake Response of Simple Hysteretic Structures", J. Eng. Mech. Div., ASCE 105, EM3, 391-405, June 1979.

6. Udwadia, F.E. and D.K. Sharma, "Some Uniqueness Results Related to Building Structural Identification", SIAM J. Appl. Math., 34, 104-118.

7. Udwadia, F.E., D.K. Sharma and P.C. Shah, "Uniqueness of Damping and Stiffness Distributions in the Identification of Soil and Structural Systems", J. Appl. Mech., ASME 45, 1, 181-7, March 1978.

III. TWO WELL-STUDIED BUILDINGS: MILLIKAN LIBRARY AND JPL BUILDING 180

3.1 INTRODUCTION

As the first applications of the identification technique to real earthquake data, two structures which have been studied many times before, Millikan Library and JPL Building 180, are considered. Both buildings have been subjected to many vibration tests, which have shown changes in their dynamic properties, and there have been several analytical studies of their earthquake response. There are small amplitude records from the Lytle Creek and Borrego Mountain earthquakes as well as the large amplitude responses from the San Fernando earthquake available for identification. Thus it is possible to identify the dynamic characteristics for different levels of earthquake response, and compare the results with those of earlier studies and vibration tests. The San Fernando responses furnish challenging examples for identification with linear models because some of the data show indications of significant nonlinear behavior.

3.2 MILLIKAN LIBRARY BUILDING

3.2.1 INTRODUCTION

The nine-story Robert A. Millikan Library building (Fig. 3.1) stands on the campus of the California Institute of Technology in Pasadena. The reinforced concrete structure reaches a height of 144 feet above ground level, and 158 feet above the basement level. The horizontal dimensions are 75 feet in the east-west direction by 69 feet in the north-south direction. Shear walls at the ends of the building

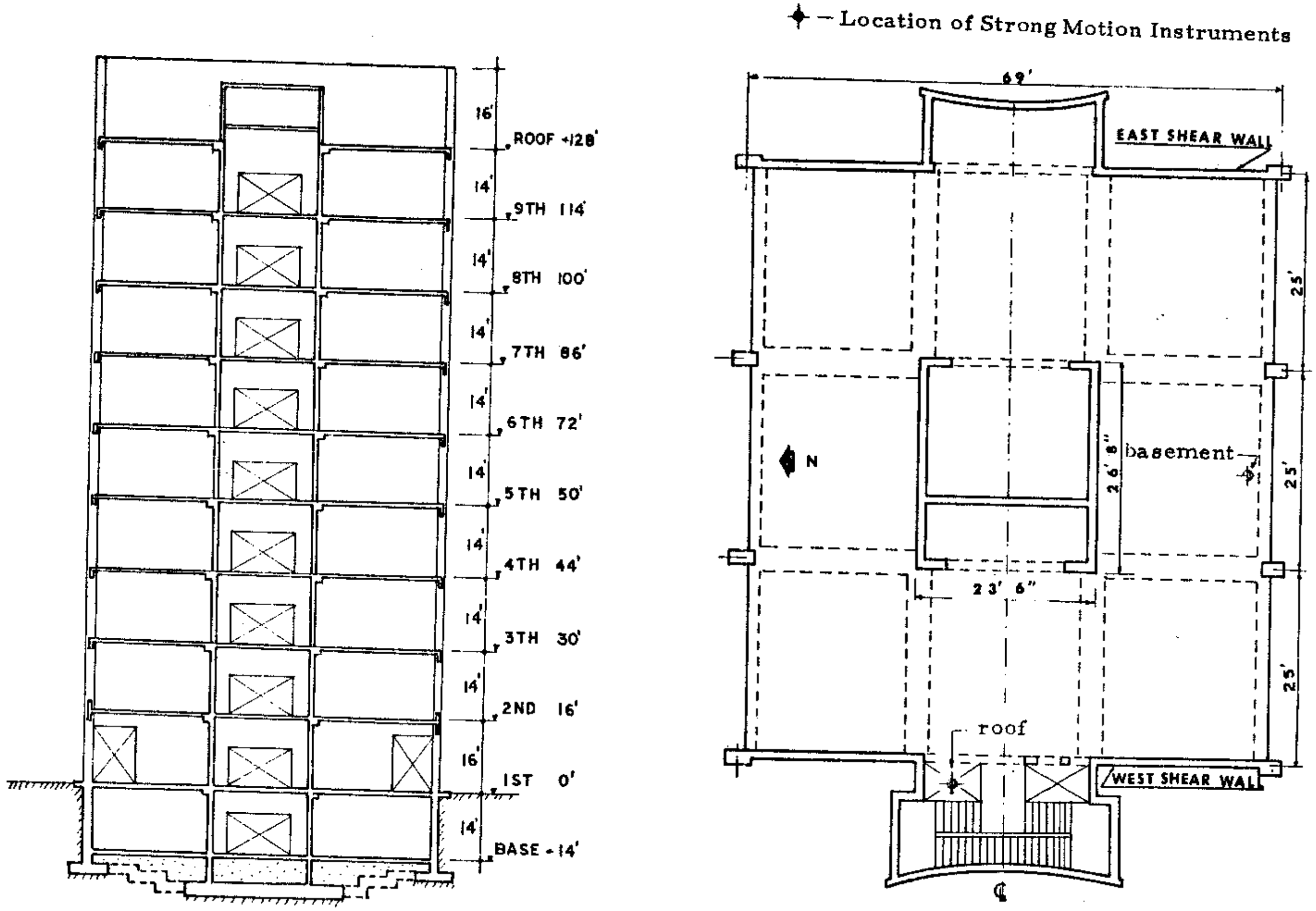


Figure 3.1 Schematic of the Millikan Library structural system. (a) Transverse section (b) Typical floor plan (from Foutch et al.[4]).

provide the primary lateral resistance in the north-south direction, while a central core wall contributes most of the east-west stiffness. Vibration tests during construction showed that precast window-wall panels which are bolted to the north and south walls add appreciably to the stiffness, at least for low levels of vibration. Kuroiwa [10] provides more detailed information about the building.

The results of vibration tests of the structure are summarized in Table 3.1. The dynamic properties measured after the San Fernando earthquake of February 9, 1971 varied significantly from those found prior to the earthquake. The most noticeable change was the lengthening of the modal periods.

Earlier studies [5,7,9,13,14] have attributed some features of the San Fernando response to nonlinear, time-varying behavior. It has been suggested that this behavior is a consequence of a marked change of the foundation compliance and a softening of the structure, with stiff but brittle non-structural elements contributing much reduced lateral resistance following the first large amplitude vibrations during the earthquake.

The responses of the structure to the Lytle Creek and San Fernando earthquakes were studied by moving-window Fourier analysis techniques by Udwadia and Trifunac [14]. They found a reduction in the apparent natural frequency in the east-west direction of 50 percent during the San Fernando response. The natural frequency in ambient tests showed a change of about a third of this amount, with a progressive partial recovery of a few percent towards the pre-earthquake frequency in the two years following the earthquake.

TABLE 3.1

NATURAL PERIODS OF MILLIKAN LIBRARY FROM VIBRATION TESTS

Kuroiwa's Tests During Construction (Reference 10)				
Mode	Period (sec)	Excitation	Typical Response Acceleration	Construction Phase
NS1 EW1	0.46 0.71	Man excited "	$\sim 10^{-5}$g	Before precast wall panel placed
NS1 T1 EW1 EW2	0.491 0.338 0.671 0.155	Shaker operated at lowest force level " "	$\sim 5 \times 10^{-4}$g	After north facade placed
NS1 EW1	0.505-0.530 0.662-0.682	Shaker at range of force levels	$5\text{-}20 \times 10^{-3}$g $3\text{-}17 \times 10^{-3}$g	Both facades placed
NS1 EW1	0.52 0.66	Man excited "	$\sim 10^{-5}$g	Finishing work completed

Ambient and Man-Excited Tests					
Mode	March 1967	April 1968	July 1969	After Feb. 1971 Earthquake	May 1976
NS1	0.524	0.526	0.530	0.556	0.54
NS2	-	0.109	0.110	0.111	-
EW1	0.67	0.69	0.69	0.80	0.79
EW2	0.164	0.164	0.170	0.190	0.190
T1	0.35	0.345	-	0.378	0.377
T2	-	0.104	-	0.104	0.107
Typical Acceleration Response Amplitude for Ambient Tests $\sim 10^{-6}$g					

The nonlinear behavior of Millikan Library during its response to the San Fernando earthquake has been demonstrated by Iemura and Jennings [7]. In a study of the response of the fundamental east-west mode, they found much better matches of the recorded response with time-varying linear and bilinear hysteretic models than with time-invariant models.

Udwadia and Marmarelis [13] found that the linear and nonlinear contributions to the San Fernando response were of comparable amplitude during the strongest portion of the motion. They used the Wiener technique of nonparametric identification in their study.

Recently Jerath and Udwadia [9] identified a time-varying linear model for the fundamental mode from the east-west component of the San Fernando response. They allowed the stiffness and damping coefficients to be polynomial functions of time and thus were able to trace the time variation of the effective linear parameters. The frequency reduced by 35 percent from the initial value during the course of the motion.

3.2.2 IDENTIFICATION STUDIES

Strong-motion records were obtained in the basement and on the roof from the Lytle Creek earthquake of September 12, 1970 and the San Fernando earthquake of February 9, 1971. The magnitude 5.4 Lytle Creek earthquake, centered 40 miles from Millikan Library, produced a maximum ground acceleration of approximately 0.02g and a roof acceleration of 0.05g in the building, fairly low vibration levels for measured earthquake motion. The larger magnitude 6.4 San Fernando earthquake, centered 19 miles from the library, produced a maximum ground acceleration of about 0.2g, and a roof response of 0.35g. The structure was also shaken in the more distant magnitude 6.8 Borrego Mountain earthquake of April 8, 1968, but only the basement instrument triggered, recording a maximum ground acceleration of 0.01g. The level of excitation in the Lytle Creek and San Fernando earthquakes differed by a factor of ten, and the Lytle Creek response was another factor of three larger than the greatest forced vibration response. The amplitudes during the ambient tests were diminished by another factor of ten thousand.

LYTLE CREEK RESPONSE

One- and two-mode identifications have been performed for both components of the Lytle Creek earthquake. The results of the identifications are presented in Table 3.2 and Figures 3.2 and 3.3.

The north-south identifications produced a fundamental period of 0.516 seconds, in the range of periods measured in the pre-earthquake vibration tests, a first mode damping of three percent, and an effective participation factor of 1.46 at the roof. The normalized measure-of-fit was 0.133 for the one mode model and 0.103 for two modes, indicating that the linear, time-invariant models produced a reasonable approximation to the measured response.

In the east-west case, the fundamental period was identified as 0.710 seconds, slightly longer than the values obtained in vibration tests after the completion of the structure, but the same as that found by Kuroiwa before the pre-cast wall panels were placed, indicating that perhaps the increased stiffness contributed by the wall panels in the vibration tests was not effective even at the amplitudes experienced in

TABLE 3.2

PARAMETER ESTIMATES FOR MILLIKAN LIBRARY FROM LYTLE CREEK RESPONSE

Frequency Band of Identifications = 0.34 - 10.9 Hz

North-South						
	Initial Estimates			Final Estimates		
Model	Period (sec)	Damping	Participation Factor	Period (sec)	Damping	Participation Factor
1 Mode	0.53	0.03	1.44	0.516	0.030	1.486
	Normalized Error = 0.391			Normalized Error = 0.133		
2 Modes	0.53 0.11	0.03 0.03	1.44 -0.50	0.516 0.116	0.029 0.010	1.459 -0.212
	Normalized Error = 0.392			Normalized Error = 0.103		

East-West						
	Initial Estimates			Final Estimates		
Model	Period (sec)	Damping	Participation Factor	Period (sec)	Damping	Participation Factor
1 Mode	0.67	0.03	1.44	0.710	0.023	1.345
	Normalized Error = 0.899			Normalized Error = 0.334		
2 Modes	0.67 0.16	0.03 0.03	1.44 -0.50	0.710 0.175	0.022 0.036	1.291 -0.458
	Normalized Error = 0.991			Normalized Error = 0.140		

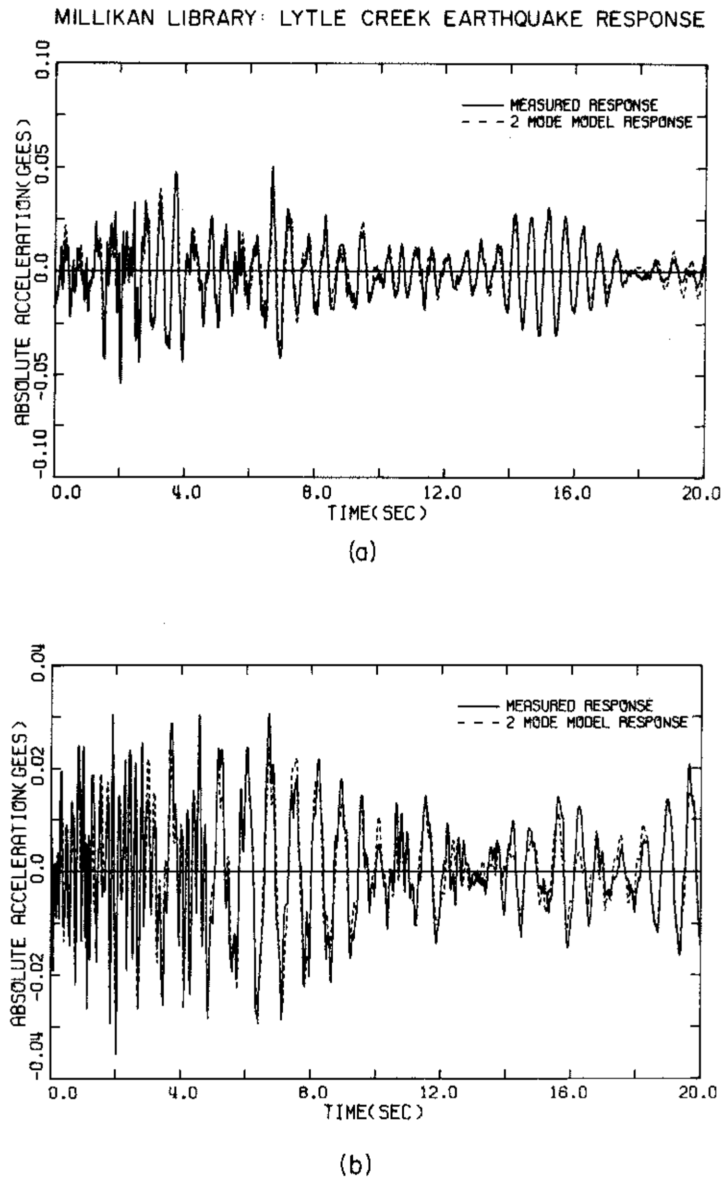


Figure 3.2 The measured and optimal two mode acceleration response of Millikan Library to the Lytle Creek earthquake. (a) North-south response. (b) East-west response.

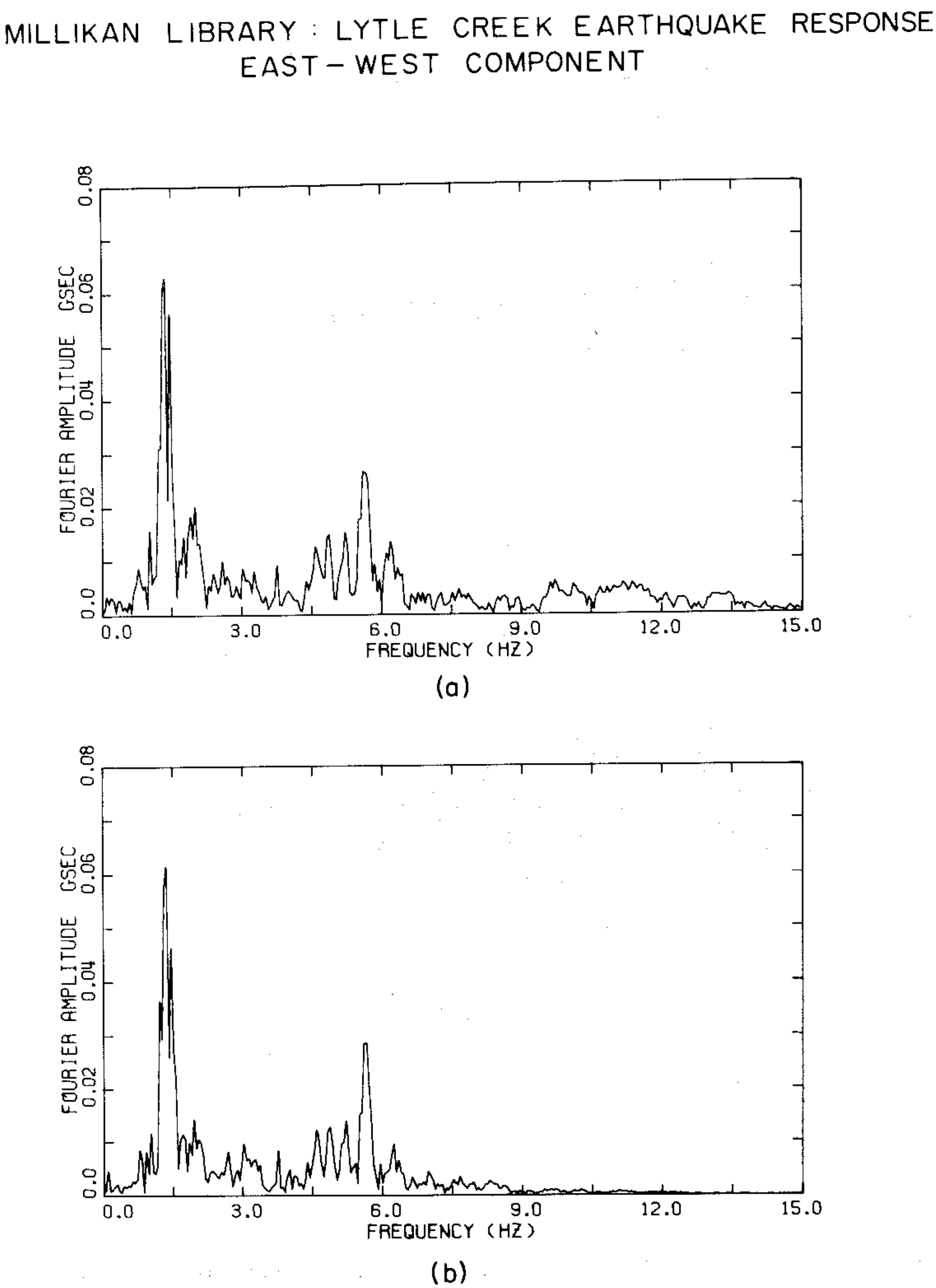


Figure 3.3 The Fourier amplitude spectrum of the east-west acceleration response of Millikan Library to the Lytle Creek earthquake. (a) The measured spectrum. (b) The optimal two-mode model spectrum.

this low level earthquake response. The second period was identified as 0.175 seconds, also slightly longer than recorded in the vibration tests. The measure-of-fit for the two models was 0.33 and 0.14, showing that the second mode made an important contribution to the response. The main sources of error were peaks at 1.9-2.0 Hz in the acceleration response transform, which neither model reproduced well, and further high frequency response beyond the second mode frequency (Fig. 3.3). The 2.0 Hz response is at the fundamental north-south frequency, suggesting an east-west component in the response of this mode. However, the inclusion in the model of a mode at this frequency capable of responding to input in both directions did not improve the fit, suggesting that horizontal coupling was not the cause of the discrepancy. Because the first torsional frequency is near 2.9 Hz, it is also unlikely that torsion is responsible for this peak.

The identifications performed for the Lytle Creek records showed that for this low level of earthquake excitation Millikan Library behaved as it did in vibration tests. The identified periods were close to those measured in tests, and the two-mode linear time-invariant models produced a good approximation to the measured response (Fig. 3.2).

SAN FERNANDO RESPONSE

The San Fernando records represent the response to a much stronger earthquake excitation, with a maximum ground acceleration near 0.2g and a roof acceleration peaking at 0.35g. The first 40.96 seconds of the north-south and east-west records, which include all the significant response, have been used as the data to identify the parameters of one-and two-mode models. The time variation of the equivalent linear parameters was investigated by using segments of the data. Near the beginning of the response where the variations were most pronounced, intervals of 2.56 seconds were used, but more typically the segment lengths were either 10.24 or 20.48 seconds. Some sensitivity analyses were also performed to determine the relative accuracy of the estimates of the various parameters and to investigate the degree of coupling between different parameters.

The results of the north-south identifications are shown in Figure 3.4, in which the parameter values are plotted as a function of the mid-interval time of the segment from which they were identified. The first mode period lengthens from the vibration test value of 0.52 seconds at the beginning of the record to 0.62 seconds during the strongest portion of the response. Most of the period lengthening occurs during the first five seconds of the response at the onset of the large amplitude motion, with the period remaining fairly constant during the remainder of the response. The first mode damping reaches a maximum of just over eight percent of critical during the largest amplitude motion, before dropping to five percent for the second twenty seconds of the response. The participation factor is reasonably constant, at approximately 1.5. The second mode parameters vary rather erratically, and trends are not apparent.

A two-mode fit for the entire 40.96 second record produced the parameter values listed in Table 3.3. The error bounds indicate the changes in the parameter estimates which would produce a ten percent change in J, as calculated from the diagonal elements of the sensitivity matrix (equation 2.52). The measure-of-fit of 0.13 indicates that despite the variation of the parameters in the early part of the record, a linear time-invariant model provides a reasonably good approximation to the actual system. The effective first mode period for the time-invariant model is the maximum value of 0.62 seconds which occurred during the largest amplitude portion of the motion. The equivalent damping of 0.064 is less than the value of 0.082 effective during the strongest motion, but still more than double the value of 0.030 exhibited in the Lytle Creek response. The participation factor of 1.49 is close to that from the Lytle Creek response. The quality of the fit obtained can be seen by comparing the measured acceleration and velocity histories with those calculated for the time-invariant model (Fig. 3.5), and by examining the acceleration transform match (Fig. 3.6). The displacement records obtained from the double integration of the recorded accelerations contained long period components even after the application of the standard correction techniques [6,12]. These long period components, thought to be a spurious result of the measurement and processing techniques, complicated comparison of the displacements.

The error bounds indicate that the periods are identified accurately, but the damping and participation factor estimates are poorer. The first mode damping and participation factor vary by about 10 and 20 percent respectively for a 10 percent change in the measure-of-fit J. The bounds on the second mode damping and participation factor, coupled with the nonlinear response of the structure, indicate that the estimates of these parameters are unreliable. The interaction coefficients were small except between $a_1 = 2\zeta_1\omega_1$ and c_1, and $a_2 = 2\zeta_2\omega_2$ and c_2,

TABLE 3.3

PARAMETER ESTIMATES FOR TIME-INVARIANT MODELS OF MILLIKAN LIBRARY FROM THE SAN FERNANDO RESPONSE

North-South

Identification Frequency Band = 0.37 - 14.6 Hz

Mode	Period (Sec)	Damping	Participation Factor
1 2	0.622 ± 0.008* 0.128 ± 0.008	0.064 ± 0.013 0.047 ± 0.006	1.487 ± 0.16 -0.447 ± 0.30
Normalized Error E = 0.129			

East-West

Identification Frequency Band = 0.37 - 7.3 Hz

Mode	Period (Sec)	Damping	Participation Factor
1 2	0.975 ± 0.02 0.201 ± 0.015	0.070 ± 0.02 0.059 ± 0.08	1.480 ± 0.28 -0.463 ± 0.5
Normalized Error E = 0.271			

*Error bounds correspond to a 10 percent change in E.

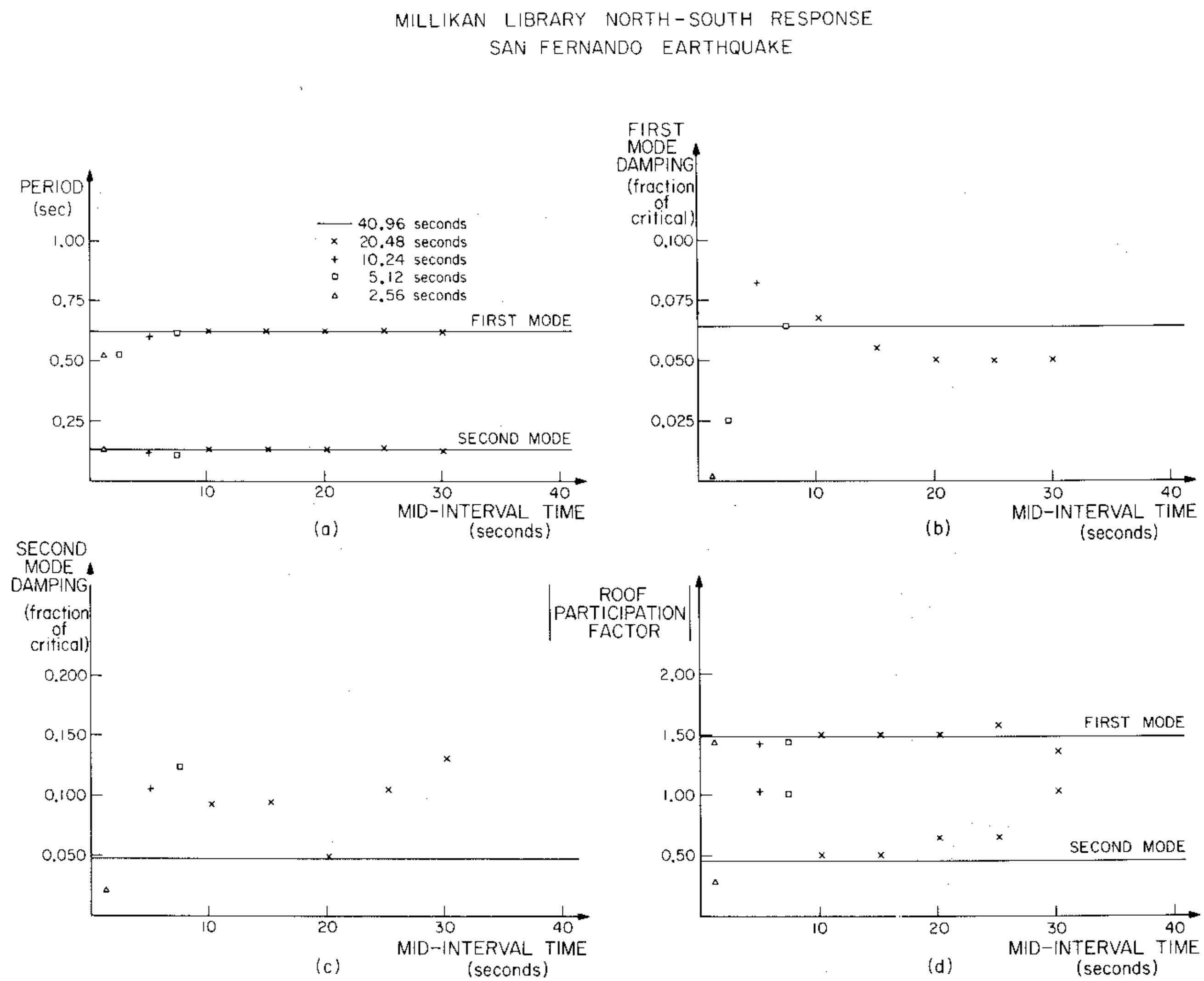


Figure 3.4 Time variation of the parameters of linear models of Millikan Library identified from segments of the San Fernando north-south response.

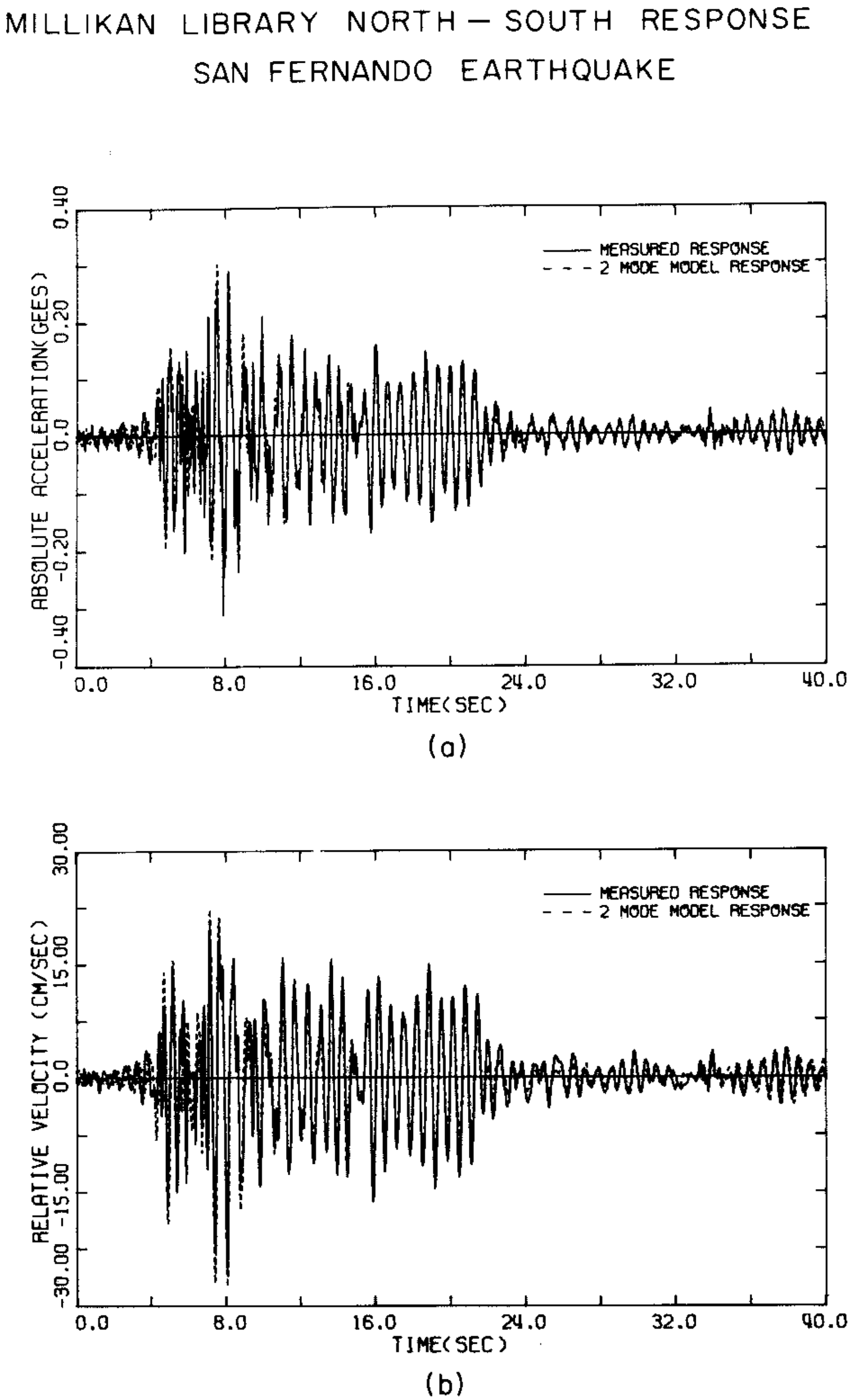


Figure 3.5 Comparison of the time histories of the measured and optimal two-mode model responses of Millikan Library to the San Fernando earthquake, north-south component. (a) Acceleration history. (b) Velocity history.

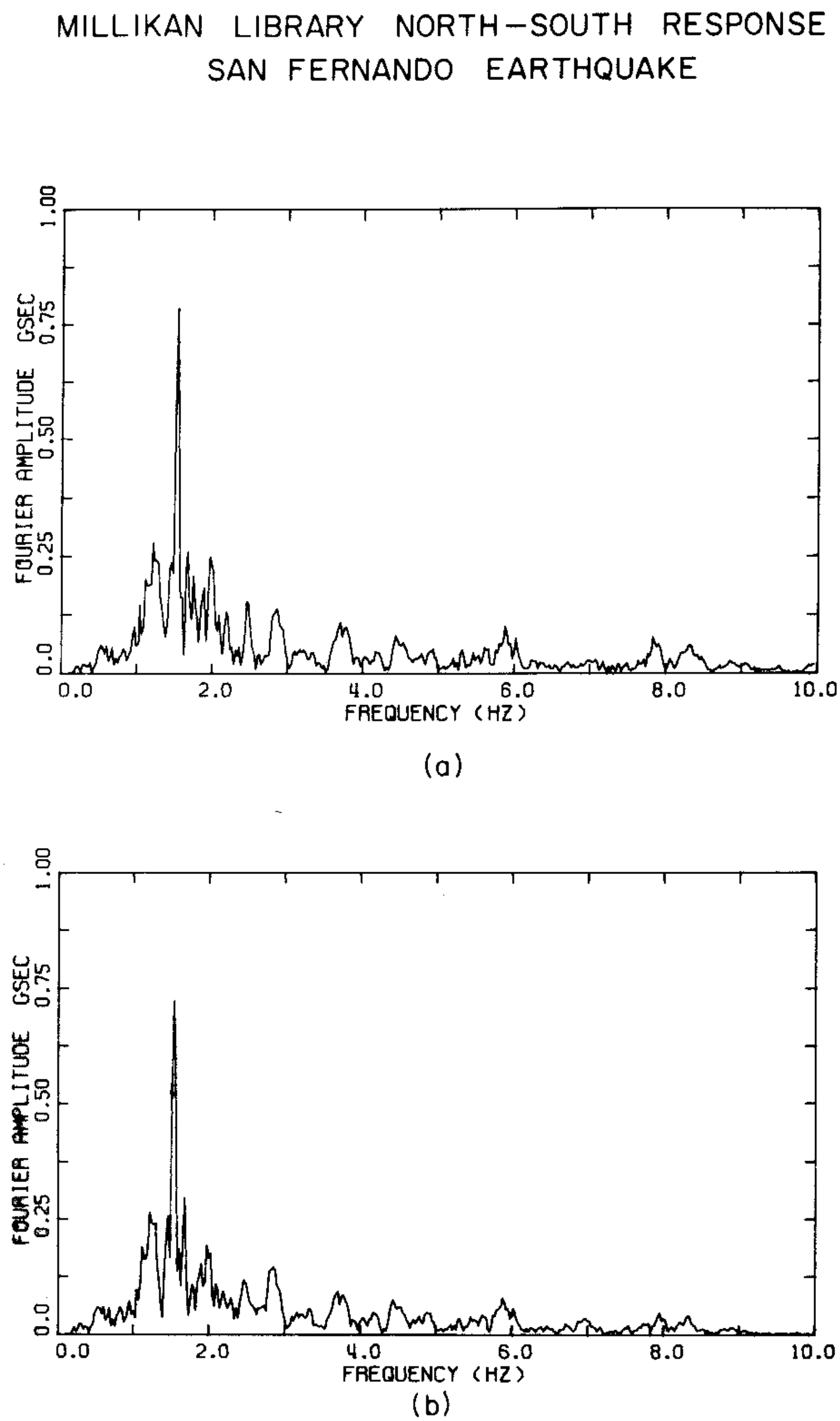


Figure 3.6 The Fourier amplitude spectrum of the north-south acceleration response of Millikan Library to the San Fernando earthquake. (a) The measured spectrum. (b) The optimal two-mode model spectrum.

effectively the damping and participation factor for each mode, for which the values were 0.54 and 0.57 respectively, indicating moderate but not severe coupling.

Udwadia and Marmarelis [13] obtained normalized acceleration errors of 0.39 and 0.32 for linear and nonlinear models respectively, compared with 0.13 for the linear model in this study. The improved fit obtained in the present study can be attributed to two features of the identification technique which overcame limitations recognized by Udwadia and Marmarelis in their non-parametric approach based on cross-correlation analysis. Firstly, in their method the input was assumed to be band-limited Gaussian white noise, while the present method utilizes the measured input. Secondly, limitations of computation time and storage allowed only truncated versions of the kernel functions of the non-parametric models to be estimated, and these estimates were subject to statistical variances in the cross-correlation because of the shortness of the available response record. The present technique is a parametric identification, requiring the estimation of only a few parameters to characterize the system. The improved fit obtained in the present study illustrates the advantages of using a parametric model and the measured input in the identification.

In the east-west direction, time-invariant models for the full forty seconds provide much poorer fits of the measured response to the San Fernando earthquake than for the north-south component (Figures 3.9 and 3.10). The mean square error is 0.27 times the mean square response for two and three mode models compared to less than 0.13 for the north-south component. The main cause of the poor fit appears to be a variation in time of the parameters, particularly the first mode period and damping.

By using a series of short time segments near the beginning of the record, it was possible to trace the lengthening of the east-west fundamental period from the pre-earthquake value of 0.69 seconds to the value of 1.00 seconds effective during much of the response. As for the north-south direction, most of the period lengthening occurs within the first ten seconds of the response, as shown by the plot of first mode period as a function of mid-interval time in Figure 3.7.

The first mode damping and participation factor also vary during the response, at first increasing and then gradually decreasing. The damping reaches a maximum of slightly over 8 percent during the largest amplitude response, falling to 3½ percent later in the motion.

However, less confidence is placed on the estimates of these parameters than upon the period. The estimation of the damping from short segments of low amplitude motion at the beginning of the records is believed to be ill-conditioned, while the measure-of-fit is insensitive to the participation factor during the essentially free vibration response in the last twenty seconds. As an extreme example of this insensitivity and the coupling between the identified values of the participation factors and dampings, the identification of a two-mode model from the last twenty seconds of the record produced participation factors of 1.16 and -0.67 and dampings of 0.034 and 0.107. The identification was repeated with the participation factors constrained to the values of 1.48 and -0.46 obtained from the full record. The measure-of-fit of 0.037 was unchanged, as were the period estimates, but the dampings changed to 0.040 and 0.075.

Some results of the sensitivity analysis are shown in Table 3.4. The normalized sensitivity matrix for the identification performed over the 40.96 second record length is listed, along with the corresponding interaction coefficients. It can be seen that interaction is insignificant except between the damping and participation factor of the same mode, with values of 0.68 and 0.75 for these interaction coefficients for the first and second mode respectively. The interaction coefficients for these two pairs of parameters are also listed for identifications performed over these segments of the record. The values for the full record are typical of those for the shorter segments, but the value of 0.82 for the segment from 20 to 40 seconds indicates that the coupling is critical there. This was the segment already discussed in which significantly different sets of participation factors and dampings produced the same measure-of-fit.

In the analyses that produced Figure 3.7, it was found that the normalized error was smaller for segments taken later in the record, primarily because the parameters changed mainly in the first part of the earthquake. Also, higher modes neglected in the model contributed significantly to the early response, while the later response consisted almost solely of vibrations of the fundamental mode.

The ability of the algorithm to identify the initial conditions as well as the system parameters was illustrated by the excellent fits obtained for segments in the later part of the response. For several of these segments there was little earthquake excitation and the response was mainly free vibrations of the fundamental mode. For example, for the segment from 20 seconds to 30.24 seconds (Fig. 3.8), a single-mode model produced an excellent measure-of-fit of 0.013, while there was insufficient response of the second mode to allow its parameters to be identified.

The parameters identified for a time-invariant model for the full 40.96 seconds record reflected the properties relevant to the largest amplitude response, while the large normalized error of 0.27 warned that such a model provided a limited representation of the system. The first mode period was identified as 0.975 seconds, close to the maximum value of 1.00 seconds. The damping for the overall record was 0.070 critical, closer to the maximum value of 0.081 than to the value of 0.035 identified from the smaller amplitude response of the second half of the record. A comparison of the measured and model time histories (Fig. 3.9) indicates that 0.070 critical damping is too high for the later part of the record as the amplitude of the model response decayed faster than the measured response beyond twenty seconds. The time variation of the damping, which occurs over a large portion of the record, appears to be more important than the 50 percent increase in the period, which occurs rapidly in the first five to ten seconds of the response, in causing the time-invariant model to be a poor representation of the structure in the east-west direction. The overall participation factor was 1.48, close to the value of 1.44 calculated from the mass distribution of the structure and Kuroiwa's measured mode shape.

The time-varying nature of the system for this record is in agreement with the conclusions of Iemura and Jennings [7]. They found that

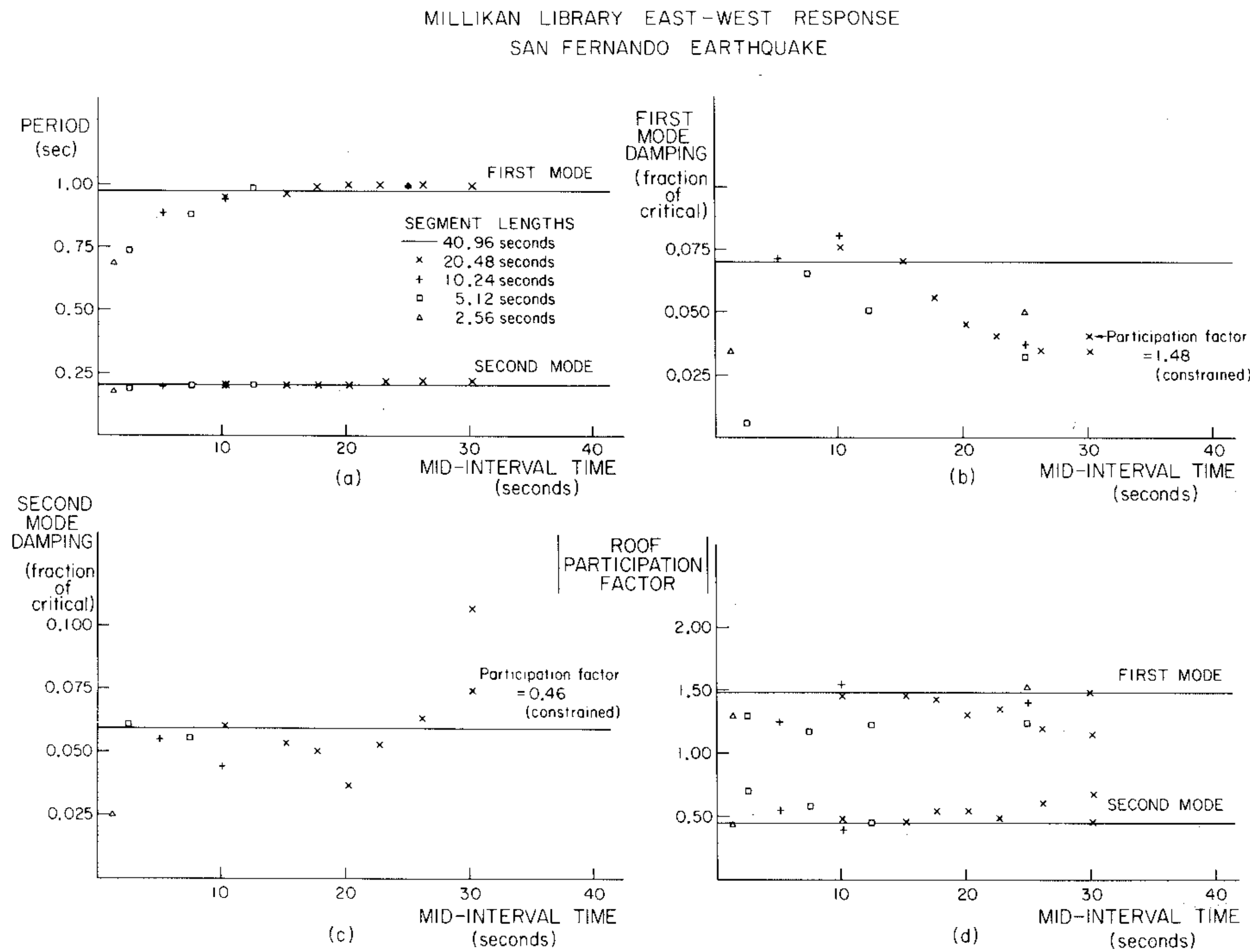


Figure 3.7 Time variation of the parameters of linear models of Millikan Library identified from segments of the San Fernando east-west response.

TABLE 3.4: SENSITIVITY ANALYSES FOR THE SAN FERNANDO EAST-WEST RESPONSE OF MILLIKAN LIBRARY

(a) The Normalized Sensitivity Matrix $\overline{S}_{ij}$ for the 0-40.96 second segment

	$\zeta_1\omega_1$	$\zeta_2\omega_2$	ω_1^2	ω_2^2	C_1	C_2
$\zeta_1\omega_1$	2.60	0.0000	-0.0021	0.0086	-2.60	-0.0049
$\zeta_2\omega_2$		0.099	-0.0015	0.0000	-0.014	-0.099
ω_1^2			131.	-0.0065	1.1	0.0081
ω_2^2					-0.0038	-0.14
C_1					5.5	0.099
C_2						0.17

(b) Interaction Coefficients $[= -\overline{S}_{ij}/(\overline{S}_{ii}\ \overline{S}_{jj})^{\frac{1}{2}}]$ for the 0-40.96 second segment

	$\zeta_1\omega_1$	$\zeta_2\omega_2$	ω_1^2	ω_2^2	C_1	C_2
$\zeta_1\omega_1$	1	0.0000	0.0001	-0.0020	0.68	0.0073
$\zeta_2\omega_2$		1	0.0004	0.0000	0.019	0.75
ω_1^2			1	0.0002	0.040	-0.0017
ω_2^2				1	0.0006	0.12
C_1					1	-0.10
C_2						1

(c) Participation Factor/Damping Interactions for Various Time Segments

Segment	$C_1-\zeta_1\omega_1$	$C_2-\zeta_2\omega_2$
0-20 sec	0.69	0.76
10-30	0.60	0.60
20-40	0.82	0.63
0-40	0.68	0.75

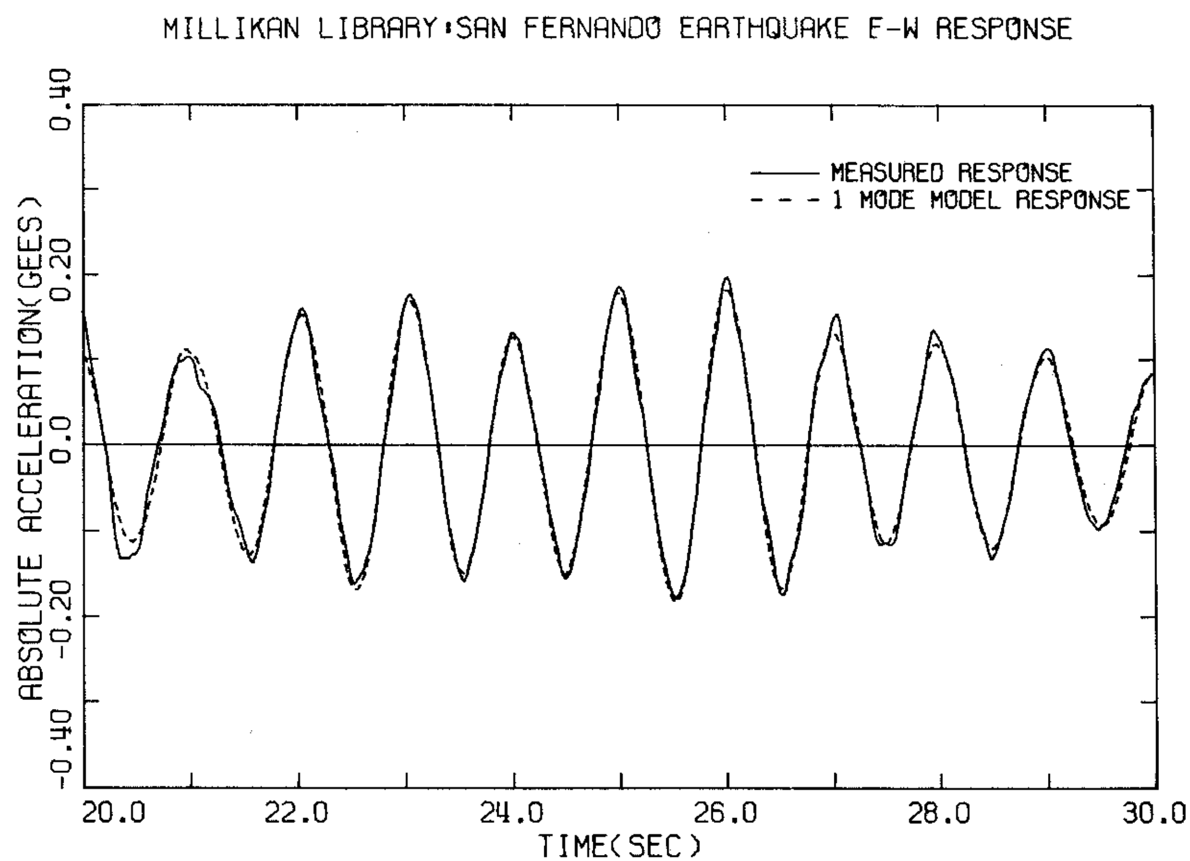


Figure 3.8 Comparison of the measured and one-mode model response acceleration for the segment from 20 to 30.24 seconds. Millikan Library, San Fernando east-west component.

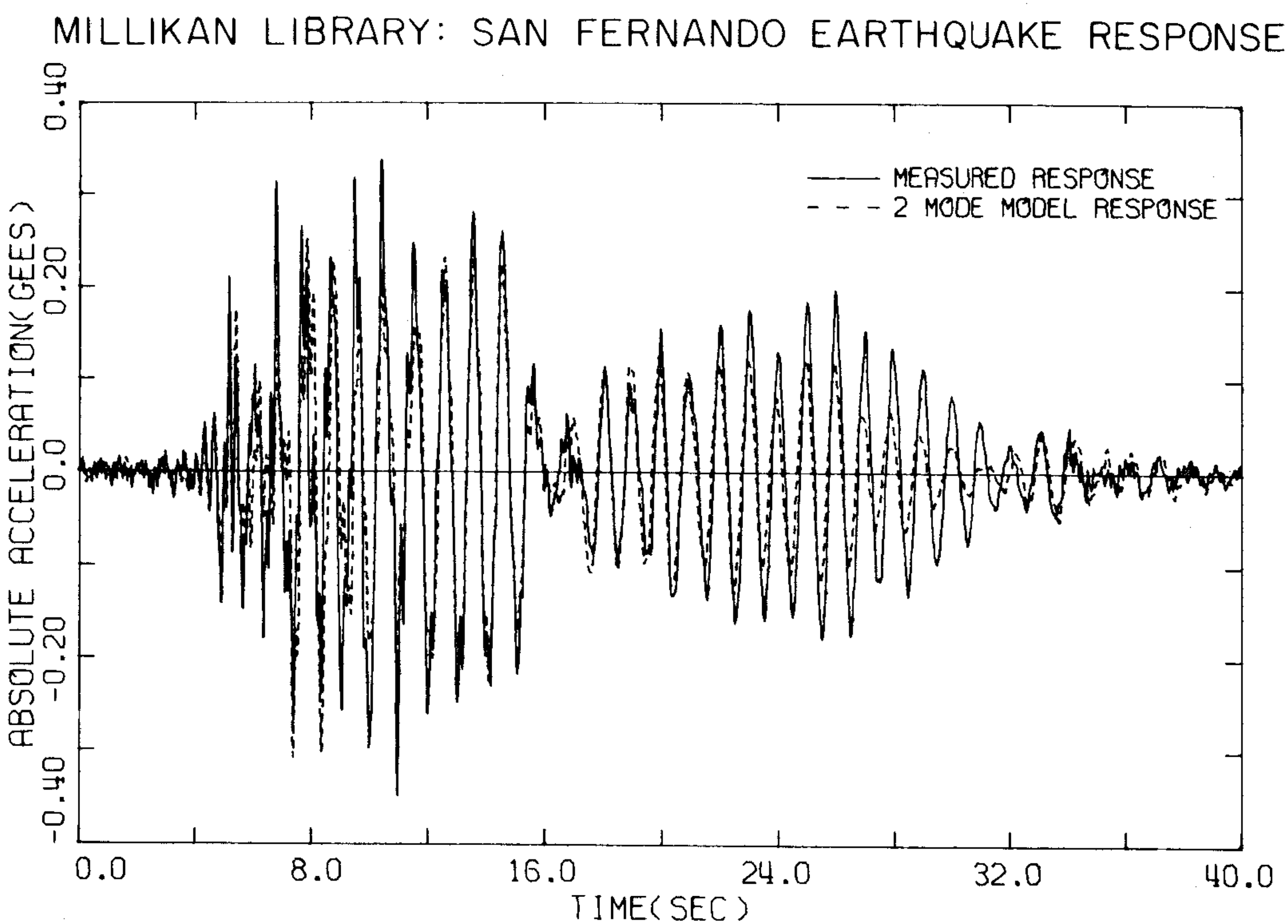


Figure 3.9 Comparison of the measured and two-mode model acceleration response of Millikan Library. San Fernando east-west component.

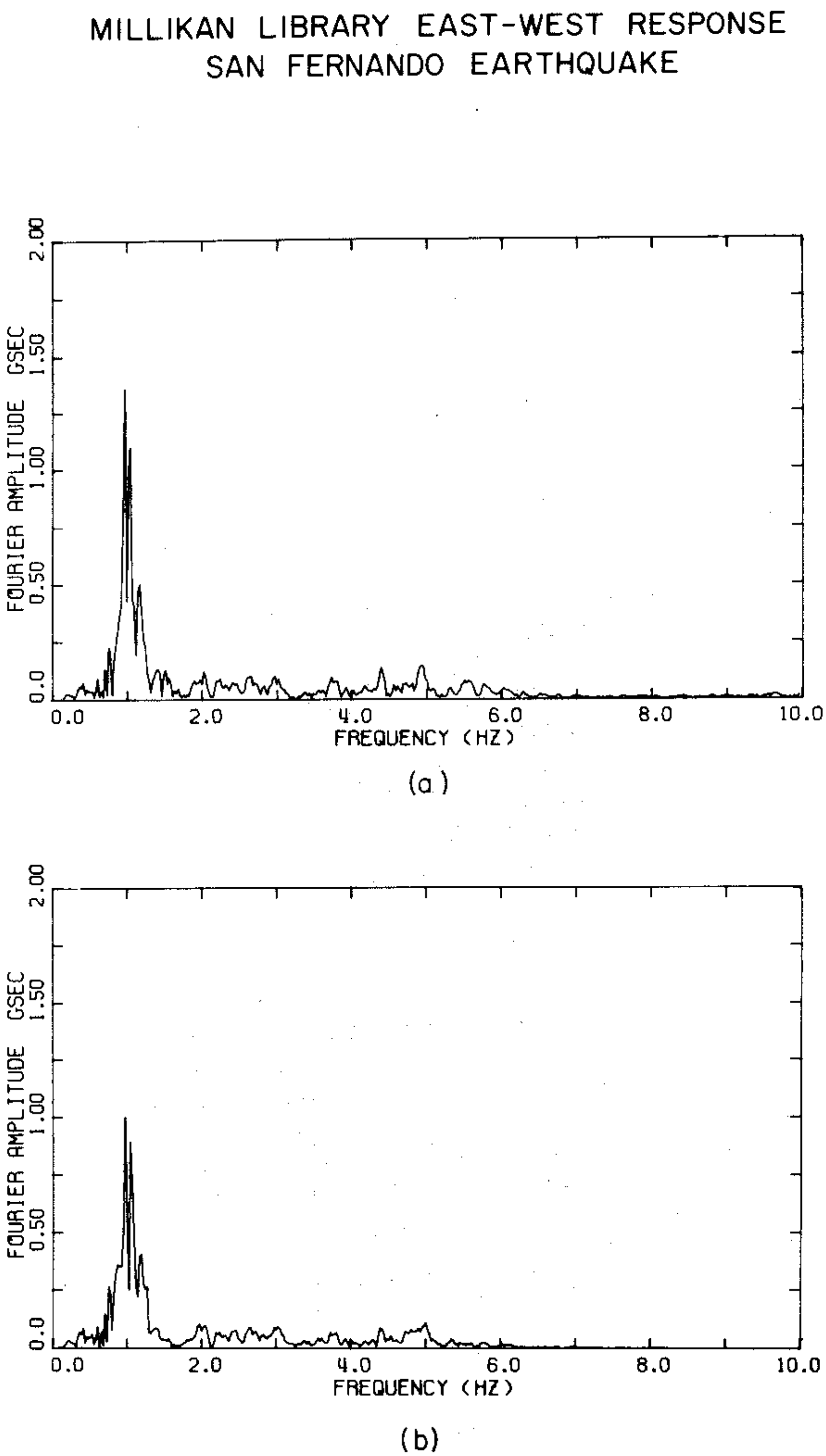


Figure 3.10 The Fourier amplitude spectrum of the east-west acceleration response of Millikan Library to the San Fernando earthquake. (a) The measured spectrum. (b) The optimal two-mode model spectrum.

much better fits were obtained with two nonstationary models, one a linear model and the other a bilinear hysteretic model, both with properties that were changed at four times during the earthquake, than with stationary linear and bilinear hysteretic models. For the segment beyond 16 seconds, their linear, one-mode model had a period of 1.0 second, a damping of 0.045 critical and a participation factor of 1.40, similar to the values of the first mode parameters identified in this study (Fig. 3.7). Their maximum damping was 0.080, close to the maximum damping found in this study for the first mode.

The results are also in basic agreement with those of Udwadia and Marmarelis who concluded that nonlinear effects made a substantial contribution to the response in the east-west direction. For a linear model they interpreted the first mode period as 0.98 seconds and the damping as 0.055 critical, but obtained a poor measure-of-fit of 0.31, similar to the value of 0.27 obtained for the single mode time-invariant model in this study. A nonlinear model produced an improved measure-of-fit of 0.19. They found that the nonlinear contribution to the response became negligible beyond about 12 seconds, in agreement with the excellent linear fits obtained in this study for segments beyond 12.5 seconds.

3.2.3 SUMMARY

In summary, application of the identification method to the response of Millikan Library during the two earthquakes gave the following results.

For the low-level vibrations of the Lytle Creek earthquake, the response of the structure was found to be similar to that in vibration tests, with fundamental periods of 0.52 seconds and 0.71 seconds in the north-south and east-west directions, respectively. The damping was between two and three percent of critical. The measured response was closely reproduced by two-mode models.

For the strong shaking of the San Fernando earthquake, the parameters took values significantly different from those found in vibration tests and in the Lytle Creek response. For the stronger excitation the structure exhibited softening behavior and a greater rate of energy dissipation. The structure was modeled well by a linear two-mode system with constant parameters for the north-south motion. However, the fundamental north-south period had lengthened from 0.52 seconds to 0.62 seconds, and the energy dissipation had increased to 6½ percent of critical viscous damping. The structure responded nonlinearly in the east-west direction, with the parameters of the equivalent linear model varying during the response. The east-west fundamental period increased by fifty percent to 1.00 second during the first ten seconds of the response, with the structure responding at this period during the remainder of the strong motion. The first mode viscous damping grew to 8 percent critical during the highest amplitude response ten seconds from the start of the record, and then gradually decreased to 3½ percent for the last fifteen seconds of response.

Sensitivity analyses indicate that the modal periods can be estimated very accurately. Unfortunately, the estimates of the dampings and participation factors are less reliable. There is also a problem of coupling between the estimates of the damping and participation factor of the same mode, which suggests that it may be necessary to constrain one of these parameters to a chosen value and to identify only the other parameters from the least squares match of the response.

A single time-invariant linear model for the entire forty second length of the San Fernando east-west record produced poor matches of the detailed time histories of the measured response. However, a series of linear models produced good matches for ten-second segments of the response. Moreover, these time-segment analyses showed that the parameter values estimated from the overall response were similar to those identified from the strongest part of the motion. This result is consistent with the observation that the overall model matched the amplitudes of the strongest motion well, but was poorer for the smaller amplitude response later in the motion for which the damping was too high (Fig. 3.9).

The information from the identification studies can be interpreted to assess the usefulness of linear time-invariant models for design calculations for structures undergoing levels of response comparable to that recorded in the east-west direction in Millikan Library during the San Fernando earthquake. At this level of response, structures exhibit nonlinear time-varying behavior but suffer no structural damage. The identification studies have shown that for this type of response a linear model is valid for calculating the maximum acceleration, velocity and displacement responses, provided that the correct values of the periods and dampings are used, allowing the common response spectrum techniques to be used. Of course, choosing the correct parameter values is a difficult task, and one of the aims of structural identification is to provide information to aid in the parameter selection. Unfortunately, the model used for calculating the maximum response will be appropriate only for the strongest part of the motion. Accordingly, it may provide misleading information about properties dependent on the detailed response history, for example the number of cycles above a certain amplitude level.

3.3 JPL BUILDING 180

3.3.1 INTRODUCTION

Building 180, the administration building at the Jet Propulsion Laboratory in Pasadena (Fig. 3.11), ranks with Millikan Library as one of the structures with the most extensively studied dynamic properties. Vibration tests have been performed on the building since the construction phase, beginning with Nielsen [11], and have revealed considerable variations in the dynamic properties as a result of earthquake shaking and structural alterations. The responses to three earthquakes have been recorded in the building, including strong shaking from the San Fernando earthquake which was centered fifteen miles to the northwest of JPL. The earthquake response has been analyzed previously by Brandow and Johnston [2], Wood [15, 16], Udwadia and Trifunac [14], and Beck [1]. The availability of much vibration test data, the opportunity to perform identifications for the different response levels of the three earthquakes, and the chance to compare the results with those of the earlier analyses make the study of this structure attractive.

The height of the nine-story building designed in 1961 is 146 feet from the foundation to the roof, and the horizontal dimensions are 40 feet by 220 feet. The lateral loads in the transverse north-south direction are resisted by welded steel spandrel trusses, rather than

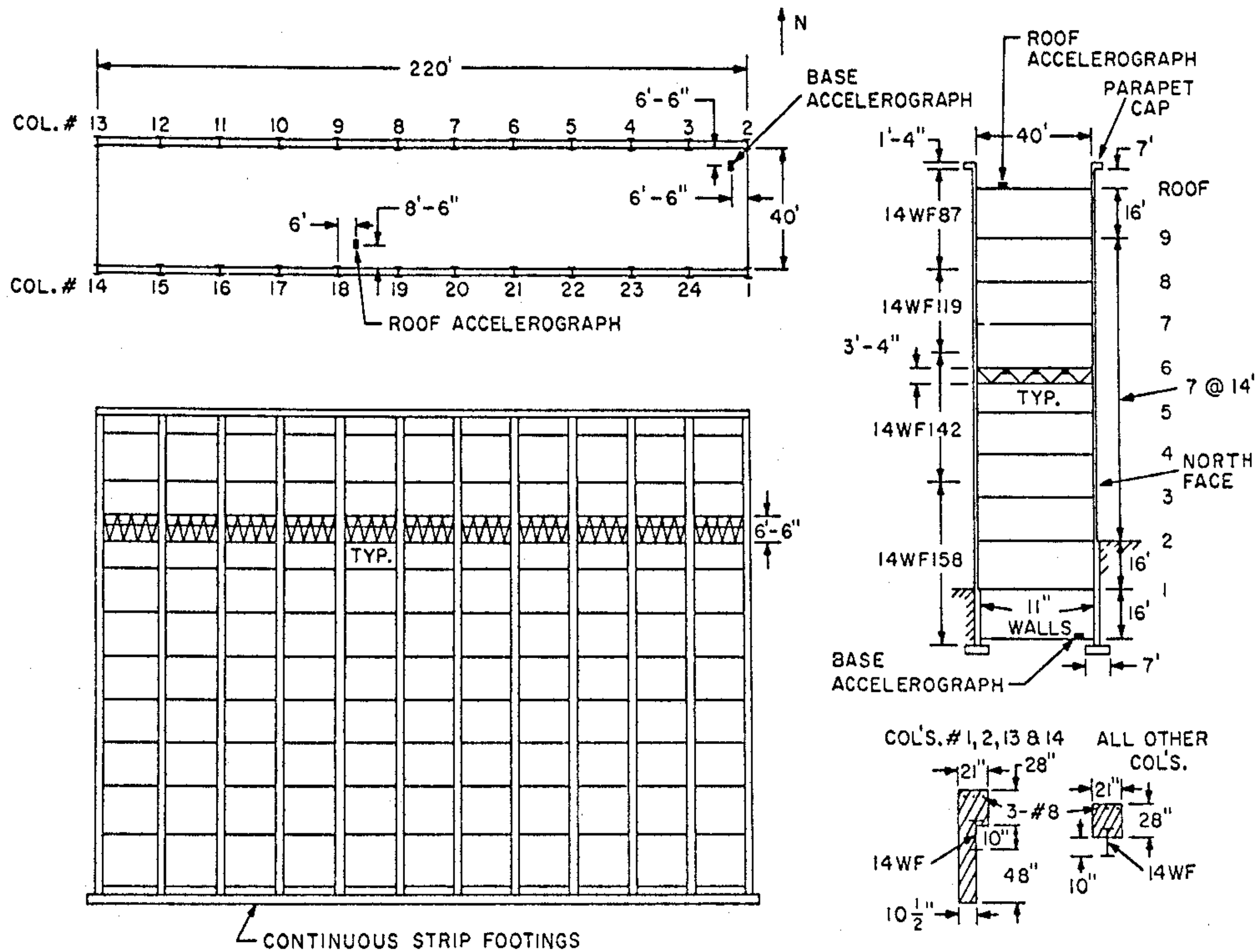


Figure 3.11 Typical floor plan and longitudinal and transverse sections of JPL Building 180 (from Foutch et al. [4]).

the I-beams more typical of Southern California, and by steel columns partially encased in concrete. The longitudinal loads are carried by a frame consisting of steel girders and columns (Fig. 3.11). Glass curtain walls form the long north and south faces of the building, while precast concrete panels supported by the steel frame make up the east and west end walls. The foundation system consists of continuous strip footings running longitudinally. More detailed descriptions of the building and site conditions are contained in the design report by Brandow and Johnston [2], and in the studies by Nielsen [11] and Wood [15,16]. A summary of building properties and a presentation of the data from the San Fernando earthquake can be found in a report by Foutch, Housner and Jennings [4].

The vibration test data of Table 3.5 reveal several interesting features. The most obvious is the change of the periods between the different tests. The variations during the construction phase were to be expected as the mass and effective stiffness changed with the addition of components. The dominant periods shown during the San Fernando earthquake were considerably longer than those measured in the vibration tests. The ambient tests conducted soon after the earthquake (July 1971) showed a lengthening of the periods compared to the pre-earthquake tests, but not as dramatically as occurred in the earthquake response itself. Foutch and Housner [3] suggested that the changes were the result of "a combination of concrete crushing combined with non-structural damage." Later tests in July 1975 and June 1976 showed some recovery of stiffness, at least partially due to strengthening work performed after the earthquake. There was no significant change in the periods between 1975 and 1976.

Another point to note in the vibration test data is the closeness of the torsional mode periods to the translational periods, particularly in the east-west direction. This raises the possibility that although the building is nominally symmetric, some small eccentricity may cause significant horizontal coupling between the torsional and translational modes. Wood was unable to identify torsional response from the earthquake records, a difficult task with response records from only one location. However, as discussed later, there is a double peak, which suggests the possibility of torsional response, near the second mode frequency in the east-west Fourier spectrum for the San Fernando earthquake.

The previous analyses of the earthquake behavior of the building illustrated three different approaches to the problem. The methods used by Brandow and Johnston [2] and Wood [15,16] involved trial-and-error modification of simple models of the structure synthesized from the design data. The properties of these models are summarized in Table 3.6.

Trifunac and Udwadia [14], who considered the responses to all three earthquakes, Borrego Mountain (April 8, 1968), Lytle Creek (September 12, 1970) and San Fernando (February 9, 1971), used a moving-window Fourier analysis approach. Beck [1] performed a parametric time-domain identification using the east-west component of the San Fernando response.

TABLE 3.5: MODAL PERIODS OF JPL BUILDING 180 FROM VIBRATION TESTS AND THE FOURIER SPECTRA OF THE SAN FERNANDO EARTHQUAKE

Test	S82E (Longitudinal)			S08W (Transverse)			Torsion	
	T_1	T_2	T_3	T_1	T_2	T_3	T_1	T_2
Pre-Earthquake Tests								
Forced vibration during construction - Nielsen 1963	0.99	0.33	0.20	1.03	0.31	0.16	0.92	0.27
Man-excited 1963 - Nielsen (Ready for occupancy)	0.91	0.29	-	0.88	0.29	-	0.93	0.26
San Fernando Earthquake Spectra								
0-40 seconds	1.29	0.42	0.26	1.44	0.44	0.24	-	-
40-98 seconds	1.11	0.38	-	1.46	0.39	0.22	-	-
Post Earthquake Tests								
Ambient July 1971 - Teledyne	1.05	0.33	0.20	1.11	0.35	0.16	0.91	0.28
Man-excited February 1972 - Teledyne	1.00	0.30	-	1.15	0.31	-	0.93	-
Ambient July 1975 (after strengthening)	0.96	-	-	1.04	-	-	0.88	-
Ambient June 1976	0.95	-	-	1.05	-	-	0.88	-

TABLE 3.6: SYNTHESIZED MODELS OF JPL BUILDING 180

(a) Wood's Full Composite Models

Mode	S08W		S82E	
	Period (Sec)	Participation Factor	Period (Sec)	Participation Factor
1	1.163	1.322	1.089	1.288
2	0.338	-0.487	0.361	-0.438
3	0.165	0.253	0.213	0.234
4	0.095	-0.140	0.147	-0.127
5	0.061	0.077	0.108	0.065
6	0.044	-0.035	0.084	-0.032

(b) Wood's Partial Composite Models

Mode	S08W		S82E	
	Period (Sec)	Participation Factor	Period (Sec)	Participation Factor
1	1.457	1.306	1.209	1.303
2	0.469	-0.460	0.418	-0.463
3	0.259	0.242	0.257	0.240
4	0.168	-0.139	0.184	-0.113
5	0.118	0.079	0.142	0.047
6	0.087	-0.042	0.114	-0.018

TABLE 3.6: SYNTHESIZED MODELS OF JPL BUILDING 180

(Cont'd)

(c) Wood's Refined Models (to fit earthquake responses)

Mode	S08W			S82E		
	Period (sec)	Damping (%)	Participation Factor	Period (sec)	Damping (%)	Participation Factor
1	1.440	3.0	1.306	1.290	4.0	1.303
2	0.436	3.0	-0.460	0.420	6.0	-0.463
3	0.235	4.0	0.242	0.260	6.0	0.240
4	0.167	2.0	-0.139	0.184	5.0	-0.113
5	0.117	2.0	0.079	0.142	2.0	0.047
6	0.087	2.0	-0.042	0.114	2.0	-0.018

(d) Brandow and Johnston's Models

Modal Periods (Seconds)

	S08W			S82E		
	T_1	T_2	T_3	T_1	T_2	T_3
Initial Model	1.12	0.37	0.22	1.06	0.38	0.23
Adjusted Model	1.25	0.42	0.24	1.23	0.45	0.28

Damping (% Critical)

Time Segment	S08W	S82E
0-5 sec	6.6	5.0
5-10 sec	4.9	3.7
10+ sec	3.3	2.5

The present method is also a parametric identification using the same type of linear modal model as Beck, but utilizes the data in the frequency domain. The model parameters to be identified are the same as those in Wood's study, except Wood constrained the participation factors to the values calculated from his synthetic model. The approach also has parallels to Wood's analysis in the sense that the response acceleration transform is the quantity matched, although Wood considered only the amplitude spectrum rather than the full complex-valued transform.

A point to note about the synthesized models of Table 3.6 is the relative insensitivity of the participation factors of the lower modes to the different stiffness distributions which produce substantially different modal periods. It is this insensitivity which suggests constraining the participation factors to predetermined values. As noted earlier, if valid this approach allows tracing of the variation in damping.

3.3.2 FOURIER SPECTRA ANALYSES

The Fourier spectra of the ground accelerations of the three earthquakes recorded in the structure are markedly different in frequency content and amplitude. The spectra of the magnitude 6.4 Borrego Mountain earthquake are dominated by low frequency content, peaking around 1.0 to 1.5 Hz with amplitudes near 10 cm/sec in both directions, with little content beyond 6 Hz. It seems that most of the ground motion at the JPL site from this earthquake was caused by long period surface waves. The magnitude 5.4 Lytle Creek earthquake of September 12, 1970 produced much broader spectra, peaking at around 10 cm/sec again for both horizontal components, but near a frequency of 2 Hz, with significant frequency content to at least 15 Hz. The magnitude 6.4 San Fernando earthquake produced much stronger ground shaking. Like the Lytle Creek earthquake, it was a broad band excitation. The peak of the S82E component occurred at about 3 Hz with an amplitude of approximately 130 cm/sec, while the S08W component contained peaks of about 65 cm/sec between 3 and 5 Hz. The largest amplitude components were at frequencies up to 7 Hz, but there was some motion up to at least 20 Hz.

The different frequency content of the various ground acceleration records was reflected in the response records. The Borrego Mountain responses in both directions were almost purely in the fundamental mode with only slight second and third mode response, with no higher mode responses discernible in the Fourier spectra plots (Fig. 3.12a). The Lytle Creek response (Fig. 3.13a) showed large peaks at the first three modal frequencies, with significant higher mode response in the S08W component. The Borrego Mountain excitation caused much higher first mode response than the Lytle Creek excitation in both directions, but Lytle Creek caused larger amplitude response in the second and third modes. The San Fernando S82E response spectrum (Fig. 3.14a) showed three modes contributing strongly, although at frequencies lengthened from the values seen in vibration tests and in the other earthquakes. There was virtually no response beyond 5 Hz for this component of the San Fernando earthquake. The S08W response reflected in the broader band excitation in this direction, with significant response up to 9 Hz.

TABLE 3.7: EARTHQUAKE RESPONSE CHARACTERISTICS OF JPL BUILDING 180

Response Quantity	Earthquake Component					
	BM S82E	BM S08W	LC S82E	LC S08W	SF S82E	SF S08W
Maximum of ground acceleration spectrum	9 cm/sec	11	9	11	130	64
First mode spectral amplitude	92	97	26	27	910	285
Second mode spectral amplitude	10	9	20	22	330	125
Third mode spectral amplitude	4	5	5	18	85	120
Maximum ground acceleration	7 cm/sec^2	7	15	24	208	139
Maximum response acceleration	31	23	25	37	375	206
Maximum relative velocity response	4.5 cm/sec	4.7 cm/sec	2.9 cm/sec	3.6 cm/sec	31 cm/sec	35 cm/sec

BM = Borrego mountain earthquake LC = Lytle Creek earthquake SF = San Fernando earthquake

The Fourier amplitudes at the modal peaks of the acceleration response spectra for the three earthquakes are summarized in Table 3.7, along with the maximum accelerations and relative velocity responses.

The Fourier amplitude spectra and transfer functions of the building are useful for providing initial estimates of the parameter values. However, the erratic nature of the transfer functions, illustrated by Figure 3.15, makes estimation of the modal properties from the dominant peaks difficult, and indeed was one of the reasons for the development of more systematic identification techniques.

3.3.3 BORREGO MOUNTAIN AND LYTLE CREEK IDENTIFICATION STUDIES

The parameters identified from the Borrego Mountain and Lytle Creek responses are listed in Table 3.8. The identified periods agree reasonably well with the peaks of the Fourier amplitude spectra and transfer functions, but the resonant peaks of the models generally underestimate the measured peaks. This is possibly due to a slight overestimation of the damping which broadens the peaks, to compensate for the broadening caused by the slight period lengthening over the course of the response. The matches of the Borrego Mountain responses are excellent for both directions, as reflected by the normalized errors of 0.050 and 0.051. The Lytle Creek fits are poorer, mainly due to the presence of more modes in the response than are accounted for in the models. In this respect, the response of JPL Building 180 to the Lytle Creek earthquake was more complicated than the response of Millikan Library for which, with its wide separation of modal periods, fewer modes were excited.

The modal periods in even these low amplitude earthquake responses are from ten to thirty percent longer than those measured in man-excited tests on the completion of the structure. The fundamental mode dampings are higher by factors of four to eight than in Nielsen's forced vibration tests, in which the damping was 0.6 percent in both directions.

There is no simple explanation for the variations in damping between the different components. It would be expected that the stronger excitation in a given direction would cause the larger response, which in turn would produce the greater period lengthening and be associated with the larger damping. However, because of the different frequency content of the Borrego Mountain and Lytle Creek earthquakes, it is difficult to rank one as stronger than the other at the JPL site. The Borrego Mountain components are considerably stronger than the Lytle Creek components in terms of properties based on low frequency content, such as the first mode spectral amplitude and the maximum relative velocity response, while the Lytle Creek components are stronger at high frequencies, reflected in the maximum ground accelerations and the higher mode spectral amplitudes (Table 3.7). In terms of maximum acceleration responses, the Borrego Mountain is stronger in the S82E direction, while the order is reversed for the other direction. A consideration of the damping values in conjunction with the amount of period lengthening from the vibration tests confuses the picture even more. The Borrego Mountain responses are associated with longer modal periods in each case but, contrary to expectation, lower values of damping. In conclusion, it would appear that damping values cannot be correlated directly with the amount of period lengthening. It would appear that for earthquakes of this strength, no single damping value can be specified as appropriate. The average value for the first modes is 3½ percent, but the amount of scatter suggests that design calculations should be performed for values of 3 and 5 percent damping for the fundamental modes to provide approximate bounds on the response. A value of 5 percent appears sufficiently accurate for the higher modes.

The periods identified from the Borrego Mountain responses in particular are very close to those of Wood's full composite model (Table 3.6a). Wood considered this model appropriate when the concrete encasing the steel columns remained uncracked, as occurred in these two earthquakes. For larger amplitude responses in which the concrete would crack, Wood suggested the partial composite model which was based on the assumption that the concrete in the flexural tension zones of the columns provided no flexural stiffness.

The quality of the matching of the measured and model responses for these two earthquakes can be seen in Figures 3.12 and 3.13. These figures show the acceleration transform and time history matches with the two-mode models of Table 3.8 for the S82E components of the Borrego Mountain and Lytle Creek earthquakes.

In summary, linear models provide good matches of the recorded responses for these two earthquakes since for the relatively small shaking, compared to the San Fernando response for example, the parameters vary only slightly over the record, apart from an initial rapid lengthening of the periods from the vibration test values. For design purposes, the periods and participation factors of Wood's full composite

TABLE 3.8: IDENTIFIED PARAMETER VALUES FROM THE BORREGO MOUNTAIN AND LYTLE CREEK RESPONSES

(a) S82E Component

Mode	Borrego Mountain (*)			Lytle Creek (*)		
	Period (Sec)	Damping	Participation Factor	Period (Sec)	Damping	Participation Factor
1	1.092	0.029	1.250	1.016	0.047	1.327
2	0.362	0.051	-0.376	0.322	0.054	-0.382
	Measure-of-fit J = 0.050			J = 0.178		

(b) S08W Component

Mode	Borrego Mountain (*)			Lytle Creek (**)		
	Period (Sec)	Damping	Participation Factor	Period (Sec)	Damping	Participation Factor
1	1.147	0.027	1.423	1.128	0.035	1.304
2	0.350	0.047	-0.497	0.335	0.046	-0.556
3	-	-	-	0.191	0.004	0.052
4	-	-	-	0.183	0.029	0.126
	Measure-of-fit J = 0.051			J = 0.229		

(*) Record segment 0-20.48 seconds
Identification frequency band 0.34 - 7.76 Hz

(**) Record segment 0-20.48 seconds
Identification frequency band 0.34 - 11.67 Hz

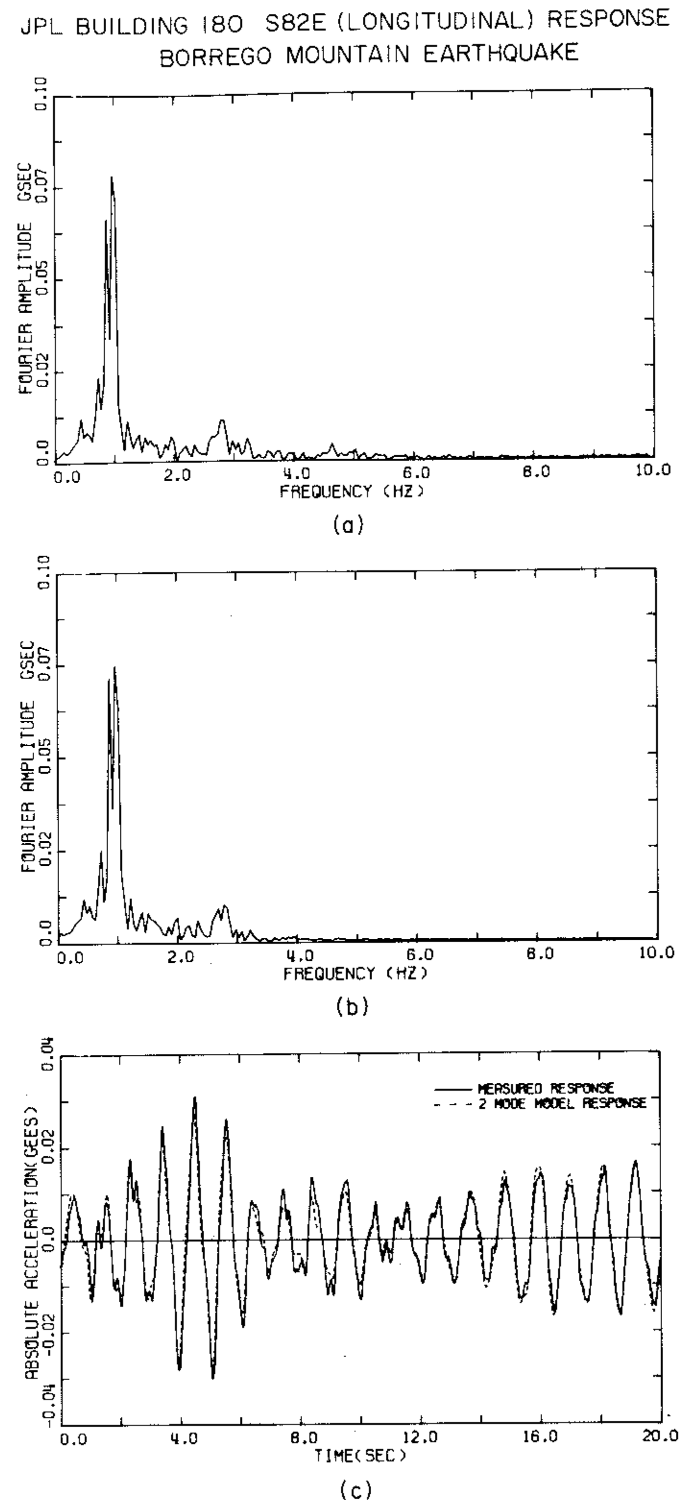


Figure 3.12 JPL Building 180, Borrego Mountain S82E component. Fourier amplitude spectrum of (a) the measured acceleration response; (b) the two-mode model acceleration response; (c) acceleration histories.

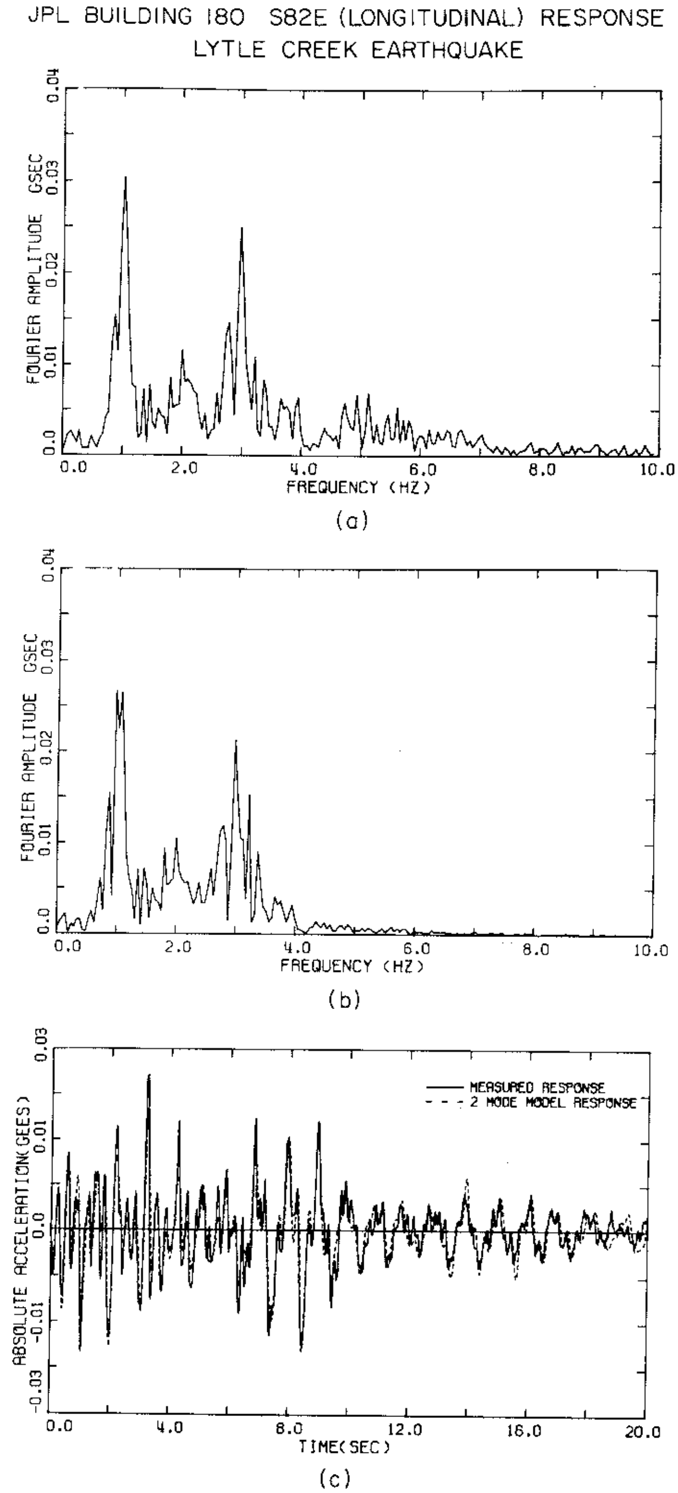


Figure 3.13 JPL Building 180, Lytle Creek S82E component. Fourier amplitude spectrum of (a) the measured acceleration response; (b) the two-mode model acceleration response; (c) Acceleration histories.

models appear satisfactory for low levels of earthquake excitation of this type. The choice of the dampings is not clear-cut, particularly for the fundamental modes. Values of 3 and 5 percent for the fundamental mode, with 5 percent for the higher modes, should provide approximate bounds on the response.

3.3.4 IDENTIFICATIONS FROM THE SAN FERNANDO DATA

The San Fernando data were different in that significant variations with time of the effective linear parameters could be traced over the course of the response. Most of the variation occurred in the first ten seconds of the response, up to the time of the largest motion. Thereafter the periods remained fairly constant, although the damping reduced as the amplitude of the motion decreased. Figures 3.16 and 3.17 show the effective linear parameters for the two horizontal components as functions of the mid-interval time of the segment of response from which they were identified.

S82E RESPONSE

The parameter values identified for a time-invariant three-mode model of the San Fernando east-west (S82E) response are listed in Table 3.9a, along with the variations which would cause a ten percent change in the measure-of-fit. The first and second mode periods of 1.26 ± 0.01 seconds and 0.38 ± 0.01 seconds were lengthened by 38 and 31 percent over the pre-earthquake vibration test values of 0.91 and 0.29 seconds. They were also considerably longer than the values of 1.09 seconds and 0.36 seconds effective during the response to the Borrego Mountain earthquake. The first two modal dampings were 3.8 and 5.3 percent of

TABLE 3.9: IDENTIFIED PARAMETER VALUES FROM THE SAN FERNANDO RESPONSE

(a) S82E Component

Record segment = 0-40.96 seconds

Identification frequency band = 0.37 - 5.84 Hz

Mode	Period (Sec)	Damping	Participation Factor
1	1.255 ± 0.010*	0.038 ± 0.008	1.44 ± 0.17
2	0.38 ± 0.01	0.053 ± 0.02	-0.235 ± 0.065
3	0.30 ± 0.02	0.12 ± 0.07	-0.41 ± 0.15

Measure-of-fit E = 0.135

(b) S08W Component

Record segment = 0-40.96 seconds

Identification frequency band = 0.27 - 5.86 Hz

Mode	Period (Sec)	Damping	Participation Factor
1	1.42 ± 0.04	0.064 ± 0.025	1.49 ± 0.25
2	0.40 ± 0.03	0.19 ± 0.08	-0.76 ± 0.22
3	0.21 ± 0.02	0.11 ± 0.09	0.32 ± 0.20

Measure-of-fit E = 0.270

*Error bounds are for a 10 percent change in E.

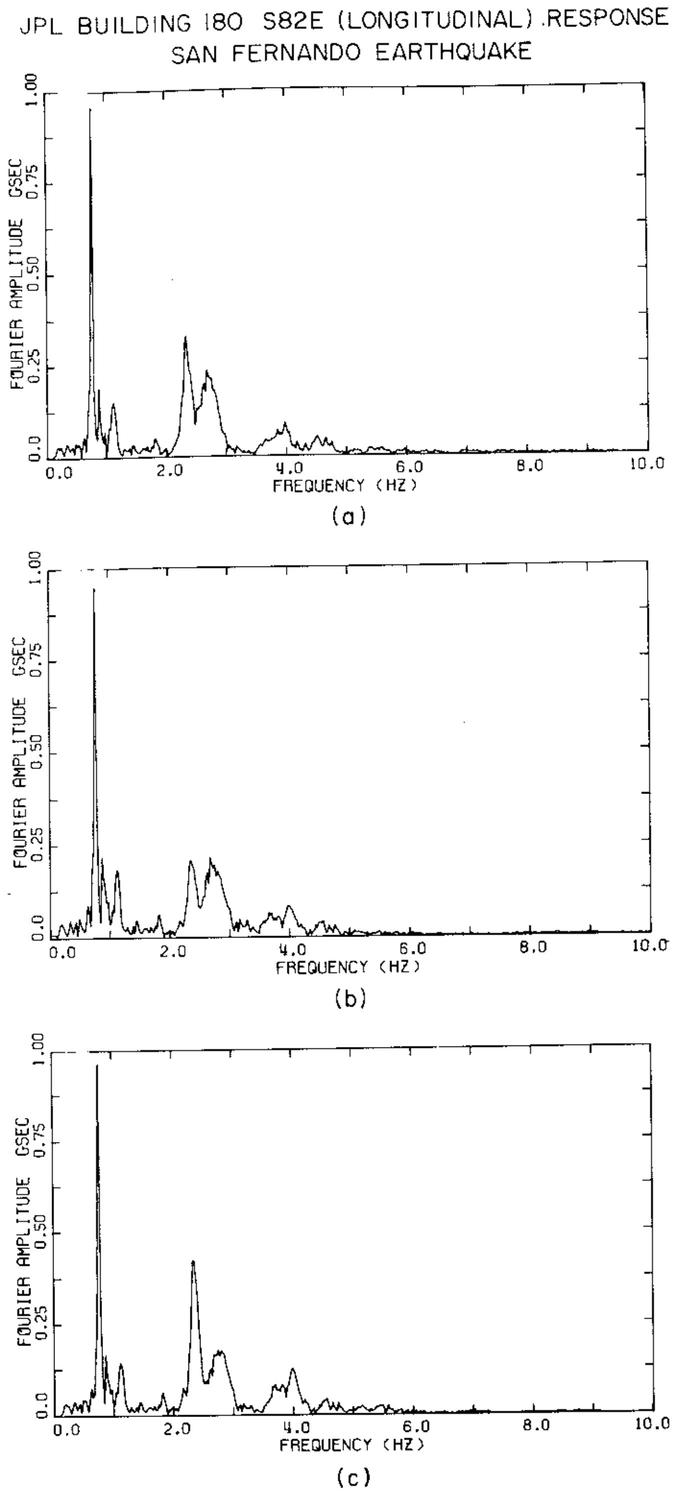


Figure 3.14 JPL Building 180, San Fernando S82E component. Fourier amplitude spectrum of (a) the measured acceleration response; (b) the optimal three-mode model acceleration response; (c) Wood's six-mode model acceleration response.

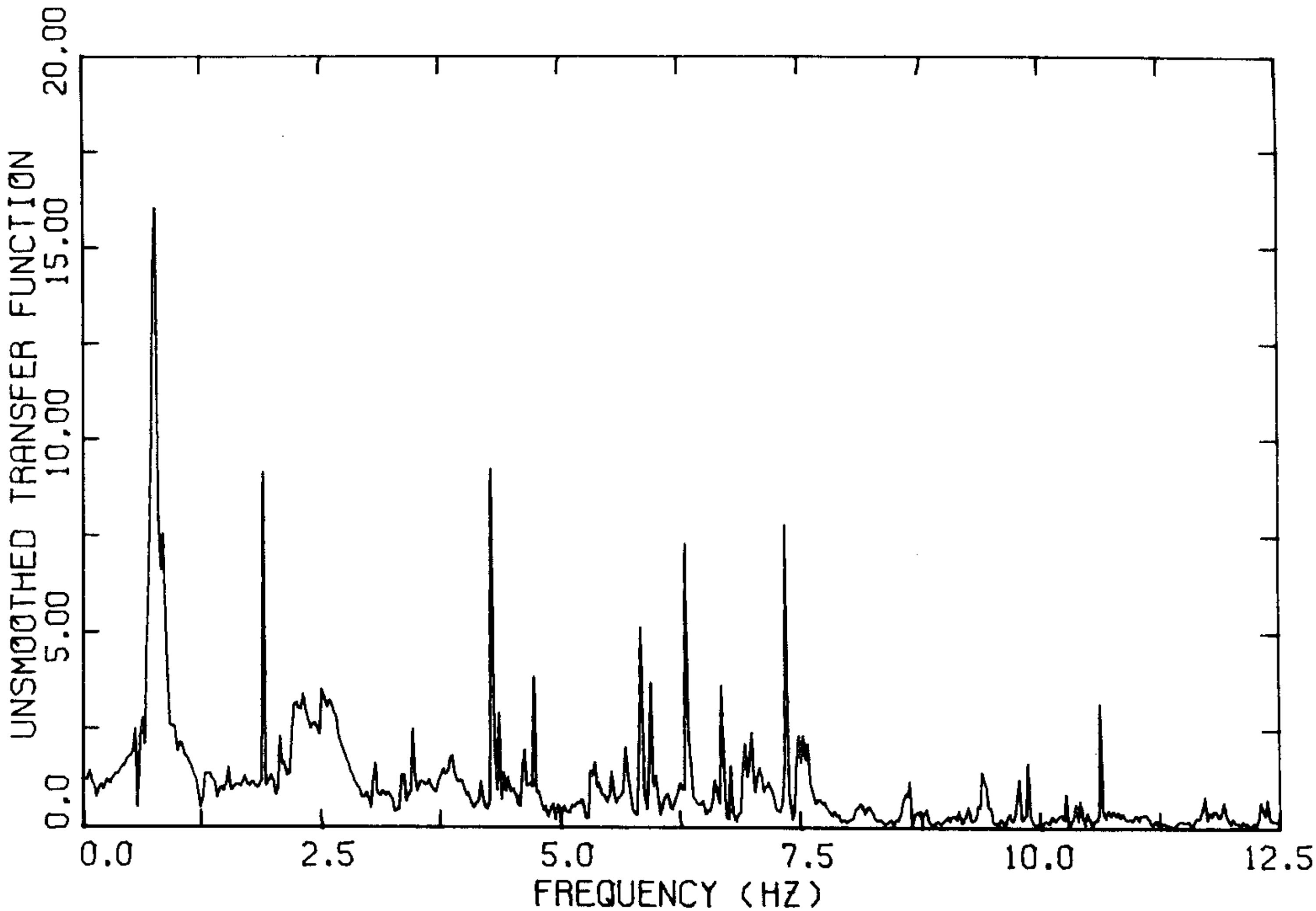


Figure 3.15 JPL Building 180, San Fernando S82E component. Amplitude of the unsmoothed transfer function between the absolute acceleration recorded in the basement and on the roof.

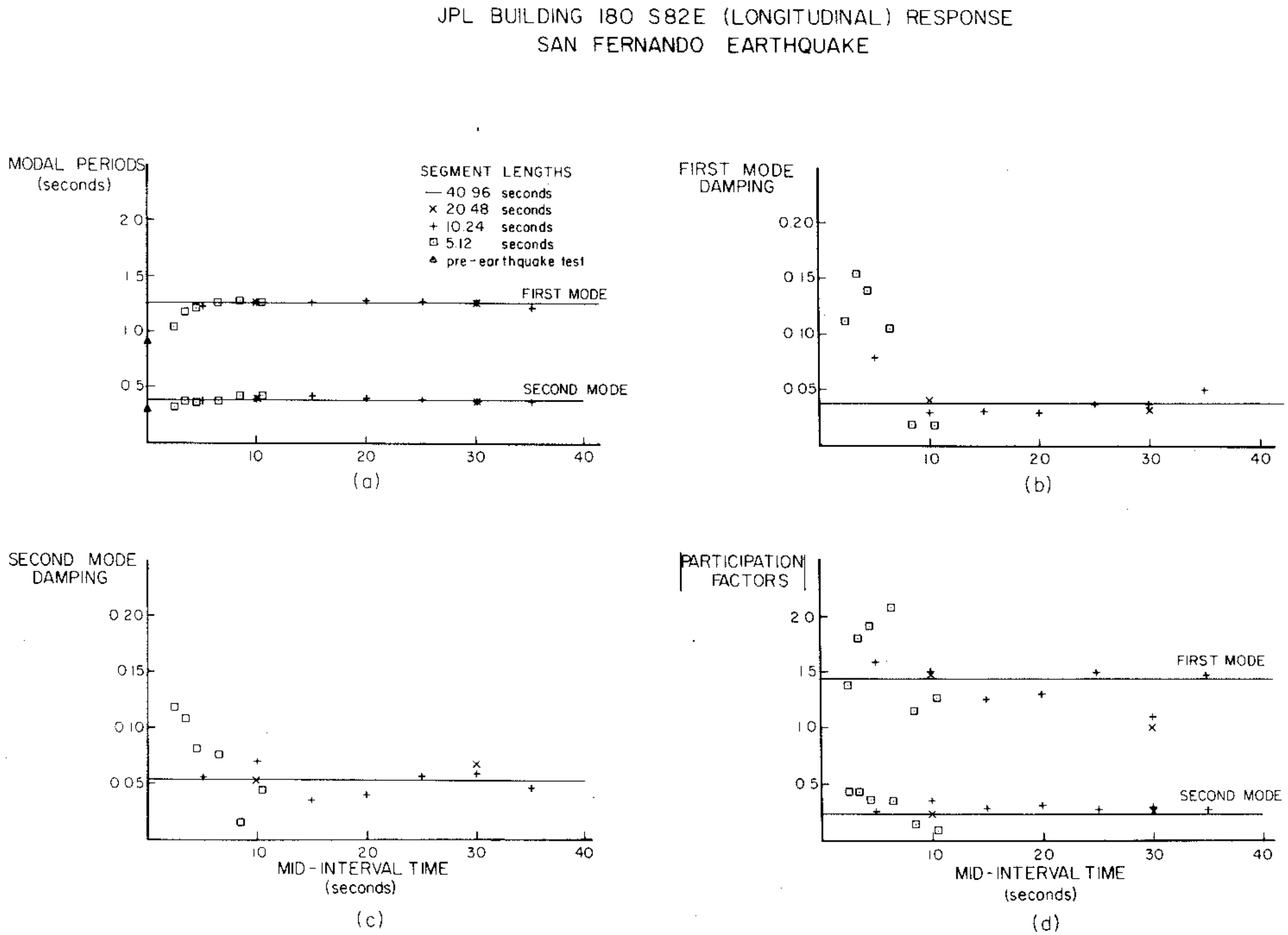


Figure 3.16 Time variation of the parameters of linear models of JPL Building 180 identified from segments of the San Fernando S82E response.

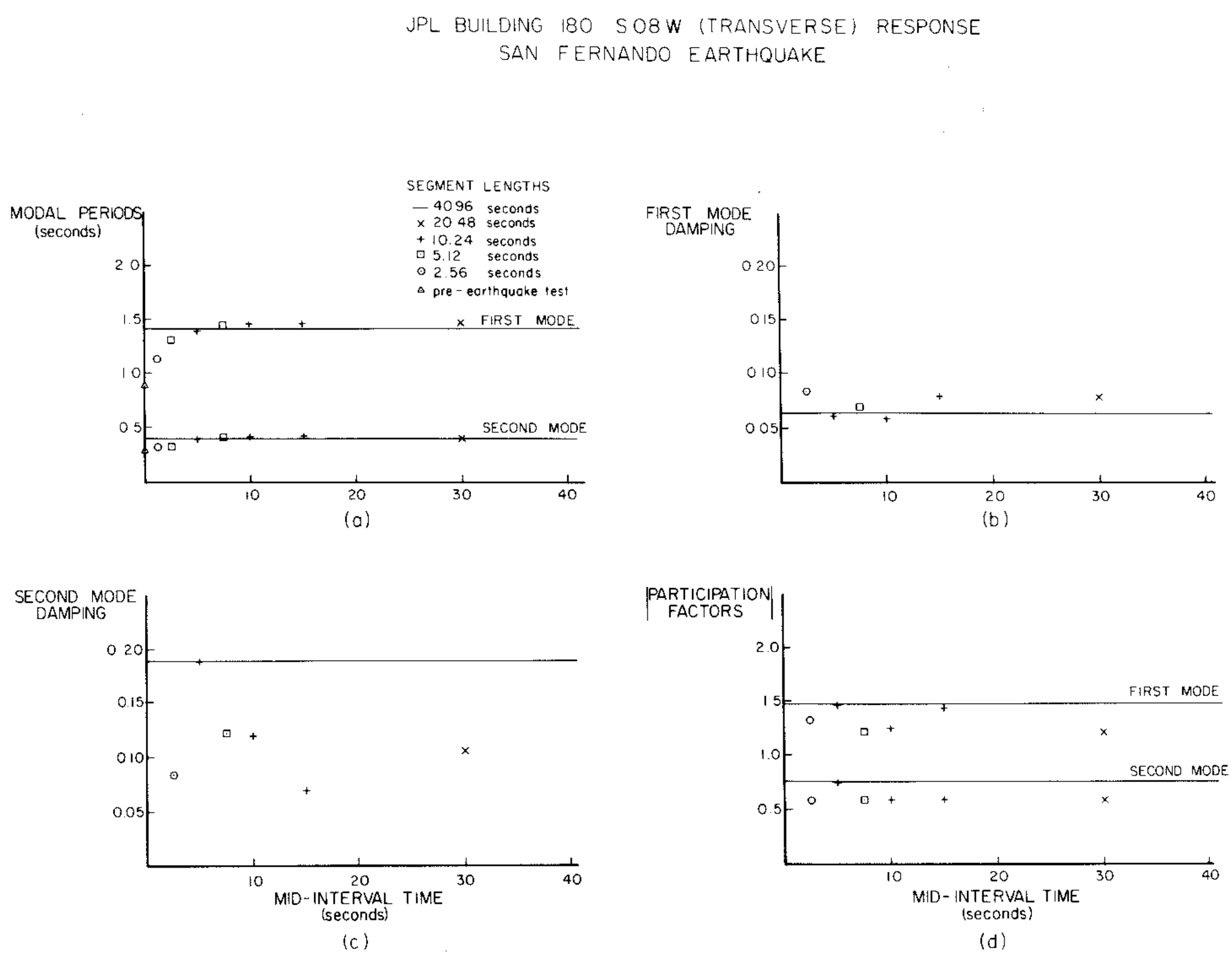


Figure 3.17 Time variation of the parameters of linear models of JPL Building 180 identified from segments of the San Fernando S08W response.

critical. The first mode value, somewhat surprisingly, was less than the Lytle Creek damping of 4.7 percent of critical. It was thought that the damping in the considerably higher amplitude response of the San Fernando earthquake would be greater than for the Lytle Creek response. The San Fernando value was in agreement with Wood's value of 4 percent for this component. Performing the identifications for both the Lytle Creek and San Fernando records over narrower frequency bands so as to include only the first and second mode response peaks produced no significant changes in the first mode dampings.

The sensitivity analyses indicated that the relative accuracy of the corresponding parameters estimated from the San Fernando data was better for JPL Building 180 than for Millikan Library. Again the period estimates were the most accurate, followed by the participation factors and dampings. The estimates of the first mode parameters were more accurate than those of the higher modes. There was also significant coupling between the estimates of the damping and participation factors, with interaction coefficients of 0.6 and 0.7 for the first two modes. Somewhat surprising was the extent of interaction between the estimate of the modal frequencies and participation factors, particularly for some of the shorter segments. Unlike the interaction of the damping and participation factor, there is no simple physical explanation for this behavior. For example, for an identification performed for the 15-25 second interval, the interaction coefficients between these parameters were 0.71 and 0.52 for the first two modes, greater than the values of 0.66 and 0.45 for the interactions between the estimates of the dampings and participation factors. Such high values for the interaction coefficients indicate computational difficulties involved in extracting accurate estimates of the parameters from the data.

The time history matches achieved by the three-mode model identified from the acceleration transform of the full record are shown in Figures 3.18 to 3.21. These figures also show the matches given by Wood's six-mode model, which was obtained by trial-and-error adjustment of the parameter values.

An obvious feature of these figures is the poor match of the measured response achieved over the first two seconds by the systematic identification technique caused by spurious estimates of the initial velocities and displacements. The identified initial conditions give the best overall match in terms of minimizing the measure-of-fit. However, the estimated values are governed by the strongest portion of the response rather than by the small amplitude vibrations at the beginning of the motion. This is confirmed by Figure 3.22, which shows the response matches achieved by an identification in which the initial velocities and displacements are constrained to zero, resulting in slight changes in the optimal parameter values. The initial portion of the motion is reproduced well, but the strongest motion is reproduced more poorly than by the unconstrained identification. The same problem was encountered by Beck in a time-domain identification.

Wood's model is considered to be one of the best examples of the trial-and-error approach to the identification of building parameters. His model, which assumed zero initial conditions, matched the amplitude of each of the peaks very well. The main source of error in his match was an offset between the measured and model response caused by a slight

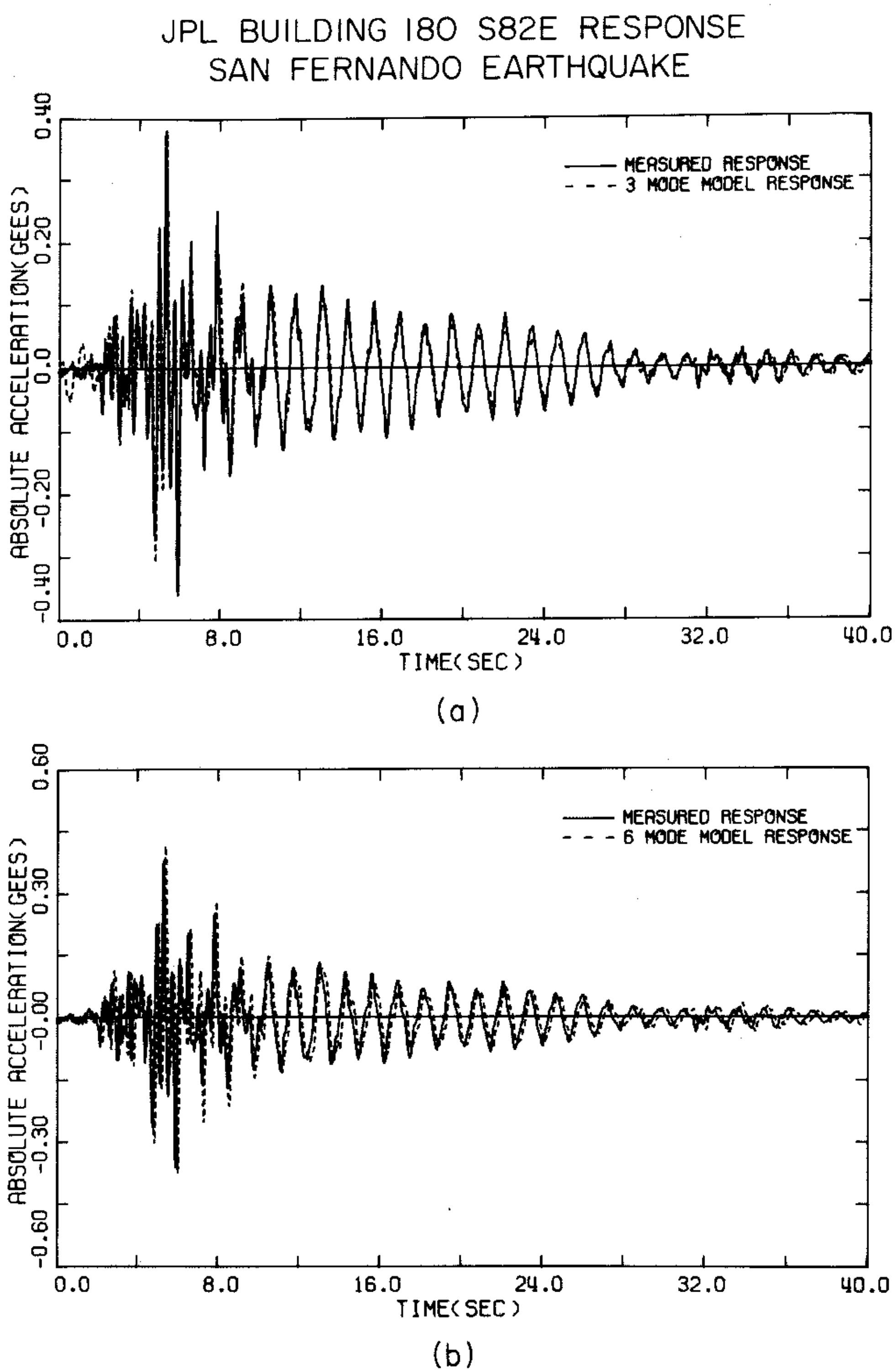


Figure 3.18 JPL Building 180, San Fernando S82E component. Comparison of the measured and model acceleration responses. (a) The optimal three-mode model. (b) Wood's six-mode model.

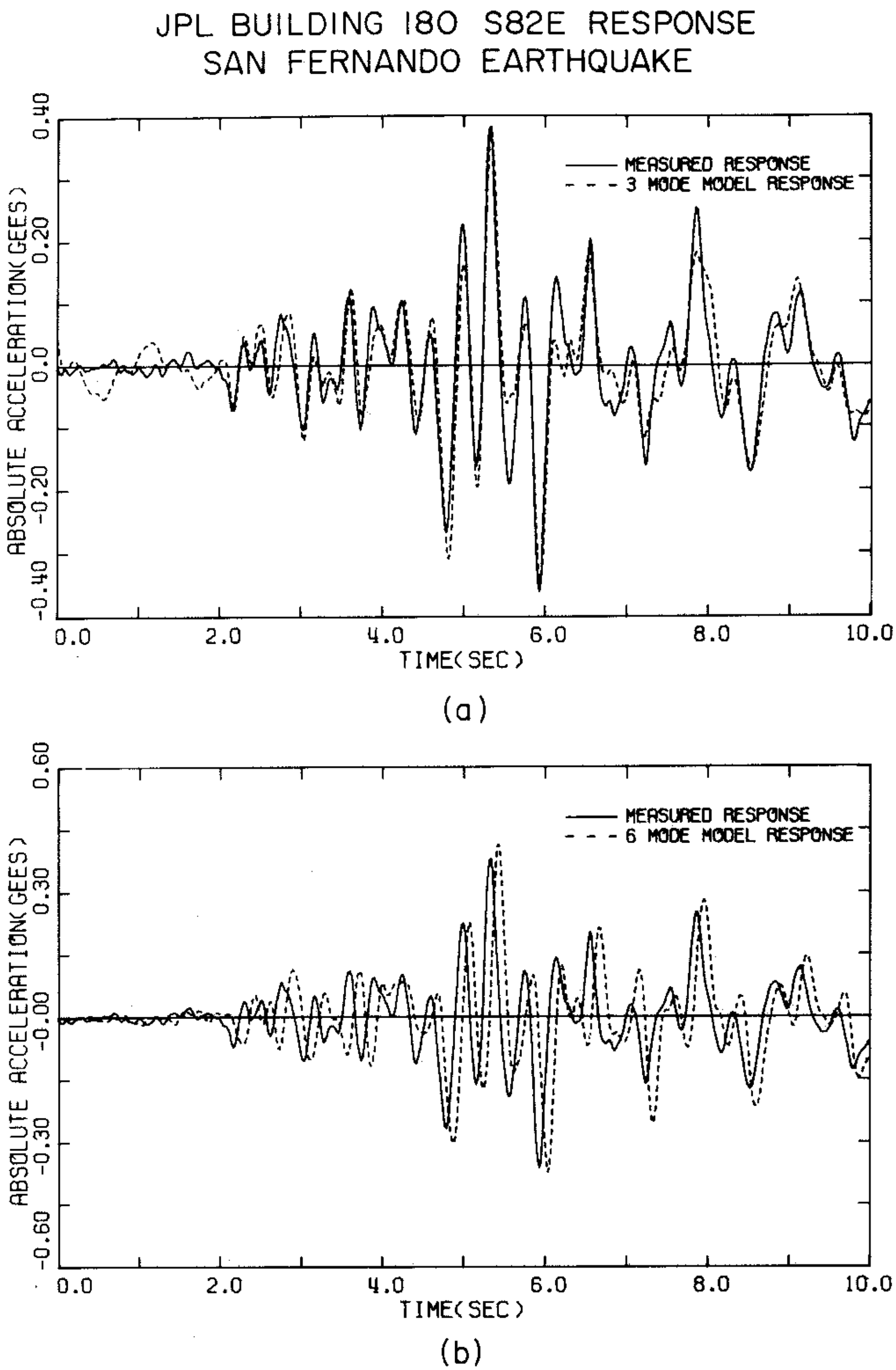


Figure 3.19 JPL Building 180, San Fernando S82E component. The first ten seconds of the acceleration comparisons of Figure 3.18. (a) The optimal three-mode model; (b) Wood's six-mode model.

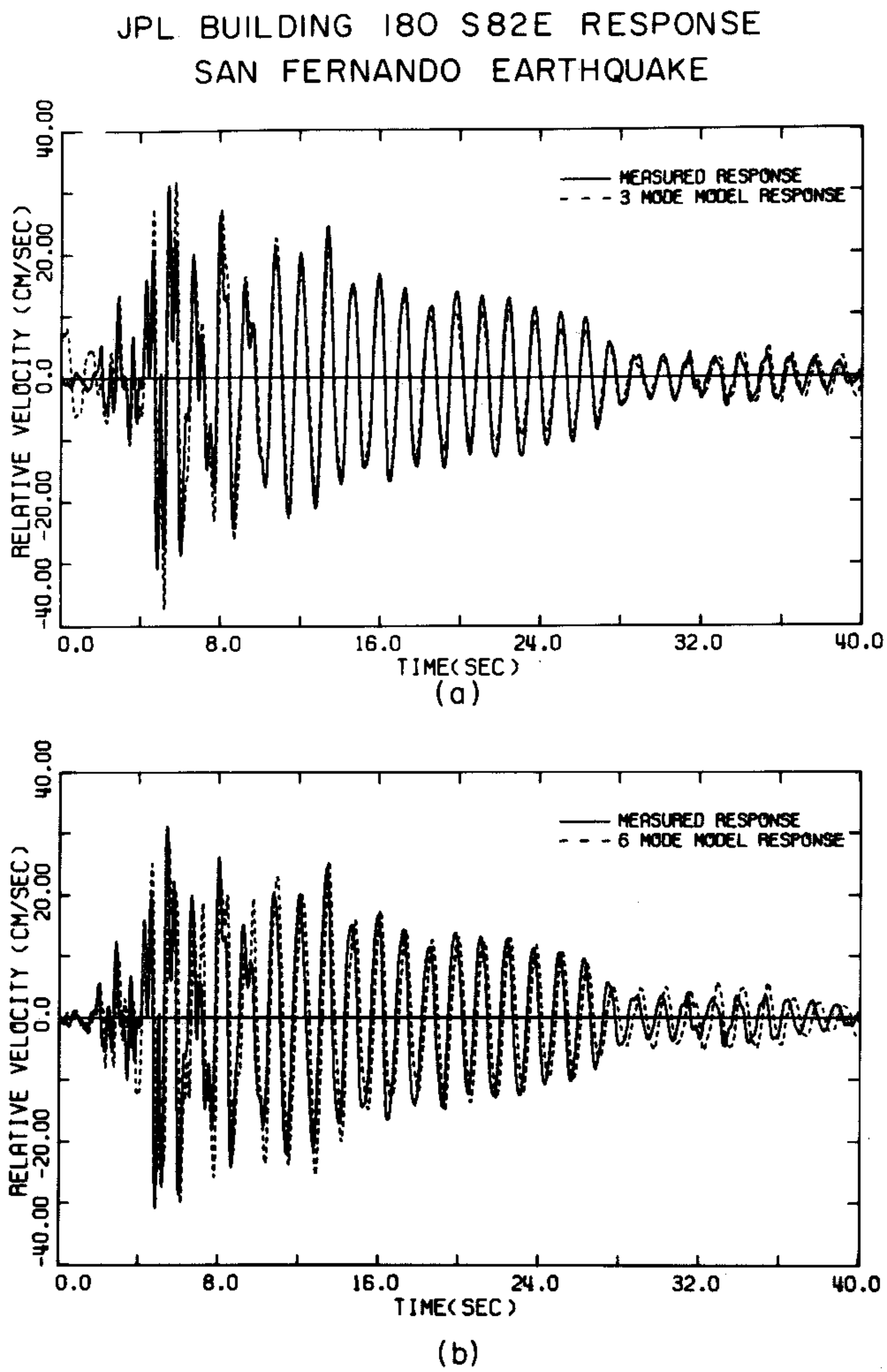


Figure 3.20 Comparison of the measured and model velocity responses of JPL Building 180 to the San Fernando earthquake, S82E component. (a) The optimal three-mode model. (b) Wood's six-mode model.

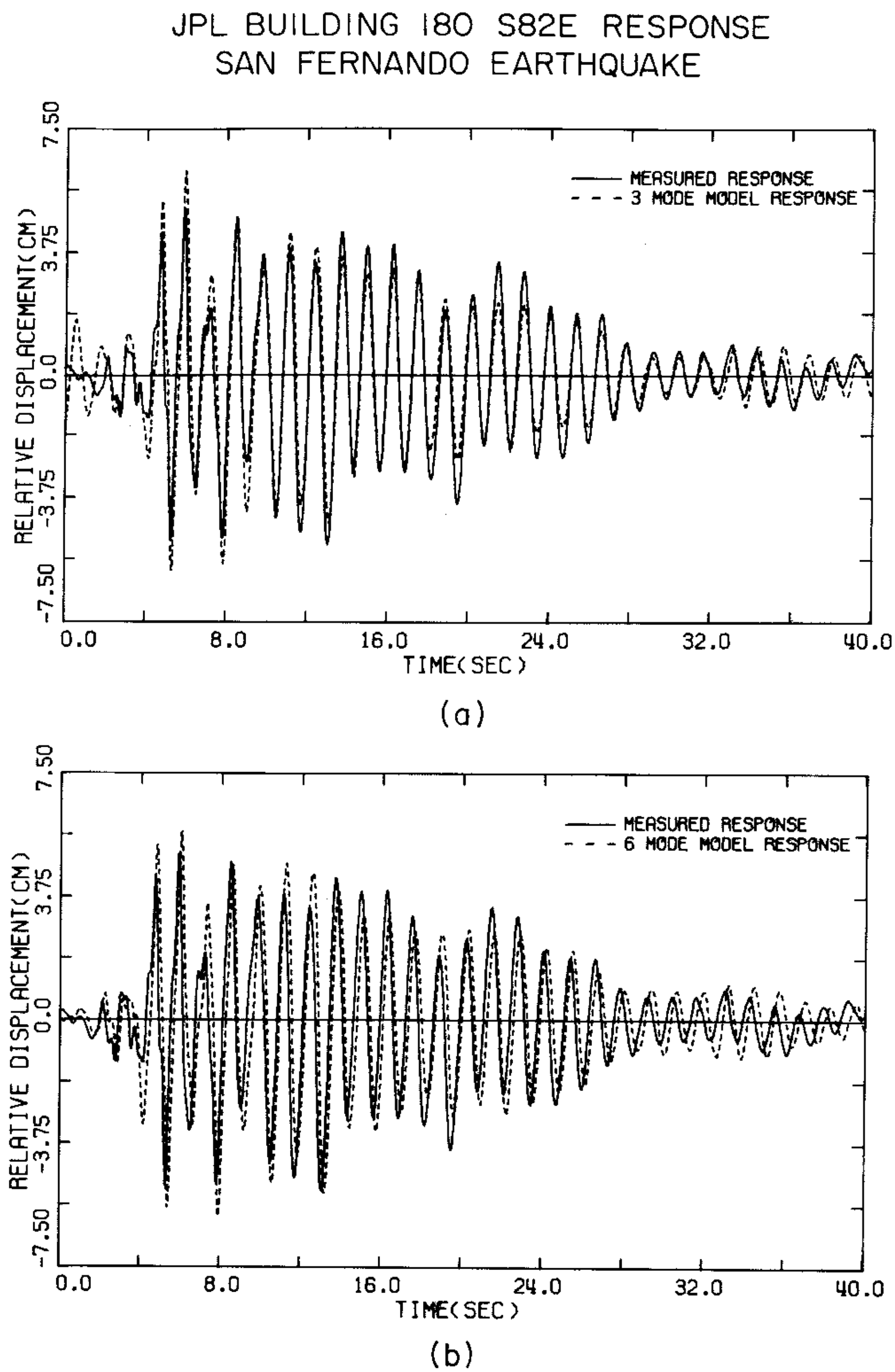


Figure 3.21 Comparison of the measured and model displacement responses of JPL Building 180 to the San Fernando earthquake, S82E component. (a) The optimal three-mode model. (b) Wood's six-mode model.

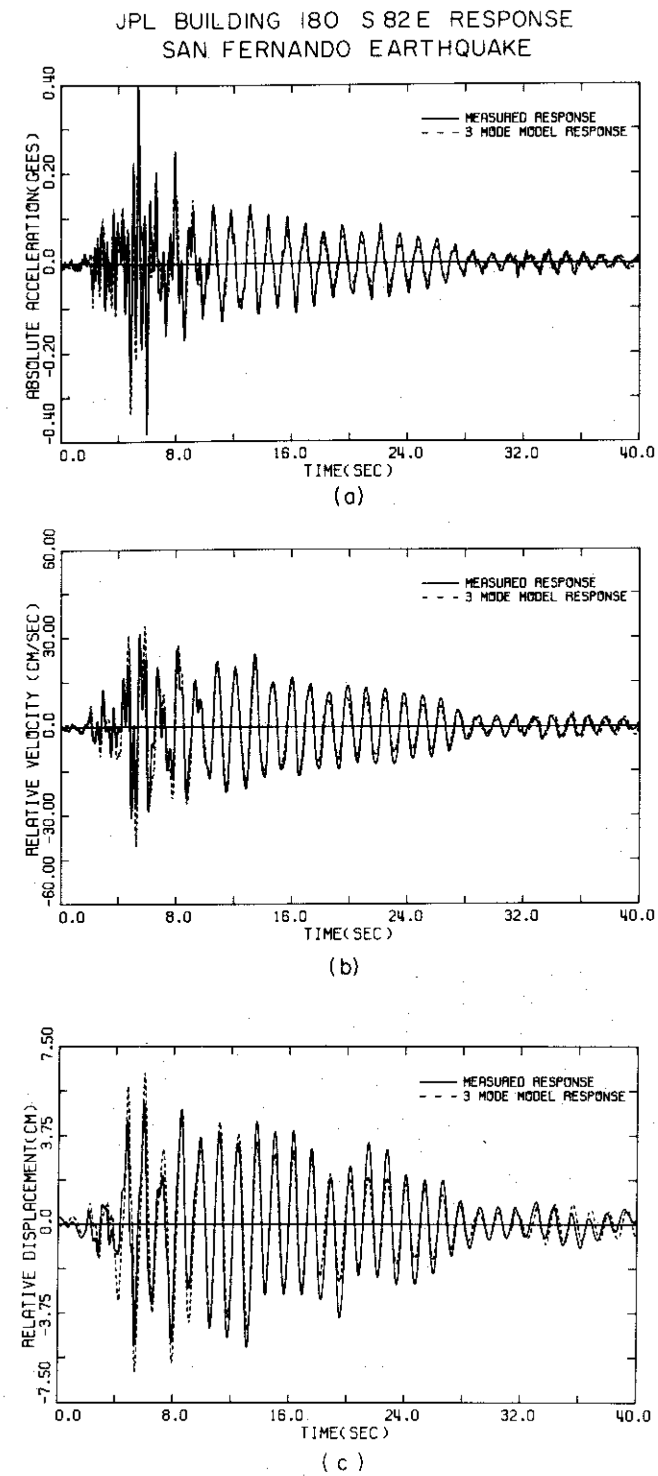


Figure 3.22 JPL Building 180, San Fernando S82E component. The response of the identified three-mode model constrained to zero initial conditions. (a) Acceleration histories. (b) Relative velocity histories. (c) Relative displacement histories.

error in the estimation of the fundamental period. The correct phasing of the responses achieved by the systematic identification technique enhances its margin of superiority when the measured and model responses are superimposed. For practical design purposes the two models are equally good.

The time variation of the effective linear parameters (Fig. 3.16) showed similar behavior to Millikan Library. The fundamental period lengthened from 1.01 seconds, very close to the vibration test values, to a maximum of 1.28 seconds during the largest amplitude response before falling slightly to 1.21 seconds over the last ten seconds of the analyzed record. The identified dampings peaked at 15 and 12 percent for the first two modes about 3½ seconds from the start of the record, before falling to values of 3.5 to 5.0 percent for the first mode and 4.0 to 6.0 percent for the second mode over the last twenty seconds of the record. There appears to be considerable interaction, however, between the estimates of the dampings and participation factors, suggested by the participation factor estimates achieving maximum values at the same time as the dampings. When the first mode participation factor is constrained for all segments to the value of 1.44 identified from the full record, the maximum first mode damping is reduced to 9.7 percent.

One of the features of the Fourier spectrum of the east-west response acceleration is a double peak around the second mode frequency (Fig. 3.14a). The three-mode model identified from the full record produced a second mode period corresponding to the lower amplitude 2.55 Hz (0.39 seconds) peak, despite an initial estimate corresponding to the lower frequency 2.22 Hz (0.45 seconds) peak, which was chosen as the resonant frequency by Wood. Beck's time-domain identification technique produced the same result with Wood's value as the initial estimate.

As observed by Beck, it appears that the double peak is caused by the variation of the second mode period with time. For a series of five second segments of the response, the identified period starts at 0.34 seconds, varies from 0.37 to 0.38 seconds for intervals starting between one and four seconds from the beginning of the record, and jumps to between 0.42 and 0.43 seconds for segments starting between six and eight seconds. It remains at this value over the ten to twenty second interval, before reverting to between 0.37 and 0.38 seconds over the last twenty seconds.

The possibility of horizontal coupling was also considered because of the closeness of the two translational and the torsional second mode periods in the vibration tests. However, the inclusion of two modes, one with a period initially estimated as 0.42 seconds and the other as 0.38 seconds, capable of responding to input from both directions produced final estimates of 0.38 seconds for the periods of both modes, and small participation factors for the north-south component of the input. Similarly, a single second mode capable of responding to both inputs produced a period of 0.38 seconds and a small participation factor for the orthogonal ground component.

The parameter estimates for the third mode are strange. They are mathematically correct, in that they help minimize J, but appear to have no physical meaning. Beck obtained similar results for the same model using the minimization of the time-domain analog of J as the identification criterion. The identified period of 0.30 seconds lies in a trough of the Fourier amplitude spectrum of the response, and does not correspond to a significant peak of the transfer function. The closest peak of the response spectrum is at 3.9 Hz, corresponding to the initial period estimate of 0.26 seconds. The participation factor also has the opposite sign to that expected. It takes a value of -0.41, compared to +0.25 for both a uniform shear beam and Wood's model. The damping is also considerably larger than for the lower modes.

Beck performed time domain matches of the acceleration, velocity and displacement responses for the first twenty seconds of the motion. An unexpected result was that the damping factors and participation factors determined from matching the different response quantities were not in good agreement. This was thought to be caused by the nonlinear and time-varying character of the structure. The parameter estimates from the acceleration histories were similar to those from the acceleration transforms. As noted, Beck encountered the problems with the double second mode peaks, and his analysis also produced the unexpected values of the third mode parameters.

S08W RESPONSE

A time-invariant three-mode model for the north-south (S08W) component of the San Fernando response produced a poorer measure-of-fit and larger error bounds on the estimates than for the east-west model (Table 3.9). This is caused partly by the relatively greater contribution of the higher modes to the response than for the east-west direction, but

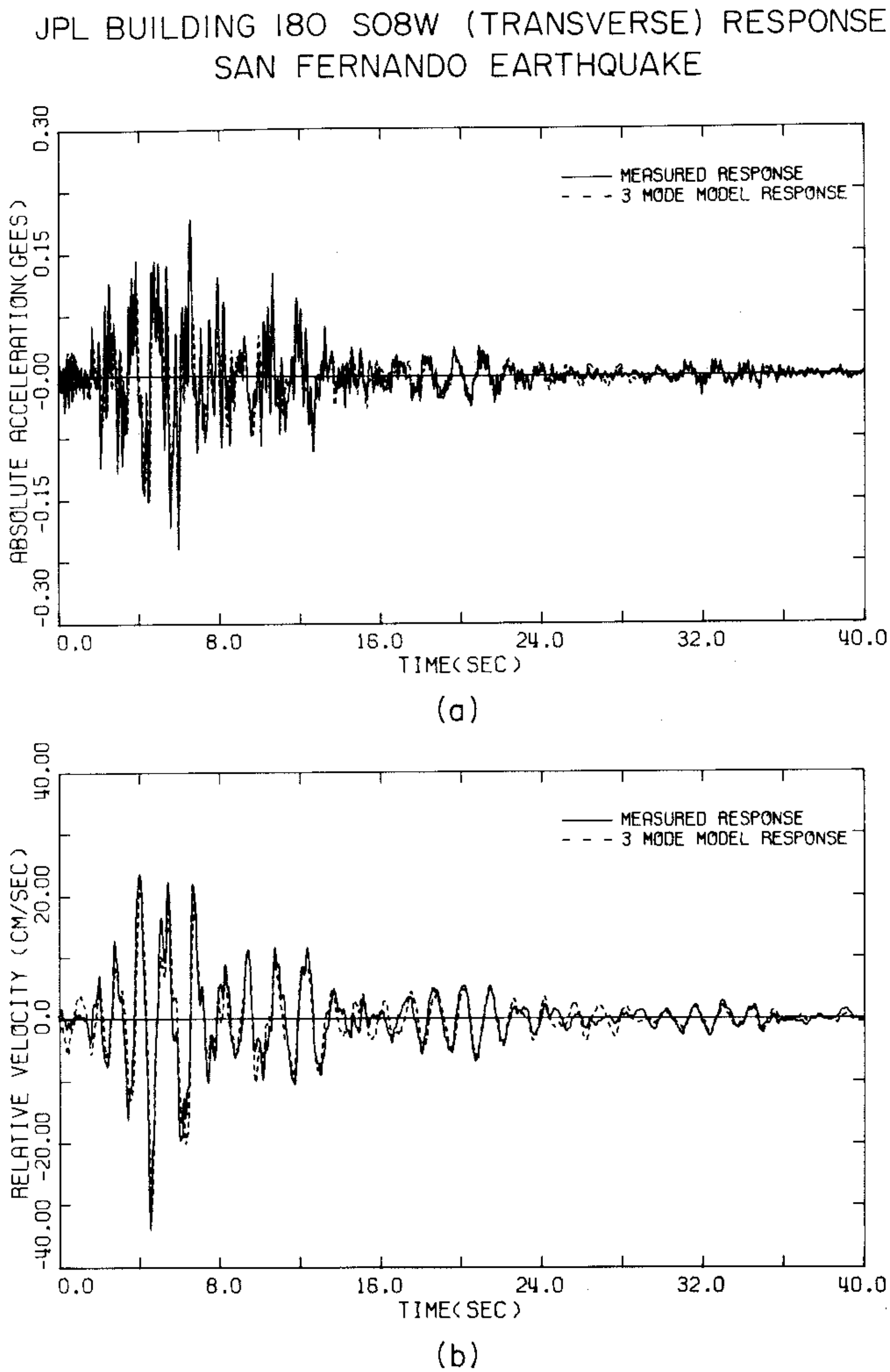


Figure 3.23 Comparison of the measured and optimal three-mode model responses of JPL Building 180 to the San Fernando earthquake, S08W component. (a) Acceleration histories. (b) Relative velocity histories.

it is also an indication of a greater amount of nonlinearity in the response.

The first mode period lengthened by over sixty percent from the pre-earthquake vibration test values, from 0.88 seconds to 1.42 seconds (Fig. 3.17). The second mode period also showed a large increase, from 0.29 to 0.40 seconds. The nonlinearity was also reflected in the damping. The value of 0.064 critical damping identified for the fundamental mode was about twice the value effective in this direction during the Borrego Mountain and Lytle Creek responses, and considerably greater as well than the 0.038 damping in the other direction for the San Fermando response. Unlike the east-west response where the identified first mode damping was similar to Wood's value, it was double his value for the north-south response. The second mode damping value of 0.19 was extremely high, but was associated with a participation factor about double that expected, suggesting yet again interaction of the estimates.

An inspection of the acceleration time history (Fig. 3.23a) reveals much high frequency content, which is not apparent, however, in the velocity response (Fig. 3.23b). The combination of the significant high frequency response and the time-variation of the parameters caused identification difficulties, but the velocity plot shows that the lower frequency content of the response was well-matched.

3.3.5 SUMMARY

The earthquake response of JPL Building 180 was more complicated than that of Millikan Library. This was caused partly by the closer spacing of the modal frequencies which resulted in more modes contributing strongly to the response than in Millikan Library.

Different effective parameter values were found in the three earthquakes. Even for the low amplitude Borrego Mountain and Lytle Creek responses, the effective periods were different from the vibration test values. They were very close, however, to those calculated from a synthesized model assuming no cracking of the concrete enclosing the steel columns.

For the San Fernando response, the fundamental periods in the two directions lengthened by about forty and sixty percent compared to the vibration test values. The second mode periods were lengthened by more than thirty percent. The effective periods were similar to those calculated from a design model which assumed cracking of the concrete in the columns. The response matches obtained in the systematic identification technique were considerably better than those achieved by a trial-and-error modification of the modal periods and damping of the synthesized model because of a better matching of the phase of the measured response. The effective dampings for the overall response were 4 and 6½ percent for the fundamental modes in the longitudinal and transverse directions, with values of about 10 percent during the maximum response.

The variation of the parameters with time was similar to that of Millikan Library. Analysis of short response segments at the beginning of the record showed that initially the effective periods equalled the vibration test values, but increased rapidly until the time of maximum response, and then stayed almost constant.

Several difficulties were encountered in the identification of the models appropriate for the San Fernando responses. Investigations of several possibilities led to the conclusion that the most probable cause of a double peak in the response spectrum around the second mode frequency for the S82E component was time variation of the modal period, and not torsional response of the structure. The estimates of all the third mode parameters agreed with the values obtained from Beck's time domain analysis, but are thought to be physically unreasonable because of the effects of noise and model error. There was interaction between the estimates of the dampings and participation factors, and also, rather unexpectedly, between the estimates of the periods and participation factors. The identification of the parameters appropriate for the S08W response was complicated by the strong contribution of several modes, each with time-varying properties. The acceleration match for this response component was poorer than for the other records, but the estimates of the parameters of the lower modes were sufficiently accurate to produce a good match of the measured velocity for which the high frequency content was unimportant.

REFERENCES

1. Beck, J.L., "Determining Models of Structures from Earthquake Records", Earthquake Engineering Research Laboratory Report EERL 78-01, California Institute of Technology, Pasadena, California, 1971.

2. Brandow and Johnston Associates, "Design Analysis of the Jet Propulsion Laboratory, Building 180", prepared for the Jet Propulsion Laboratory, California Institute of Technology, Pasadena, California, 1971.

3. Foutch, D.A. and G.W. Housner, "Observed Changes in the Natural Periods of Vibration of a Nine Story Steel Frame Building", Proc. 6th World Conf. Earthquake Eng., III, 2698-2704, New Delhi, India, January 1977.

4. Foutch, D.A., G.W. Housner and P.C. Jennings, "Dynamic Responses of Six Multistory Buildings during the San Fernando Earthquake", Earthquake Engineering Research Laboratory Report EERL 75-02, California Institute of Technology, Pasadena, California, October 1975.

5. Foutch, D.A. and P.C. Jennings, "A Study of the Apparent Change in the Foundation Response of a Nine-Story Reinforced Concrete Building ",Bulletin Seism. Soc. Am., 68, 1, 219-29, February 1978.

6. Hanks, T.C., "Current Assessment of Long-Period Errors", in 'Strong Motion Earthquake Accelerograms. Digitized and Plotted Data. Volume II, Part G', EERL 73-52, California Institute of Technology, Pasadena, California, November 1973.

7. Iemura, H. and P.C. Jennings, "Hysteretic Response of a Nine-Story Reinforced Concrete Building", International Journal Earthquake Eng., and Structural Dynamics, 3, 183-201, 1974.

8. Jennings, P.C. and J.H. Kuroiwa, "Vibration and Soil Structure Interaction Test of a Nine-Story Reinforced Concrete Building", Bulletin Seism. Soc. Am., 58, 3, 891-916, June 1978.

9. Jerath, N.E. and F.E. Udwadia, "Time Variations of Structural Properties during Strong Ground Shaking", submitted for publication in International Journal of Earthquake Eng. and Structural Dynamics.

10. Kuroiwa, J.H., "Vibration Test of a Multistory Building", Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, 1967.

11. Nielsen, N.N., "Dynamic Response of Multistory Buildings", Earthquake Research Laboratory, California Institute of Technology, Pasadena, California, June 1964.

12. Trifunac, M.D., F.E. Udwadia and A.G. Brady, "Analysis of Errors in Digitized Strong-Motion Accelerograms", Bulletin Seism. Soc. Am., 63, 157-87, 1973.

13. Udwadia, F.E. and P.Z. Marmarelis, "The Identification of Building Structural Systems. I. The Linear Case. II. The Nonlinear Case", Bulletin Seism. Soc. Am., 66, 1, 121-71, February 1976.

14. Udwadia, F.E. and M.D. Trifunac, "Time and Amplitude Dependent Response of Structures", International Journal of Earthquake Eng. and Structural Dynamics, 2, 359-78, 1974.

15. Wood, J.H., "Analysis of the Earthquake Response of a Nine-Story Steel Frame Building during the San Fernando Earthquake", Earthquake Eng. Research Laboratory, Report EERL 72-04, California Institute of Technology, Pasadena, California 1971.

16. Wood, J.H., "Earthquake Response of a Steel Frame Building", International Journal of Earthquake Eng. and Structural Dynamics, 4, 4, 349-77, 1976.

IV. DAMAGED STRUCTURES: THE HOLIDAY INNS AND BANK OF CALIFORNIA BUILDING

This chapter is concerned with the three most heavily damaged of the instrumented buildings during the San Fernando earthquake. The two Holiday Inns, located at 8244 Orion Avenue and 1640 South Marengo Street, eight and twenty-one miles, respectively, south of the center of energy release of the San Fernando earthquake at Pacoima Dam (Fig. 1.1), are virtually identical structures which were subjected to different intensities of ground shaking and suffered different amounts of damage [2,3,5]. The Bank of California building at 15250 Ventura Boulevard, fourteen miles from Pacoima Dam (Fig. 1.1), also suffered considerable damage, with about a quarter of the cost of repairs spent on structural members [1,5].

The damage in these structures was associated with more nonlinear behavior in their responses than for any of the other buildings studied. These nonlinearities, including period lengthenings of over 100 percent, provided a severe test for the identification of equivalent linear models. It was difficult to reproduce some portions of the response even with the optimal models identified from segment-by-segment analysis.

4.1 THE HOLIDAY INNS

4.1.1 INTRODUCTION

The Holiday Inn buildings are 65 feet high, seven story, reinforced concrete frame structures measuring 61 feet by 150 feet in plan. The two buildings were designed and constructed by the same architects and engineers in 1965-6. Detailed descriptions and synthesized models of the structures are given by Blume and Associates [2,3]. The following summary is based on these reports.

The Orion Avenue building was situated less than half as far as the Marengo Street structure from the center of energy release of the San Fernando earthquake and thus received generally stronger shaking, although the Marengo Street building experienced a larger peak acceleration response at the roof in the transverse direction than the Orion Avenue building. Although the maximum ground acceleration in the longitudinal direction at the two sites was about the same, the Fourier spectra showed that the Orion Avenue excitation was in fact considerably stronger at most frequencies. The maximum ground and response accelerations for the two buildings in both directions are listed in Table 4.1. The accelerograms are shown in reference [5] and in the Caltech Data Reports [4].

The natural periods of the buildings determined in ambient vibration tests are given in Table 4.2. Prior to the earthquake, the periods of the two buildings, about 0.5 seconds in both directions, were very similar, as would be expected for virtually identical structures. Further tests soon after the earthquake revealed that the small amplitude periods had lengthened considerably, by about forty percent for the Orion Avenue building and twenty-five percent for the Marengo Street building. The effective periods of the Orion Avenue structure during its response to an aftershock seven weeks after the main shock were over twice the initial ambient vibration test values. After repairs to the

TABLE 4.1: MAXIMUM ACCELERATIONS FOR THE HOLIDAY INN BUILDINGS

Component	8244 Orion Avenue	1640 South Marengo Street
Transverse	NS	S52W
1st Floor	250 cm/sec^2	130 cm/sec^2
4th Floor	195 cm/sec^2	240 cm/sec^2
8th Floor	375 cm/sec^2	412 cm/sec^2
Longitudinal	EW	N38W
1st Floor	132 cm/sec^2	118 cm/sec^2
4th Floor	232 cm/sec^2	187 cm/sec^2
8th Floor	314 cm/sec^2	230 cm/sec^2

TABLE 4.2: VIBRATION TEST PERIODS FOR THE HOLIDAY INN BUILDINGS

	8244 Orion Avenue		1640 S. Marengo Street	
	Transverse (NS)	Longitudinal (EW)	Transverse (S52W)	Longitudinal (N38W)
Pre-Earthquake	0.48 sec	0.52 sec	0.49 sec	0.53 sec
Post-Earthquake (Feb-Mar 1971)	0.68 sec	0.72 sec	0.63 sec	0.64 sec
Aftershock March 31, 1971	1.1 sec	1.2 sec	-	-
After Repairs May, 1971	0.58 sec	0.64 sec	-	-

structure, the fundamental translational periods in ambient vibrations were midway between the pre- and post-earthquake test values.

The periods effective during the earthquake response, as indicated by the Fourier response spectra, were about three times the ambient periods for the Orion Avenue building and over twice the vibration test values for the Marengo Street structure. Such large period lengthenings reflect significantly nonlinear response. The reports by Blume and Associates indicated that the approximate ductility factors, given by the ratio of the maximum relative displacement of the roof to its relative displacement at the nominal elastic limit, were eight and four for the Orion and Marengo structures, respectively.

The repair costs for the Orion Avenue and Marengo Street buildings were $145,000 and $95,000 respectively, about eleven and seven percent of the original construction costs. These costs corresponded to $2.30 and $1.50 per square foot of floor space. Almost all the expense was for nonstructural repairs, with damage to structural elements amounting to only about $2000 for each structure. The type of damage in the two buildings was similar, but more severe for the Orion Avenue structure.

The structural repair at Orion Avenue involved epoxy repair of spalled concrete at a second floor beam-column joint. At Marengo Street, structural repair was required at a stair landing between the first and second floors. Typical nonstructural damage, which occurred in almost every room in both buildings, required replacement or repair of wall partitions, bathroom tiles and plumbing fixtures.

Acceleration records were obtained for both structures during the San Fernando earthquake at the first floor, fourth floor and roof levels.

Frequency domain identifications over a series of time segments have been performed for all records.

4.1.2 ORION AVENUE

NORTH-SOUTH (TRANSVERSE) COMPONENTS

Plots of the parameter values estimated from different segments of the Orion Avenue north-south roof records show a great increase in the fundamental period during the course of the response and high damping, particularly during the strongest motion (Fig. 4.1). The period in the transverse north-south direction increased from the vibration test value of 0.48 seconds to a maximum identified value of 1.61 seconds, with an effective value of 1.43 seconds for the full 40.96 seconds of analyzed record. The effective overall first mode damping was identified as about seventeen percent. The estimate of this value is dominated by the portion of maximum response, and may also be inflated by the broadening of the modal bandwidth caused by the changes in period. Later in the response, the damping fell to half the overall value. The roof participation factor was identified as 1.15, considerably different from the value of 1.31 for the synthesized model. Such a difference raises the possibility of a significant change in the mode shape, with relatively greater modal displacements in the lower floors compared to the design model. The value identified for the fourth floor participation factor is consistent with such a change in the mode shape, giving a fourth floor modal displacement of 0.75 of the roof modal displacement compared to 0.54 for the synthesized model. This type of behavior could be produced by more severe stiffness degradation near the base than in the higher floors.

The markedly nonlinear behavior associated with the structural and nonstructural damage to the building was revealed during the identification in the extreme lengthening of the fundamental period, the high damping and the changed participation factors. The strongly nonlinear behavior made some segments of the response of the structure difficult to approximate with linear models, as reflected in the large normalized errors of around 0.5 for short segments early in the response and for time-invariant models for the entire response. Later in the record, when the parameter values became nearly constant, the matches were much better, with errors of 0.1 or less.

The availability of response records from the fourth floor as well as the roof allowed the reliability of the parameter estimates to be assessed by comparing the values derived from the two sets of data. For the north-south component, the fundamental period estimated from the forty second segment of the fourth floor response was 1.34 seconds, compared with 1.43 seconds from the roof record. The discrepancy in the period estimates for this structure seem likely to be a consequence of the nonlinear behavior of the building. For the second twenty seconds of the record for which the identified models provide a much better match of the response, the estimates were much more consistent, 1.59 seconds from the fourth floor and 1.61 seconds from the roof record.

The damping estimates were also inconsistent, 27 percent of critical from the fourth floor and 16½ percent from the roof for the full record, and 12½ percent versus 8½ percent for the 20-40 second segment. It was found that the damping estimates from the midheight and roof records became more consistent if they were scaled by the ratio of the synthesized model participation factor to the identified participation factor, a procedure suggested by the known tendency of these parameters to couple in the identification process. The damping estimates adjusted in this manner were 22 and 19½ percent from the full forty second records for the fourth floor and roof respectively, and 10½ versus 9¼ percent for the second half of the records. Similar results were obtained by setting the participation factors equal to the values of the synthesized model and performing the identifications only with respect to the other parameters. These constrained identifications produced a damping of 22 percent from the full fourth floor record, while changing the normalized error only slightly from 0.374 to 0.391, and a damping of 20 percent from the full roof record while altering the measure-of-fit from 0.411 to 0.416.

In an attempt to overcome the coupling of the damping and participation factor estimates, the parameters were estimated simultaneously from equal weightings of the two records (midheight and roof). This approach has been discussed in section 2.7.

The participation factors estimated in the simultaneous identifications from the two records were closer to the values for the synthesized model than were those from the individual identifications. For example, the forty second segments gave fourth floor and roof participation factors of 0.73 and 1.23, closer to the synthesized values of 0.70 and 1.31 than were the values of 0.86 and 1.14 for the separate

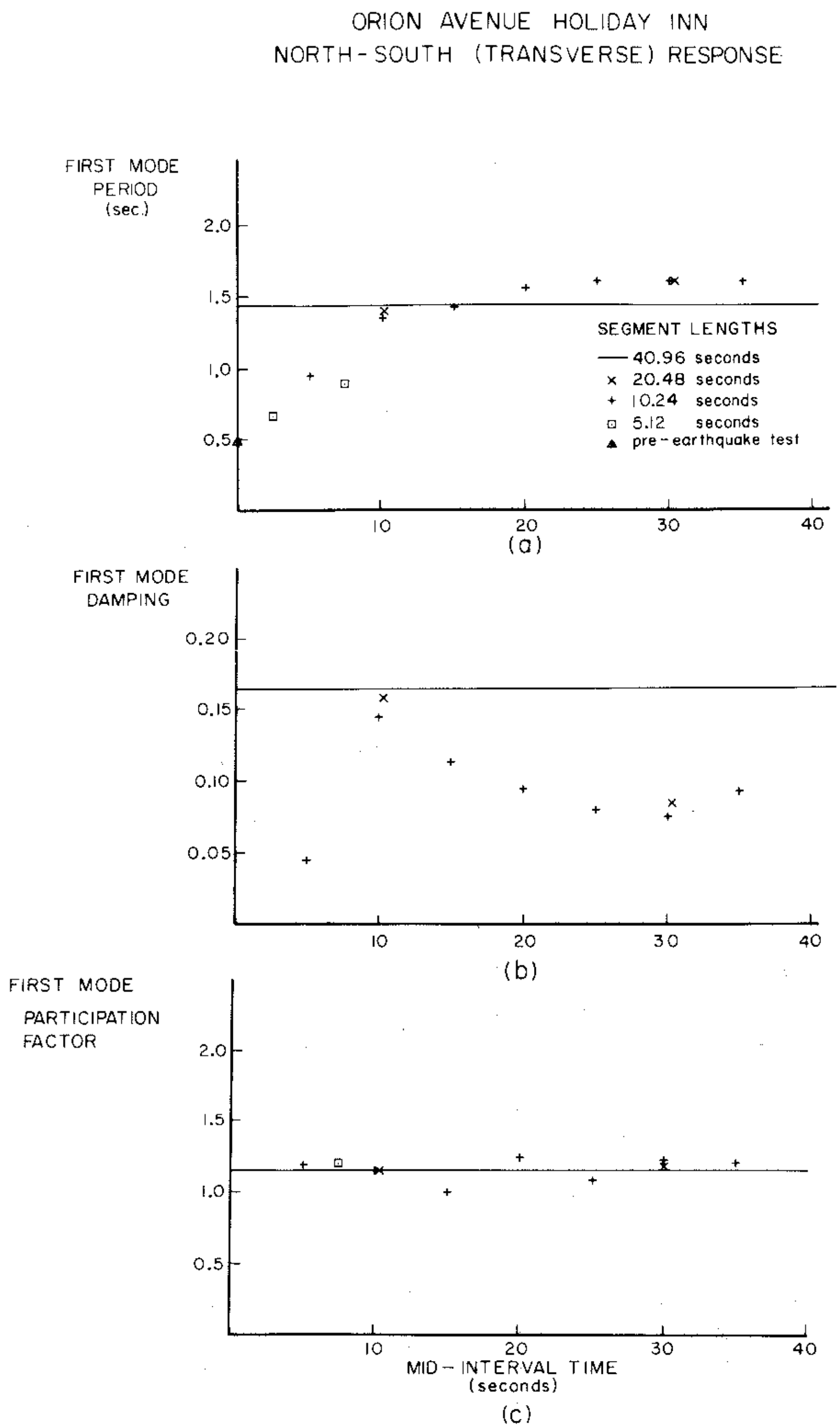


Figure 4.1 Time variation of the parameters of linear models of the Orion Avenue Holiday Inn identified from the north-south (transverse) roof response.

TABLE 4.3: ESTIMATES OF THE PARAMETERS FROM THE ORION AVENUE NORTH-SOUTH RECORDS

(a) 0 - 40.96 Seconds Record

Estimation Method	Parameter						
	T_1(sec)	ζ_1	C_1 4th	C_1 Roof	T_2(sec)	ζ_2	C_2 Roof
Unconstrained identification							
4th floor	1.34	0.269	0.86	-	-	-	-
Roof	1.43	0.165	-	1.14	0.30	0.26	-0.48
Scaled dampings							
4th floor	1.34	0.219	0.70	-	-	-	-
Roof	1.43	0.190	-	1.31	-	-	-
Constrained identification							
4th floor	1.36	0.219	0.70	-	-	-	-
Roof	1.42	0.199	-	1.31	0.32	0.27	-0.47
Simultaneous identification	1.42	0.192	0.73	1.23	-	-	-

(b) 20 - 40.48 Seconds Record

Estimation Method	Parameter			
	T_1(sec)	ζ_1	C_1 4th	C_1 Roof
Unconstrained identification				
4th floor	1.59	0.126	0.84	-
Roof	1.61	0.084	-	1.19
Scaled dampings				
4th floor	1.59	0.105	0.70	-
Roof	1.61	0.092	-	1.31
Constrained identification				
4th floor	1.58	0.107	0.70	-
Roof	1.62	0.091	-	1.31
Simultaneous identification	1.61	0.090	0.68	1.23

TABLE 4.4: ACCURACY ANALYSES FOR THE ORION AVENUE N-S IDENTIFICATIONS

(a) 0 - 40.96 Second Record

Normalized Partial Hessian Matrix

Roof Record

Parameters	a_1	b_1	c_1 Roof
a_1	1.623	-0.0001	-1.623
b_1			0.179
c_1 Roof			4.295

4th Floor Record

Parameters	a_1	b_1	c_1 4th
a_1	0.964	0.0000	-0.961
b_1		4.973	0.017
c_1 4th			2.714

Simultaneous Identification

Parameters	a_1	b_1	c_1 4th	c_1 Roof
a_1	1.334	-0.0000	-0.350	-0.984
b_1		8.985	0.026	0.073
c_1 4th			0.876	0
c_1 Roof				2.464

Interaction Coefficients

Roof Record

Parameters	a_1	b_1	c_1 Roof
a_1	1	0.0000	0.615
b_1		1	0.027
c_1 Roof			1

4th Floor Record

Parameters	a_1	b_1	c_1 4th
a_1	1	0.0000	0.595
b_1		1	0.005
c_1 4th			1

Simultaneous Identification

Parameters	a_1	b_1	c_1 4th	c_1 Roof
a_1	1	0.0000	0.324	0.543
b_1		1	0.009	0.016
c_1 4th			1	0
c_1 Roof				1

TABLE 4.4: ACCURACY ANALYSES FOR THE ORION AVENUE N-S IDENTIFICATIONS

(Cont'd)

(b) 20 - 40.48 Second Record

Roof Record

Parameters	a_1	b_1	c_1 Roof
a_1	10.23	-2.93	-6.78
b_1		302.5	23.64
c_1 Roof			15.46

Parameters	a_1	b_1	c_1 Roof
a_1	1	0.053	0.539
b_1		1	0.346
c_1 Roof			1

4th Floor Record

Parameters	a_1	b_1	c_1 4th
a_1	8.92	-2.74	-5.94
b_1		192.0	13.60
c_1 4th			10.21

Parameters	a_1	b_1	c_1 4th
a_1	1	0.066	0.622
b_1		1	0.307
c_1 4th			1

Simultaneous Identification

Parameters	a_1	b_1	c_1 4th	c_1 Roof
a_1	9.33	-2.88	-1.52	-4.56
b_1		280.2	4.96	16.62
c_1 4th			2.91	0
c_1 Roof				9.70

Parameters	a_1	b_1	c_1 4th	c_1 Roof
a_1	1	0.056	0.291	0.479
b_1		1	0.174	0.319
c_1 4th			1	0
c_1 Roof				1

identifications. The first mode dampings estimated from the two segments were 19 and 9 percent, close to the "adjusted values" obtained previously. The estimates of the parameters obtained by the various strategies are summarized in Table 4.3.

The simultaneous identification from records at two locations has the inherent advantage over two separate identifications that the constraint of the records being from the same structure is automatically included. This property should help reduce the coupling between the estimates of the dampings and participation factors. The amount of coupling is determined by the normalized partial Hessian matrix and the interaction coefficients. Table 4.4 lists these matrices for the two segments of record. The simultaneous identification reduced the size of the interactions, most notably by a factor of about two between the fourth floor participation factor and the damping parameter, $a_1 = 2\zeta_1\omega_1$, of the first mode. Thus, as expected, the simultaneous identification succeeded in reducing the interaction effect and produced more reliable estimates of the parameters than those from the individual identifications.

EAST-WEST (LONGITUDINAL) COMPONENTS

The behavior of the Orion Avenue structure in the longitudinal east-west direction was similar to that in the north-south direction (Fig. 4.2). Again a large period increase was traced, from the vibration test value of 0.52 seconds to the maximum identified value of 1.35 seconds. The average effective damping was identified from the roof record as 18 percent. However, the values estimated from shorter segments of the response were more typically near 10 percent. As in the other direction, the dampings found from the entire record may have been overestimated to broaden the resonant peaks of the time-invariant model to match the broad peaks of the measured response caused in this case by the variation of the periods of the actual system.

Unlike the north-south component, the identified value of the first mode participation factor at the roof showed an increase from the design model value, from 1.28 to 1.37. This increase may have partially compensated for the effects of an overestimation of the damping. Consequently, it seems that the estimated participation factors may not have much meaning when there are substantial nonlinearities present in the response as for this building.

In the east-west direction, the period estimates from the fourth floor and roof records were in better agreement than for the north-south direction. For example, the identified fundamental periods from the full records were 1.18 seconds and 1.21 seconds from the fourth floor and roof records respectively.

The dampings were again inconsistent, however, with estimates of 13 and 18 percent for the overall value. The participation factor estimates of 0.43 and 1.37 differed considerably from the synthesized model values of 0.75 and 1.28. Scaling the damping estimates by the ratios of the synthesized to the identified participation factors produced values of 22 and 17 percent, while performing identifications with the participation factors constrained to the values for the synthesized model produced dampings of 24 and 17 percent. Thus, unlike for the

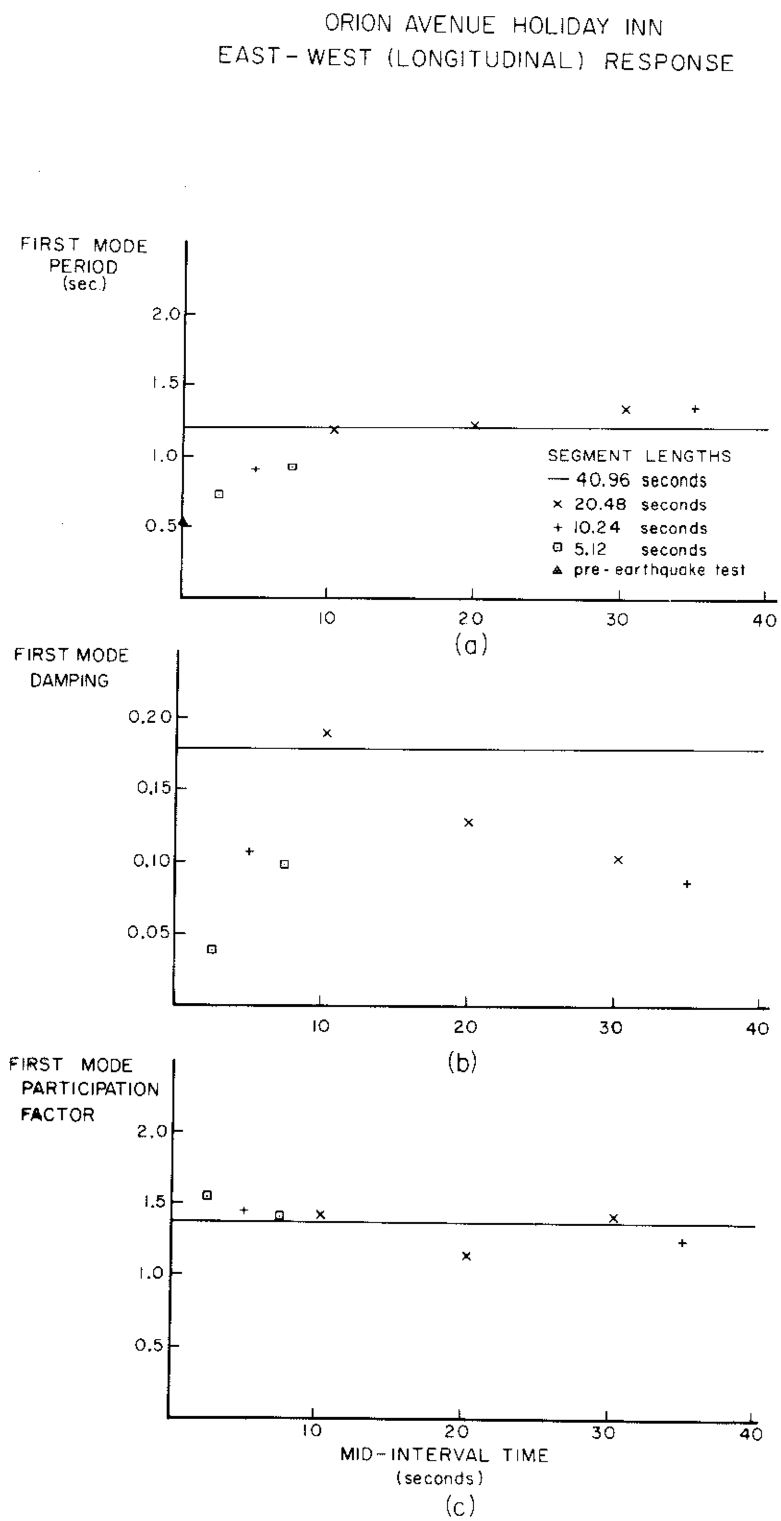


Figure 4.2 Time variation of the parameters of linear models of the Orion Avenue Holiday Inn identified from the east-west (longitudinal) roof response.

TABLE 4.5: ESTIMATES OF THE PARAMETERS FROM THE ORION AVENUE E-W RECORDS

(a) 0 - 40.96 Seconds Record

Estimation Method	Parameter			
	T_1 (sec)	ζ_1	C_1 4th	C_1 Roof
Unconstrained identification				
4th floor	1.18	0.128	0.43	-
Roof	1.21	0.179	-	1.37
Scaled dampings				
4th floor	1.18	0.224	0.75	
Roof	1.21	0.167	-	1.28
Constrained identification				
4th floor	1.14	0.244	0.750	-
Roof	1.21	0.167	-	1.28
Simultaneous identification	1.20	0.173	0.47	1.36

(b) 20 - 40.48 Seconds Record

Estimation Method	Parameter			
	T_1 (sec)	ζ_1	C_1 4th	C_1 Roof
Unconstrained identification				
4th floor	1.28	0.117	0.81	-
Roof	1.33	0.104	-	1.43
Scaled dampings				
4th floor	1.28	0.108	0.75	-
Roof	1.33	0.093	-	1.28
Simultaneous identification	1.32	0.107	0.79	1.43

north-south direction, using the participation factors of the synthesized model did not resolve the inconsistencies in the estimates of the damping.

This result was not very promising for the use of the simultaneous identification technique. The simultaneous identifications produced period and damping estimates close to those from the roof records, and participation factors little changed from the estimates from the individual records. The fourth floor value changed only from 0.43 to 0.47, still considerably smaller than the synthesized model value of 0.75. The participation factors identified for this direction were not necessarily indicative of the mode shape, but rather were the result of the considerable nonlinearities in the response.

The damping estimates from the two records were much more consistent over the last twenty seconds of the response, and the fourth floor participation factor much more closely approximated the synthesized model value. The simultaneous identification from the two records altered the estimates only slightly.

The estimates of the parameters obtained by the various methods for the two segments of the east-west records are summarized in Table 4.5.

4.1.3 MARENGO STREET

The variations with time of the fundamental periods for the less severely damaged Marengo Street building are not so obvious as for the Orion Avenue structure (Figs. 4.3 and 4.4). This is largely because attempted identifications with five second time windows at the beginning of the record were unsuccessful, thus preventing a tracing of the period variation in the initial portion of the response. Nevertheless, the increase in period from the vibration test values was considerable, from 0.53 seconds to 1.07 seconds in the longitudinal N38W direction, and from 0.49 seconds to 1.17 seconds in the transverse S52W component.

The fundamental mode damping in the N38W direction as determined for the entire record was around 18 percent, 17.7 percent from the roof record and 18.4 percent from the fourth floor. This agreement between the two records was much better than for the Orion Avenue identifications, and was typical of the agreement achieved for the different time segments. For the second half of the record, the identified damping fell to about 8½ percent. The first mode participation factors for the full record were 0.91 and 1.55 for the midheight and roof records compared to 0.75 and 1.28 for the design models. It was also interesting that the measures-of-fit for the two records were virtually identical for the same time segments, for example 0.440 and 0.436 for the full record and 0.054 and 0.053 for the 20 to 40 second segment. With the close agreement between the parameter estimates from the response records at the two locations, the simultaneous identifications using both records produced parameter values similar to those from the individual records. The results are listed in Table 4.6.

The damping and participation factor estimates found for the transverse S52W direction (Table 4.7) behaved unrealistically. The agreements between the estimates from the fourth floor and roof records for the same segments were good, but some of the values seemed highly improbable. For example, both full records gave five percent for the

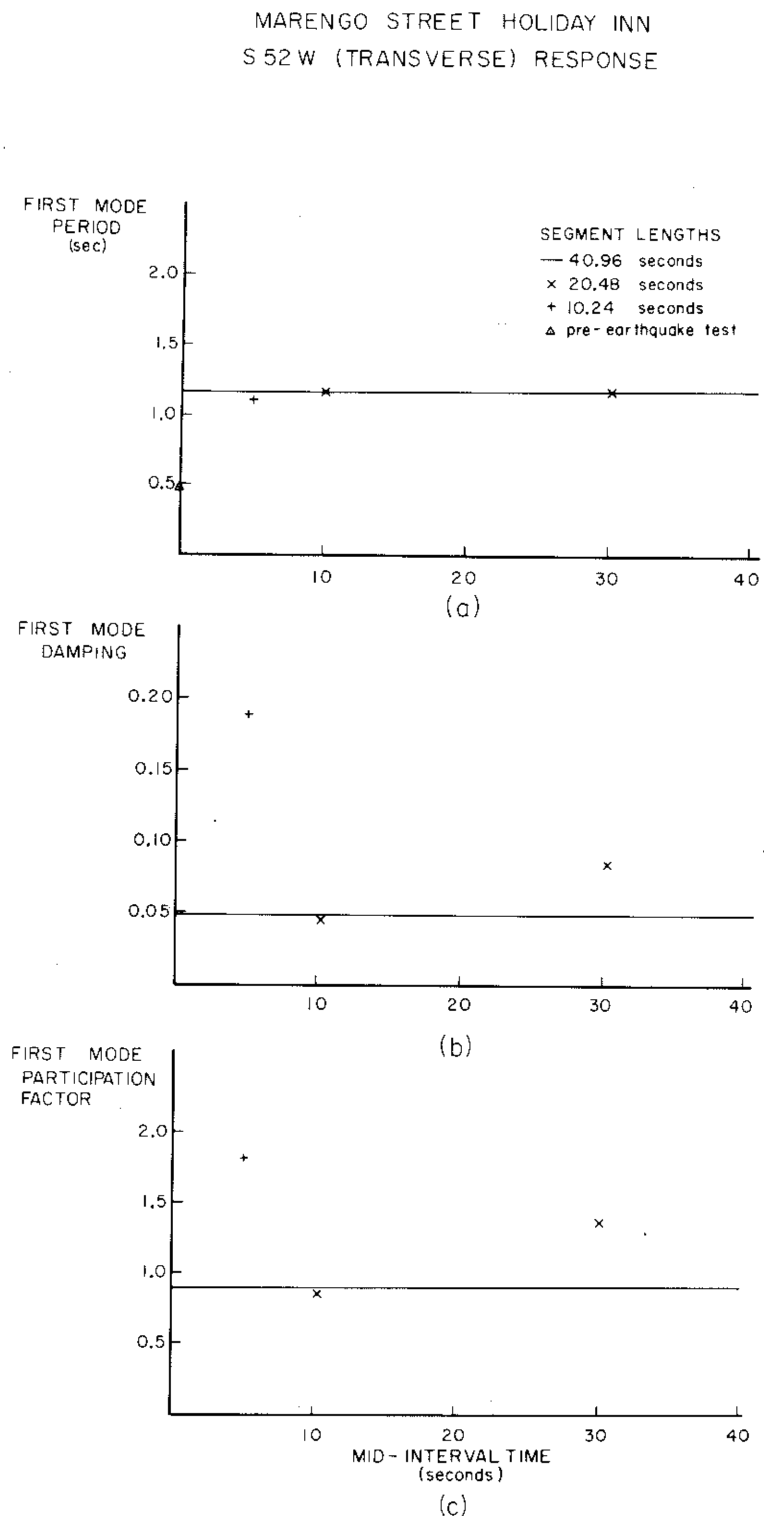


Figure 4.3 Time variation of the parameters of linear models of the Marengo Street Holiday Inn identified from the S52W (transverse) roof response.

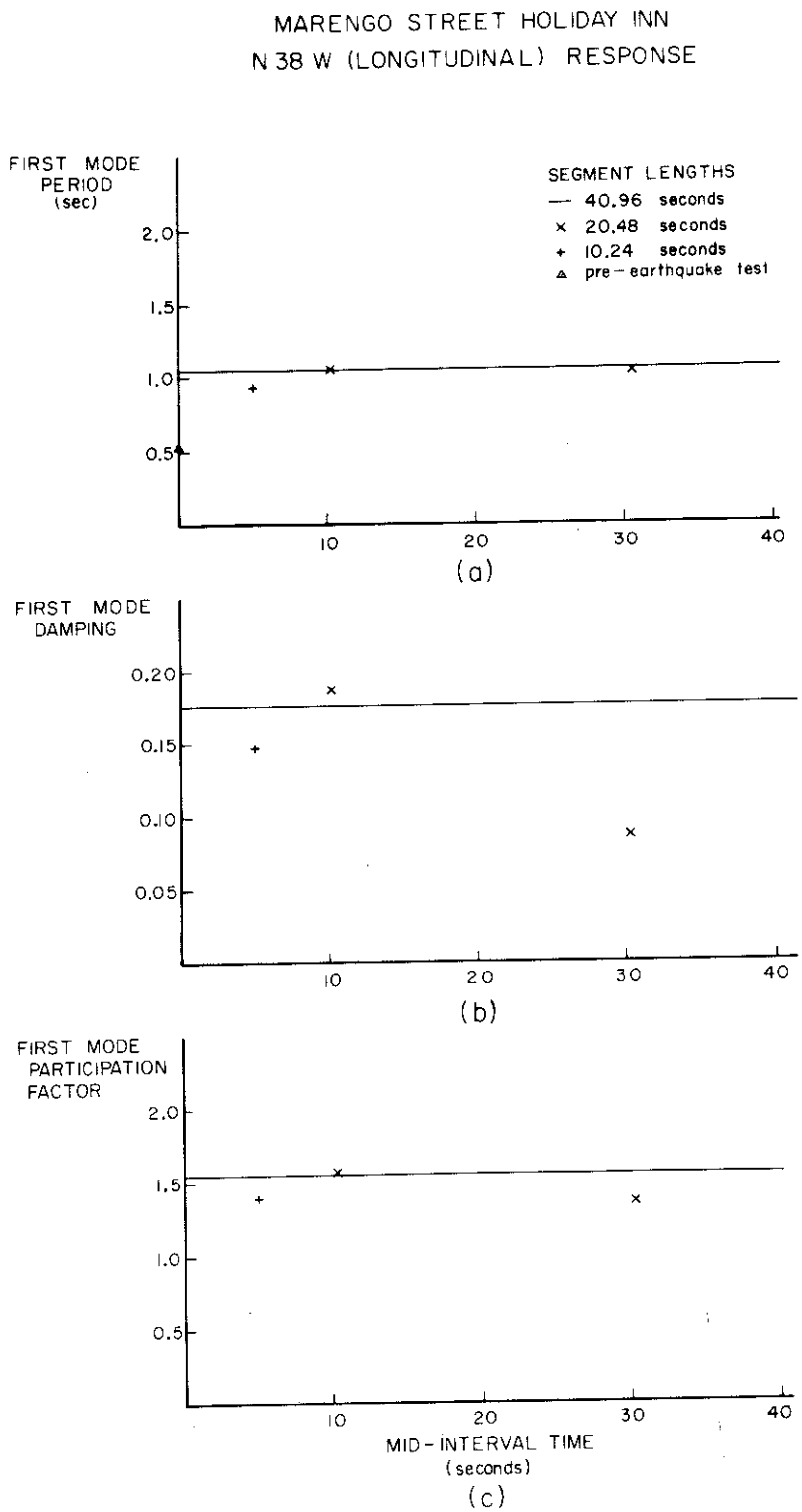


Figure 4.4 Time variation of the parameters of linear models of the Marengo Street Holiday Inn identified from N38W (longitudinal) roof response.

TABLE 4.6: ESTIMATES OF THE PARAMETERS FROM THE MARENGO STREET N38W RECORDS

Record	Parameter			
	T_1 (sec)	ζ_1	C_1 4th	C_1 Roof
0 - 40.96 Seconds				
4th Floor	1.08	0.184	0.91	-
Roof	1.06	0.177	-	1.55
4th Floor and Roof	1.06	0.178	0.90	1.56
0 - 20.48 Seconds				
4th Floor	1.09	0.199	0.93	-
Roof	1.07	0.189	-	1.57
4th Floor and Roof	1.07	0.190	0.91	1.58
20 - 40.48 Seconds				
4th Floor	1.05	0.085	0.85	-
Roof	1.05	0.087	-	1.37
4th Floor and Roof	1.05	0.086	0.86	1.36
0 - 10.24 Seconds				
4th Floor	0.96	0.156	0.81	-
Roof	0.94	0.148	-	1.40
4th Floor and Roof	0.94	0.148	0.80	1.40

TABLE 4.7: ESTIMATES OF THE PARAMETERS FROM THE MARENGO STREET S52W RECORDS

Record	Parameter			
	T_1 (sec)	ζ_1	C_1 4th	C_1 Roof
0 - 40.96 Seconds				
4th Floor	1.17	0.051	0.53	-
Roof	1.17	0.049	-	0.89
4th Floor and Roof	1.17	0.050	0.53	0.89
0 - 20.48 Seconds				
4th Floor	1.16	0.045	0.50	-
Roof	1.17	0.045	-	0.85
4th Floor and Roof	1.17	0.045	0.50	0.84
20 - 40.48 Seconds				
4th Floor	1.20	0.100	0.86	-
Roof	1.17	0.087	-	1.36
4th Floor and Roof	1.18	0.090	0.83	1.37
0 - 10.24 Seconds				
4th Floor	1.08	0.204	1.14	-
Roof	1.12	0.188	-	1.81
4th Floor and Roof				

overall damping, but this estimate was associated with a roof participation factor of 0.89 and a fourth floor value of 0.50, compared with the design model values of 1.31 and 0.70. A roof participation factor of less than one is not possible for a fundamental mode shape in which the modal displacement increases monotonically with height. The value of five percent damping appears low considering the extent of damage and the values found for the other components in these two buildings. The problem appeared to stem from attempting to match highly nonlinear response using linear models. The estimates from the second half of the record seemed reasonable.

4.2 THE BANK OF CALIFORNIA, 15250 VENTURA BOULEVARD

The Bank of California building at 15250 Ventura Boulevard, a twelve-story reinforced concrete moment-resisting frame structure situated fourteen miles from Pacoima Dam (Fig. 1.1), ranked with the two Holiday Inns as the most extensively damaged of the structures from which response records were obtained during the San Fernando earthquake. Repair of the structural damage, which consisted of cracking and spalling of concrete from columns, spandrel beams, girder stubs and a parapet wall at the first story, required $12,000 of the total repair costs of $44,000. The original construction cost of the building was $4 million. The building, which was completed in 1970, was designed in accordance with the 1968 Los Angeles City Building Code. A detailed description of the structure has been given by Blume and Associates [1]. The accelerogram data are available in the Caltech reports [4]. A brief description of the building and its response is also given by Foutch et al. [5].

Reference [1] also presents an analysis of the earthquake response of the structure based on modifications to synthesized linear models. No model was found which could closely approximate the entire response in either direction. The linear dynamic analyses indicated that the yield levels were exceeded within the first five seconds of the response The response was marked by substantial changes in the stiffness and by high damping.

The earthquake response was also studied by Hart et al. [6,7] by Fourier analysis techniques. Hart gave values for the effective periods and dampings of the first three translational modes in both directions, as well as the ambient fundamental periods measured after the earthquake (Table 4.8).

The appreciable lengthening in the fundamental periods are apparent from Table 4.8. The modal properties of the building as determined in previous studies are listed together with the values identified from various segments of the response by the present frequency domain method.

Digitized records were available for approximately forty seconds of the earthquake motion recorded at the roof, seventh and ground floors. Extensive time window analyses of the roof record have been performed for the longitudinal N11E direction, with less detailed studies of the S79W response and of the seventh floor records.

The identification was complicated in the early part of the response by the significant contribution of several modes to the motion, and by rapid changes in the values of the parameters of the effective linear models. After about fifteen seconds, the response was predominantly in the fundamental mode, and the parameter values changed slowly.

The normalized errors in early segments for three and four mode models exceeded 0.3, indicating poor matches of the recorded response, but dropped to 0.06 for a three mode model for the last twenty seconds of the record.

The response of this building was characterized by great period lengthening. The periods identified during the earthquake for the longitudinal N11E direction ranged from 1.47 seconds for the 0-10 second segment to 2.37 seconds over the final twenty seconds. No vibration test values were available from before the earthquake. However, the initial synthesized models produced periods of 0.85 to 0.93 seconds, depending on which elements were assumed to resist the lateral forces. This range seemed a reasonable indication of the initial period as estimated from an extrapolation of the plot of the fundamental period as function of mid-interval time (Fig. 4.5) back to the beginning of the record. Thus the fundamental period may well have lengthened by a factor of 2.5 during the earthquake, indicative of the nonlinear behavior associated with the damage to the building. The fundamental period measured in ambient tests after the earthquake was 1.62 seconds, showing that the large amplitude response had permanently softened the structure.

The value identified for the second mode period remained fairly constant during the response at about 0.5 seconds. Attempted identifications with five second segments at the beginning of the response were unsuccessful, preventing the tracing of any initial rapid lengthening of the second mode period from the expected initial value near the 0.30 seconds calculated for the synthesized model. Late in the response there were indications that the period had lengthened to the range of

0.65 to 0.85 seconds, but at this stage the second mode made only a minor contribution to the response rendering estimates of its parameters unreliable.

The first mode damping for most of the response was identified as approximately 15 percent of critical. The second mode damping was typically about half the first mode value.

The magnitudes of the effective participation factors for the roof for the first and second modes generally exceeded the values of 1.27 and -0.45 calculated from the synthesized model, typically taking values around 1.6 and -0.6. The increased first mode participation factor for the roof suggested that the higher floors made a relatively greater contribution to the mode shape than in the synthesized model. However, the participation factors identified from the last twenty seconds of the records gave a seventh floor fundamental mode displacement of 0.66 of the value at the roof, compared with 0.56 in the synthesized model.

The contradictory information about the shape of the fundamental mode given by the increased roof participation factor and the relatively greater modal displacement at the seventh floor, compared with the synthesized model, may be symptomatic of the difficulty of fitting the response of this structure with linear models, as illustrated by the large normalized errors. In common with identifications performed for other structures, the reliability of the estimates of the participation factors also suffer from interaction with the estimates of the damping, and from less sensitivity of the measure-of-fit J to these parameters than to the modal frequencies.

Comparisons of the time histories of the measured and model responses are shown in Figure 4.6 for two of the identified segments. The velocity rather than acceleration histories are shown, which masks the poor fit of the higher frequency components caused by neglecting the higher modes, particularly for the segment at the beginning of the record. Despite the marked nonlinear behavior of this structure, as evidenced by the widely different values of the parameters in the two segments, the matches of the velocity time histories obtained from the identifications are remarkably good, illustrating the power of the systematic identification technique in this regard. There is an obvious discrepancy between the measured and model time histories between 29 and 35 seconds, which also occurred in the seventh-floor records. Attempts to study this segment of the record with shorter time windows were unsuccessful.

There were more problems in obtaining realistic estimates of the parameters from the N79W records than from the N11E records. As for the N11E component, the nonlinear behavior was evident from a large increase in the fundamental period. The ambient period measured after the earthquake was 2.0 seconds, while the initial synthesized model produced a value of 1.6 seconds. The identified values varied from 1.88 seconds for the first twenty second window to 3.05 seconds over the last twenty seconds (Fig. 4.7a). The estimates of the participation factors were excessively small, less than unity for the roof for most segments (Fig. 4.7c). The maximum damping identified from a segment of the roof record was 10 percent over the last twenty seconds of the record, but the value for the 5 to 25 second segment became $14\frac{1}{2}$

TABLE 4.8: MODAL PROPERTIES OF THE BANK OF CALIFORNIA BUILDING

(a) Longitudinal N11E Direction

Source	Period (seconds)			Damping (%)			Participation Factors					
	T_1	T_2	T_3	ζ_1	ζ_2	ζ_3	C_1 Roof	C_2 Roof	C_3 Roof	C_{1} 7th	C_{2} 7th	C_{3} 7th
Ambient Tests: Post-Earthquake												
Hart [6]	1.62											
Mulhern and Maley [8]	2.2											
Design Models [1]												
Structural elements only LB1	0.93	0.30	0.19				1.27	-0.45	0.17	0.71	0.51	0
All elements LA	0.85											
Earthquake Response												
Fourier Spectra-Hart [6]	2.38	0.67	0.49	10.4	6.0	4.8						
Adjusted design models [1]												
LB2	1.50			5% all modes								
LB3	1.80			10% all modes								
Apparent effective properties [1]												
0-3 seconds	1.15			5								
3-15 seconds	1.5			5								
15-21 seconds	1.8			10								
21-28 seconds	2.5			10								
Frequency Domain Identification												
0-20.48 seconds Roof record	1.74	0.49	0.28	12.9	8.3	7.5	1.49	-0.57	0.24			
7th Fl. record	1.77	0.51		11.4	5.9					0.85	0.46	
5-25.48 seconds Roof record	2.19	0.50	0.29	18.8	8.3	5.9	1.72	-0.59	0.24			
19-39.48 seconds Roof record	2.35	0.82		12.1	4.8		1.52	-0.79				
7th Fl. record	2.34	0.68		12.3	5.7					1.00	0.40	

TABLE 4.8: MODAL PROPERTIES OF THE BANK OF CALIFORNIA BUILDING

(Cont'd)

(b) Transverse N79W Direction

Source	Period (seconds)			Damping (%)			Participation Factors					
	T_1	T_2	T_3	ζ_1	ζ_2	ζ_3	C_1 Roof	C_2 Roof	C_3 Roof	C_1 7th	C_2 7th	C_3 7th
Ambient Tests: Post-Earthquake												
Hart [6]	2.0											
Mulhern and Maley [8]	3.0											
Design Models [1]												
Structural elements only TB	1.60	0.50	0.28	5% all modes			1.29	-0.45	0.17	0.77	0.46	-0.10
All elements TA	1.33											
Earthquake Response												
Fourier Spectra-Hart [6]	2.94	0.98	0.59	9.0	8.0	5.3						
Adjusted design model TC2 [1]	2.50			10% all modes								
Apparent effective properties [1]												
0-5 seconds	1.33			5								
5-15 seconds	1.65			5								
15-21 seconds	2.0			5								
21-28 seconds	2.5			10								
Frequency Domain Identification												
0-20.48 seconds Roof record	1.88	0.58	0.32	5.8	5.1	7.0	1.40	-0.44	0.23			
5-25.48 seconds Roof record	2.56	0.59	0.32	9.5	3.5	7.0	0.84	-0.23	0.22			
7th Fl. record	2.65	0.59		13.0	3.9					0.64	0.44	
15-35.48 seconds Roof record	2.88			4.8			0.79					
7th Fl. record	2.92			2.6						0.48		
19-39.48 seconds Roof record	3.01			10.0			1.38					
7th Fl. record	3.05			7.9						0.91		

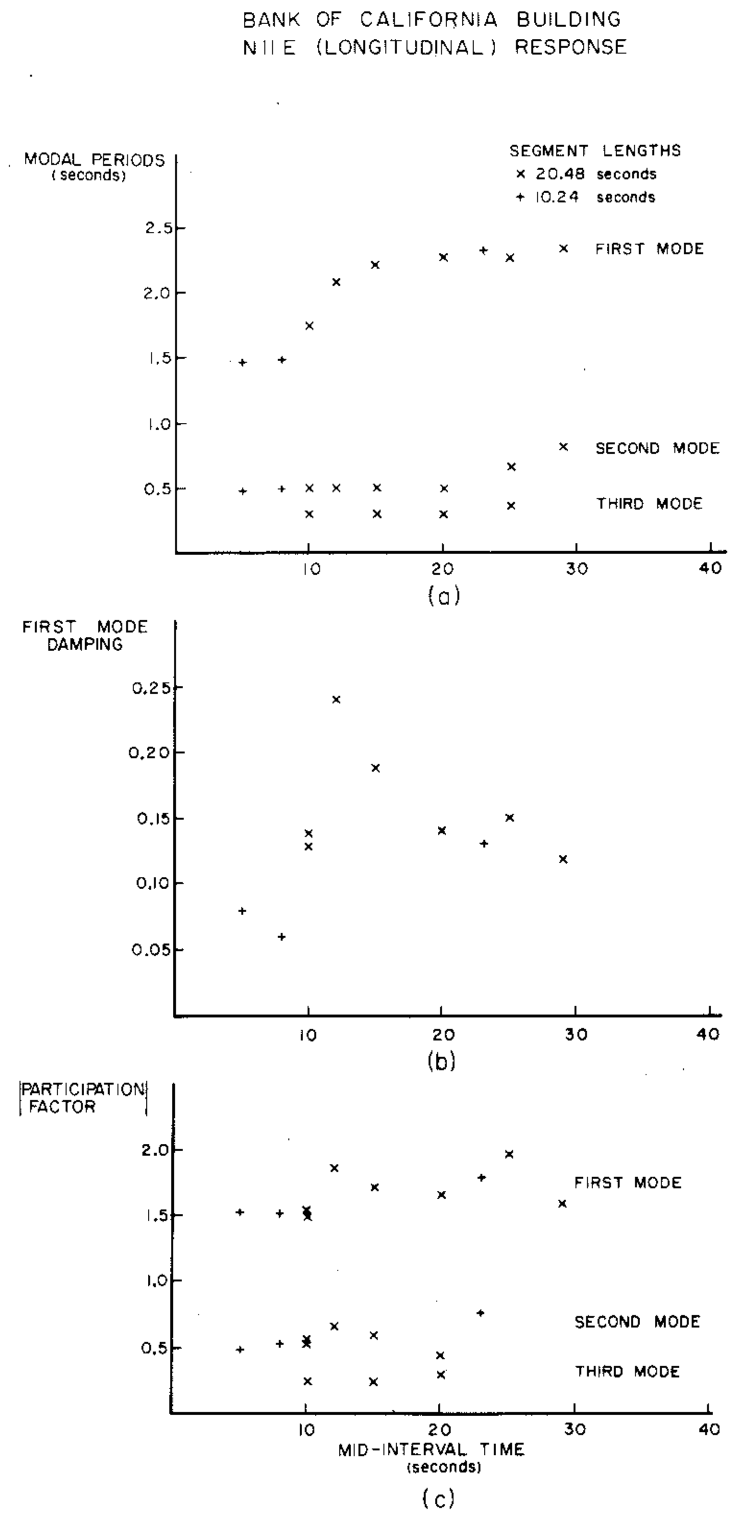
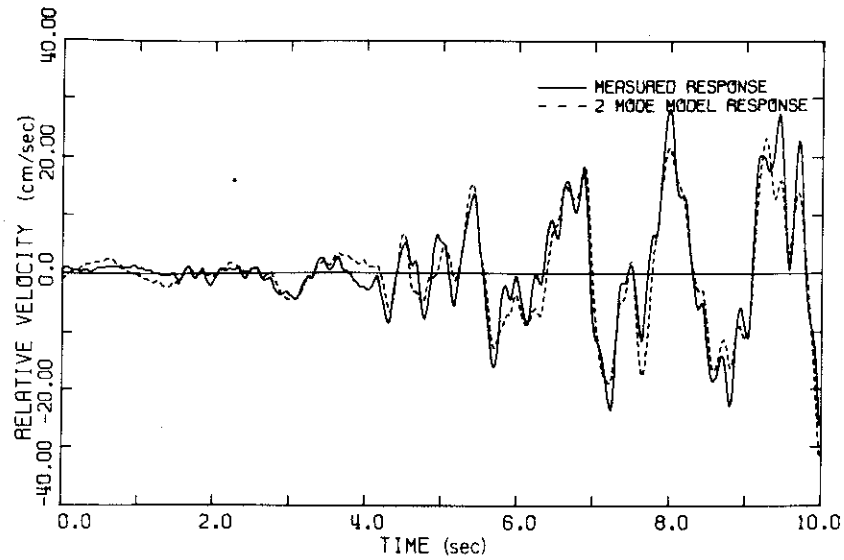
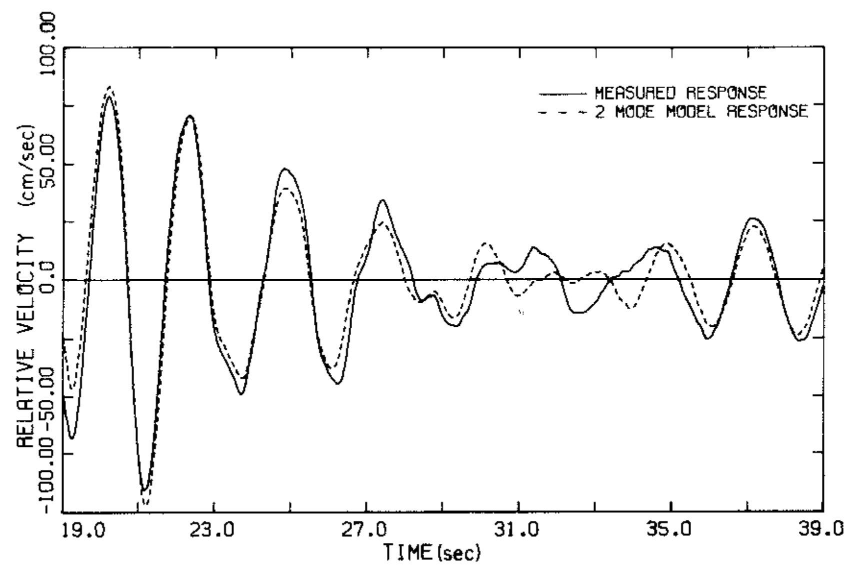


Figure 4.5 Time variation of the parameters of linear models of the Bank of California building identified from segments of the longitudinal N11E roof response.

BANK OF CALIFORNIA N11E RESPONSE



(a)



(b)

Figure 4.6 Velocity matches achieved by the optimal two-mode models for two segments of the Bank of California N11E roof response. Note the considerably different effective parameter values.

(a) 0-10 seconds. Normalized acceleration error E = 0.41.

T_1 = 1.47 seconds $\quad \zeta_1 = 0.080 \quad C_{1Roof} = 1.52$

T_2 = 0.47 seconds $\quad \zeta_2 = 0.049 \quad C_{2Roof} = 0.49$

(b) 19-39 seconds. Normalized acceleration error E = 0.064.

T_1 = 2.35 seconds $\quad \zeta_1 = 0.121 \quad C_{1Roof} = 1.52$

T_2 = 0.82 seconds $\quad \zeta_2 = 0.48$ (?) $\quad C_{2Roof} = -0.79$ (?)

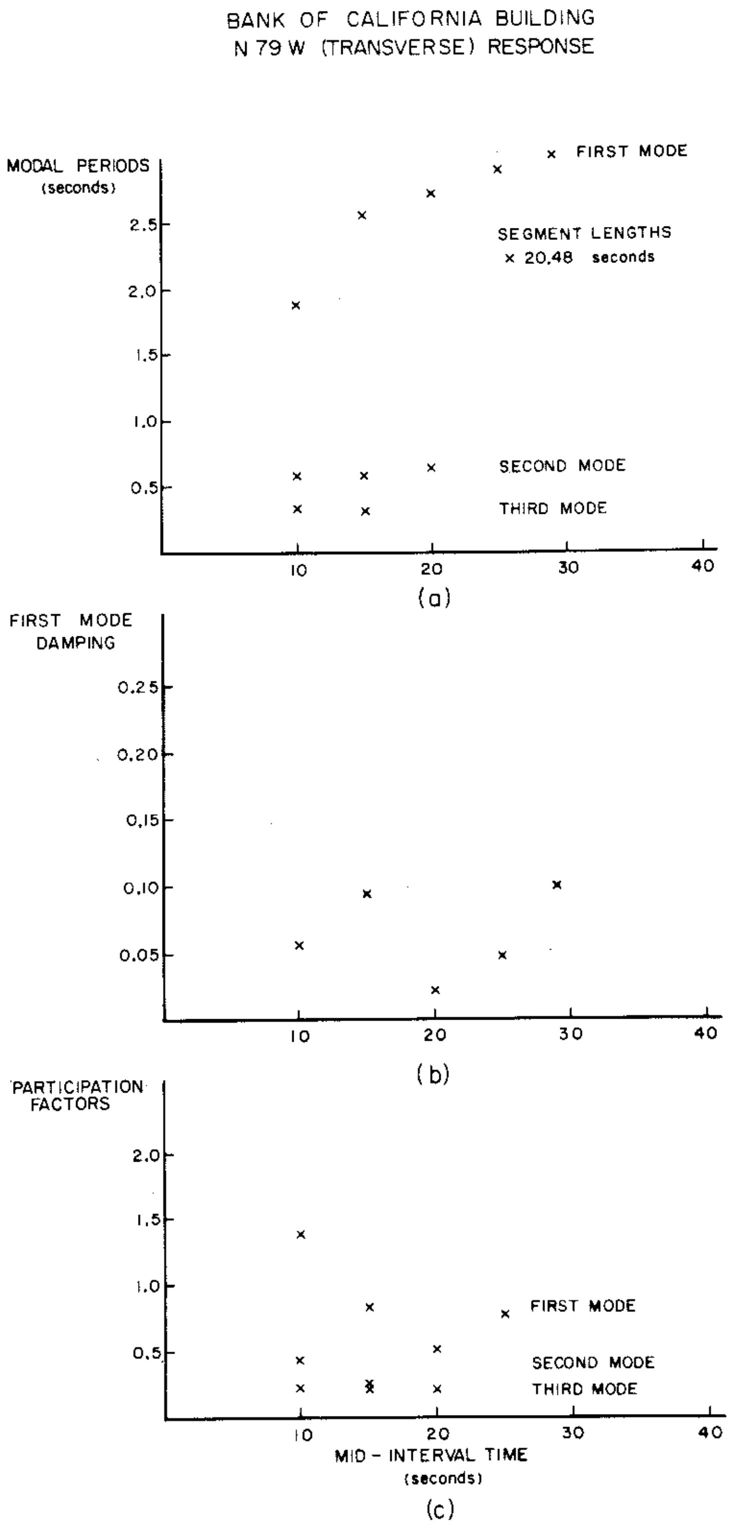


Figure 4.7 Time variation of the parameters of linear models of the Bank of California building identified from segments of the transverse N79W response.

percent when "corrected" by the ratio of the participation factor of the synthesized model to the identified model. The normalized errors for most segments were between 0.2 and 0.4, signifying problems of matching the recorded response with linear models.

4.3 SUMMARY

These examples represent the extreme limits of the applicability of the equivalent linear model concept for the estimation of structural parameters from recorded earthquake response data. The structures suffered considerable nonstructural damage, together with the onset of structural damage. Significant nonlinear behavior was asssociated with these damaging levels of response.

The buildings responded at amplitudes considerably beyond their elastic limits. The nonlinear behavior was evident in the identified lengthenings of the fundamental periods. The effective fundamental periods of the two Holiday Inns during the San Fernando earthquake response were approximately three times the initial elastic period for the Orion Avenue structure and twice the small-amplitude period for the Marengo Street building, corresponding to approximate overall ductility factors of eight and four. The fundamental period of the Bank of California in the longitudinal N11E direction may have lengthened to as much as 2.5 times its small-amplitude value.

It may be expected that such nonlinear behavior would be poorly represented by linear models, and indeed the matches of the recorded and model responses for these buildings were poorer than for the other structures studied when single time-invariant models were used for the entire response. Nevertheless, the systematic identification technique produced surprisingly good fits for response segments in the second half of the records, and some important information about the structures. Considering the quality of the response matches achieved with linear models, their main usefulness was in tracing the variation of the effective linear parameters rather than finding values appropriate for the overall response.

The nonlinear behavior was reflected by inconsistent estimates of some parameters from records at the midheight and roof levels. Some estimates were also unrealistic, as, for example, values of the effective first mode participation factor turning out less than unity. In common with structures whose response can be reproduced very well by linear models, the identifications were also plagued by interaction between the estimates of the dampings and participation factors. The problems were most serious for the transverse (S52W) records from the Marengo Street Holiday Inn and for the transverse (N79W) response of the Bank of California.

Despite the difficulties in performing the identifications, interesting information was obtained for the dampings in these reinforced concrete frame structures responding at amplitudes at the onset of significant structural damage. The effective first mode dampings reached peak values in the range of fifteen to twenty percent, typically dropping to half these values in the later stages of the responses. In interpreting these values, it should be recalled that they are associated with the lengthened values of the periods, and not with the initial periods as is the case in many uses of equivalent damping.

REFERENCES

1. Blume, John A. and Associates, "Bank of California, 15250 Ventura Boulevard, Los Angeles", in 'San Fernando, California, Earthquake of February 9, 1971', L.M. Murphy (ed.), Vol. I, Part A, 327-57, U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration (NOAA), Washington, D.C. 1973.

2. Blume, John A. Associates, "Holiday Inn, 1640 Marengo Street, Los Angeles", in 'San Fernando, California Earthquake of February 9, 1971', L.M. Murphy (ed.), Vol. I, Part A, 395-422, U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration (NOAA), Washington, D.C. 1973.

3. Blume, John A. and Associates, "Holiday Inn, 8244 Orion Avenue, Van Nuys",in 'San Fernando, California, Earthquake of February 9, 1971', L.M. Murphy (ed.), Vol. I, Part A. 359-93, U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration (NOAA), Washington, D.C. 1973.

4. California Institute of Technology, "Strong Motion Accelerograms", Earthquake Engineering Research Laboratory, Pasadena, California.

5. Foutch, D.A., G.W. Housner and P.C. Jennings, "Dynamic Responses of Six Multistory Buildings during the San Fernando Earthquake", Earthquake Engineering Research Laboratory Report No. EERL 75-02, California Institute of Technology, Pasadena, California, October 1975.

6. Hart, G.C., R.M. DiJulio and M. Lew, "Torsional Response of High-Rise Buildings", Journal of the Structural Division, Proc. ASCE, Vol. 101, ST2, 397-416, February 1975.

7. Hart, G.C. and R. Vasudevan, "Earthquake Design of Buildings: Damping", Journal of the Structural Division, Proc. ASCE, Vol. 101, ST1, 11-29, January 1975.

8. Mulhern, M.R. and R.P. Maley, "Building Period Measurements Before, During and After the San Fernando Earthquake", in 'San Fernando, California, Earthquake of February 9. 1971', L.M. Murphy (ed.), Vol. I, Part B, 725-33, U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration (NOAA), Washington, D.C., 1973.

V. BUILDINGS WITH NO STRUCTURAL DAMAGE

This chapter describes the identification analyses performed for several structures whose responses were typical of those obtained in many buildings during the San Fernando earthquake. These structures responded at moderate amplitudes not sufficient to cause structural damage, yet large enough to cause minor nonstructural damage and to cause significant changes in the dynamic properties from those measured in ambient vibration tests. Such records are particularly appropriate for analysis by systematic identification techniques utilizing linear models since the response is not dominated by nonlinear behavior, yet the effective values of the parameters are sufficiently changed from the very low amplitude values to make their determination for the amplitudes of earthquake response important. The earthquake records are the only source of this information.

5.1 UNION BANK BUILDING, 445 SOUTH FIGUEROA STREET

The Union Bank building, its behavior in ambient vibration tests, and its response to the San Fernando earthquake have been discussed by A. C. Martin and Associates [9], Trifunac [12, 14], Foutch et al. [4], and Beck [1]. The accelerogram and response spectra data are available in the Caltech reports [3]. The following summary has been derived mainly from the detailed description by the designers in reference [9].

The 42 story steel frame structure (Fig. 5.1) designed in 1964 was one of the first high-rise buildings in downtown Los Angeles. The design static loads were approximately double those required by the Los Angeles City Building Code, while the interstory displacement

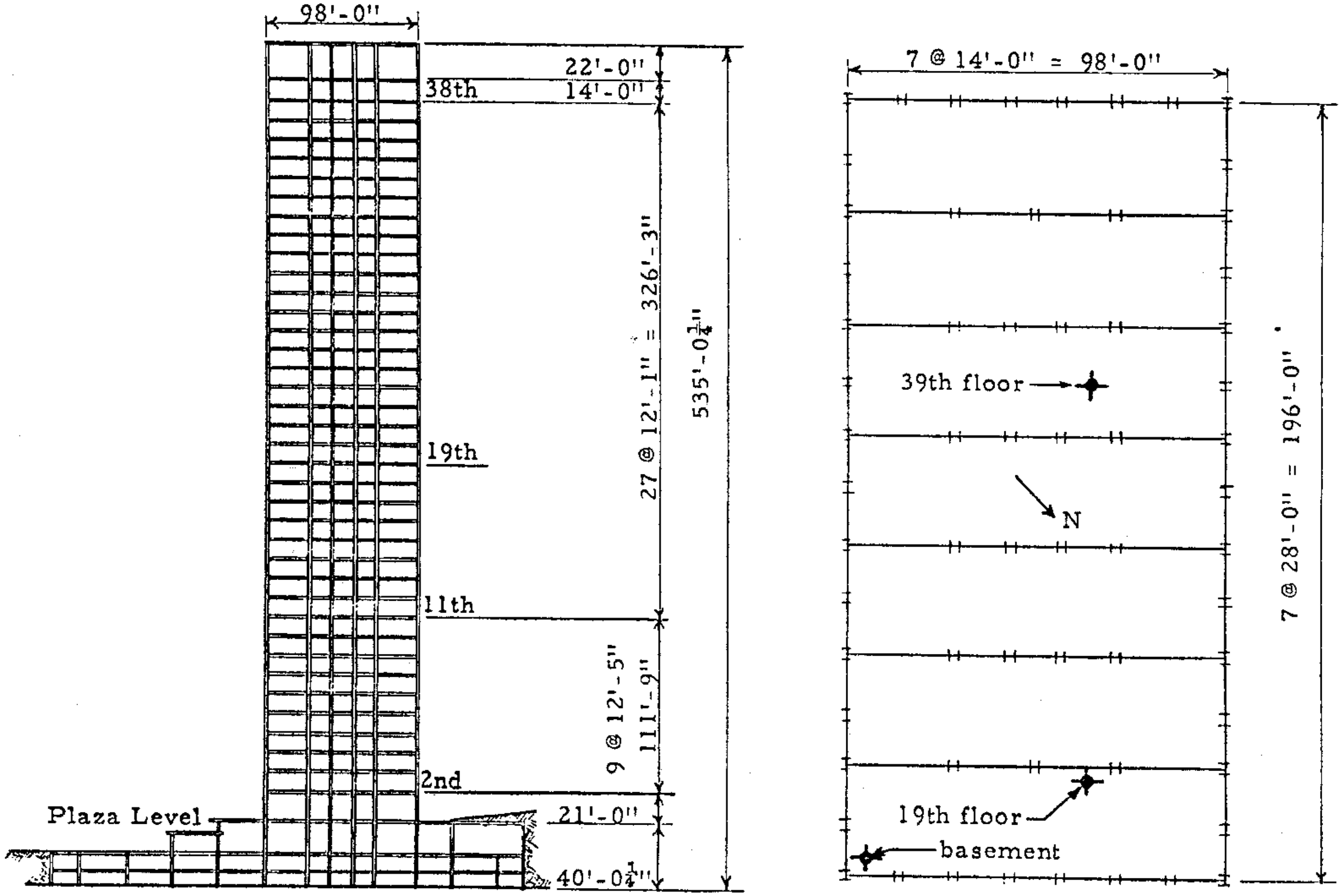


Figure 5.1 Transverse section and typical floor plan of the Union Bank building (from [9]).

limitations were half those allowed by the code. The design was evaluated by dynamic analysis of the building's response to the El Centro 1940 north-south component and other accelerograms.

The building, situated approximately 21 miles from the center of the San Fernando earthquake (Fig. 1.1), experienced peak ground accelerations of 0.15g and 0.12g in the two horizontal components. The strong-motion accelerometer on the roof malfunctioned, but the midheight instrument on the 19th floor recorded a maximum response of 0.20g. Only minor nonstructural damage, such as plaster cracking, occurred.

The modal properties of the Union Bank building as determined from various studies are listed in Table 5.1.

As shown in the table, the natural periods changed between ambient vibration tests performed before and after the earthquake. The mode shapes plotted by Trifunac [14] also showed changes following the earthquake.

The parameters of rigid- and flexible-joint design models [9] are given in Table 5.1. The designers felt that the flexible-joint models, which were used to calculate the El Centro responses, would be the more appropriate for large-amplitude earthquake response. However, in the post-earthquake analysis the rigid-joint models gave better agreement with the periods observed in the San Fernando response. The value of five percent damping assumed initially for all modes was changed to four percent for the transverse N52W direction to achieve an improved match of the earthquake response.

Beck [1] used a time domain method to identify linear models from portions of the longitudinal S38W response. The model for the entire segment of the response he considered, from 5 to 30 seconds, is compared to the model identified by the frequency domain method for the most closely corresponding segment, from 7.5 to 28 seconds. The models for both directions identified from the whole 41 seconds of the analyzed records are also listed.

The models identified from the 19th floor (midheight) response produced estimates of the parameters of four modes. For the transverse N52W direction, this provided information about the first six modes, as the 19th floor was near nodes of the third and fifth modes which therefore produced no identifiable response at the measurement location.

The overall effective fundamental periods of 4.63 seconds and 4.71 seconds identified from the earthquake response records in the longitudinal S38W and transverse N52W directions showed increases of 50 and 33 percent respectively over the periods of 3.11 seconds and 3.53 seconds measured in the pre-earthquake ambient tests. The post-earthquake ambient tests showed increases of about half these amounts.

The bulk of the changes in the fundamental periods apparently occurred rapidly at the beginning of the response. A steady increase from 4.48 seconds to 4.79 seconds was traced in the S38W direction using a series of twenty second segments of the records (Fig. 5.3a). However, this change of seven percent was only a small fraction of the total increase from the vibration test period, and it seems likely that the initial period was considerably shorter than the effective value identified from the first twenty seconds of record. Unfortunately, the long fundamental period of this tall building prevented the use of shorter time segments to identify the "instantaneous" effective parameter values, as the discretization of the fast Fourier transforms became too coarse to obtain accurate estimates from the low frequency data around the first mode response peak. The time domain technique used by Beck performed successfully with ten second intervals, but he considered only the portion of the response from 5 to 30 seconds, so did not find parameter values effective for the first ten seconds of the response. His values were in agreement with those found by the frequency domain method, with estimates of the fundamental period ranging from 4.42 seconds over the ten second interval from 5 to 15 seconds, to 4.75 seconds for the interval from 20 to 30 seconds.

The identification studies for the two directions showed that the effective dampings for the first two modes were in the range from 4 to 7 percent, in agreement with the assumed value of 5 percent used in the dynamic analyses during the design. By contrast, the values measured in ambient tests were from 1.5 to 1.7 percent. The values found for the third mode damping given in Table 5.1 are questionable because of the low level of response in this mode at the 19th floor.

The first-mode damping also varied during the response, but not as systematically as the period (Fig. 5.3b). The identified values in the S38W direction for twenty second segments started at 2.3 percent of critical for the first segment, reached a maximum of 4.4 percent in the middle of the record, and dropped to 3.9 percent at the end. Comparisons of the measured displacement response and those calculated for identified models (Fig. 5.2c) indicated that the dampings were overestimated in that the amplitude of the calculated responses reduced faster than the measured responses. It seemed that the overestimation of the damping resulted from the variation of the effective fundamental period of the system. As noted in other examples, an increased damping was the only mechanism available to broaden the frequency domain modal peaks of a time-invariant model to match the broadened bandwidth caused by the period variation of the structure. This speculation was supported by the value of 4.9 percent damping identified from the full record, larger than the value estimated for any of the shorter segments.

The identified models listed in Table 5.1 provided reasonable estimates of the participation factors of the first two modes in both directions, as well as indicating the node of the third mode in the N52W direction by the omission of this mode from the model. However, the variation of the participation factors with time as shown in Figure 5.3c was erratic. Part of the variation may be due to the insensitivity of the measure-of-fit to these parameters, and hence their poor estimation, in the later stages of the response which were dominated by the free vibration decay of the motion induced by the earlier excitation.

In both directions, the identified models gave very good matches of the measured velocity and displacement responses (Figs. 5.2b and c, 5.5c and d). The main discrepancies arose from lengthening of the periods during the response and a slight overestimation of the dampings. The acceleration fits were poorer because of the presence of high frequency components in the early part of the response which were not reproduced in the response calculated for any of the models.

There was excellent agreement between the values of the parameters identified for the four-mode model in the S38W direction from the 7.5 to 28 second segment and those estimated by Beck (Table 5.1a). One difference was in the third mode period, 1.08 seconds compared with 0.95 seconds for Beck. However, the 19th floor is near a node of this mode and its contribution to the response is small, and consequently the estimates of the third-mode parameters are less reliable than for the two lower modes.

Neither the rigid-joint nor flexible-joint design models agreed well with the identified models in both directions. Despite the synthesis of the flexible-joint models to calculate earthquake responses, the rigid-joint models gave better agreement with the measured responses in the San Fernando earthquake. In the transverse N52W direction, the rigid-joint model possessed modal properties remarkably close to those identified from the earthquake response. The periods agreed closely up to the sixth mode, apart from the absence from the identified model of the third and fifth modes which contributed little to the midheight response. The fundamental periods were identical, and the largest discrepancy in the periods was less than seven percent for the second mode. The adjusted value of four percent damping for all modes in the design model was also in excellent agreement with the values of 4.1 and 4.2 percent identified for the first two modes. However, this type of agreement between the synthesized and identified models was not obtained in the other direction. As a result of a poor estimation of the effective fundamental period by the design model, the response of the design model (Fig. 5.4a) became out of phase with the measured response to a much greater extent than the response of the identified model (Fig. 5.2).

TABLE 5.1: MODAL PROPERTIES OF THE UNION BANK BUILDING

(a) Longitudinal S38W Direction

Source	Period (Seconds)								Damping (%)				19th Floor Participation Factor				
	T_1	T_2	T_3	T_4	T_5	T_6	T_7	T_8	ζ_1	ζ_2	ζ_3	ζ_4	C_1	C_2	C_3	C_4	C_5
Ambient Tests																	
(a) Pre-earthquake [12]	3.11	1.07	0.61	0.43	0.34	0.27			1.7	1.5	1.8	2.0	0.72	0.45	-0.01	-0.18	-0.05
(b) Post-earthquake [13]	3.77	1.31	0.76	0.54	0.42												
Design Model (Rigid joint) [9]	4.13	1.45	0.87	0.63	0.50	0.42	0.35	0.31	5% all modes				0.66				
Flexible joint	5.04	1.79	1.07	0.78	0.61	0.51	0.44	0.38									
Identified From Eq. Response																	
(a) Beck's time domain method 5-30 sec [1]	4.61	1.49	0.95	0.66					4.2	5.8	13(?)	6.6	0.84	0.46	-0.13(?)	-0.15	
(b) Frequency domain method 7.5-28 sec	4.61	1.48	1.08	0.68					4.4	4.9	16(?)	5.8	0.86	0.44	-0.17(?)	-0.12	
0-41 sec	4.65	1.49	1.06	0.66					3.9	6.7	16(?)	7.5	0.81	0.52	-0.29(?)	-0.20	

(b) Transverse N52W Direction

Source	Period (Seconds)								Damping (%)							19th Floor Participation Factor				
	T_1	T_2	T_3	T_4	T_5	T_6	T_7	T_8	ζ_1	ζ_2	ζ_3	ζ_4	ζ_5	ζ_6	ζ_7	C_1	C_2	C_3	C_4	C_5
Ambient Tests																				
(a) Pre-earthquake	3.53	1.17	0.63	0.44	0.34	0.27	0.22		1.5	1.5	1.5	1.5	3.0	2.0	1.9	0.66	0.50	0.01	-0.16	-0.09
(b) Post-earthquake	4.17	1.35	0.74	0.52	0.41															
Design Model (Rigid joint)	4.71	1.58	0.91	0.65	0.51	0.42	0.35	0.31	4% all modes (initially 5%)							0.57				
Flexible joint	5.43	1.86	1.08	0.77	0.60	0.50	0.42	0.36												
Identified From Eq. Response																				
Freq. domain method 0-41 sec 4 modes	4.71	1.48	-	0.62	-	0.41			4.1	4.2	-	16(?)	-	7.8		0.64	0.50	-	-0.39	-0.24

UNION BANK S38W (LONGITUDINAL) RESPONSE

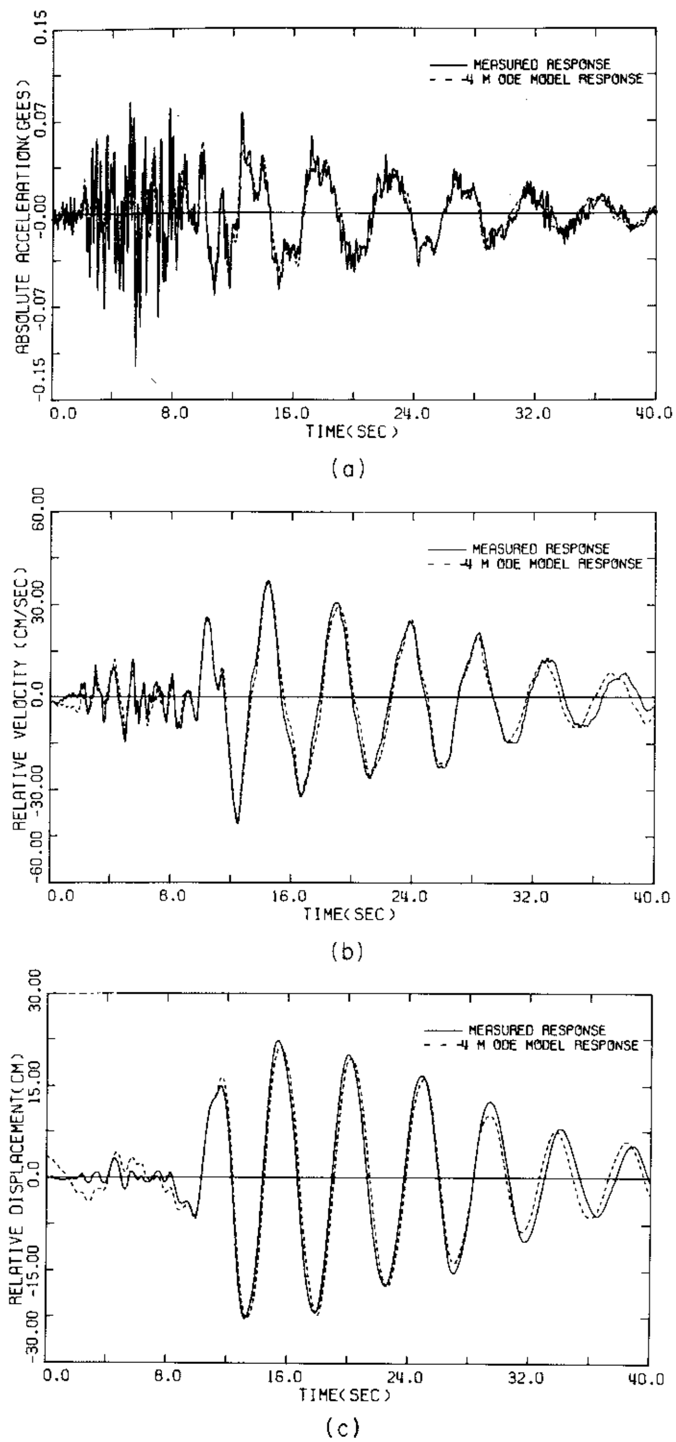


Figure 5.2 The measured and optimal four-mode model response of the Union Bank building, longitudinal (S38W) component. (a) Acceleration. (b) Velocity. (c) Displacement.

UNION BANK S 38 W (LONGITUDINAL) RESPONSE

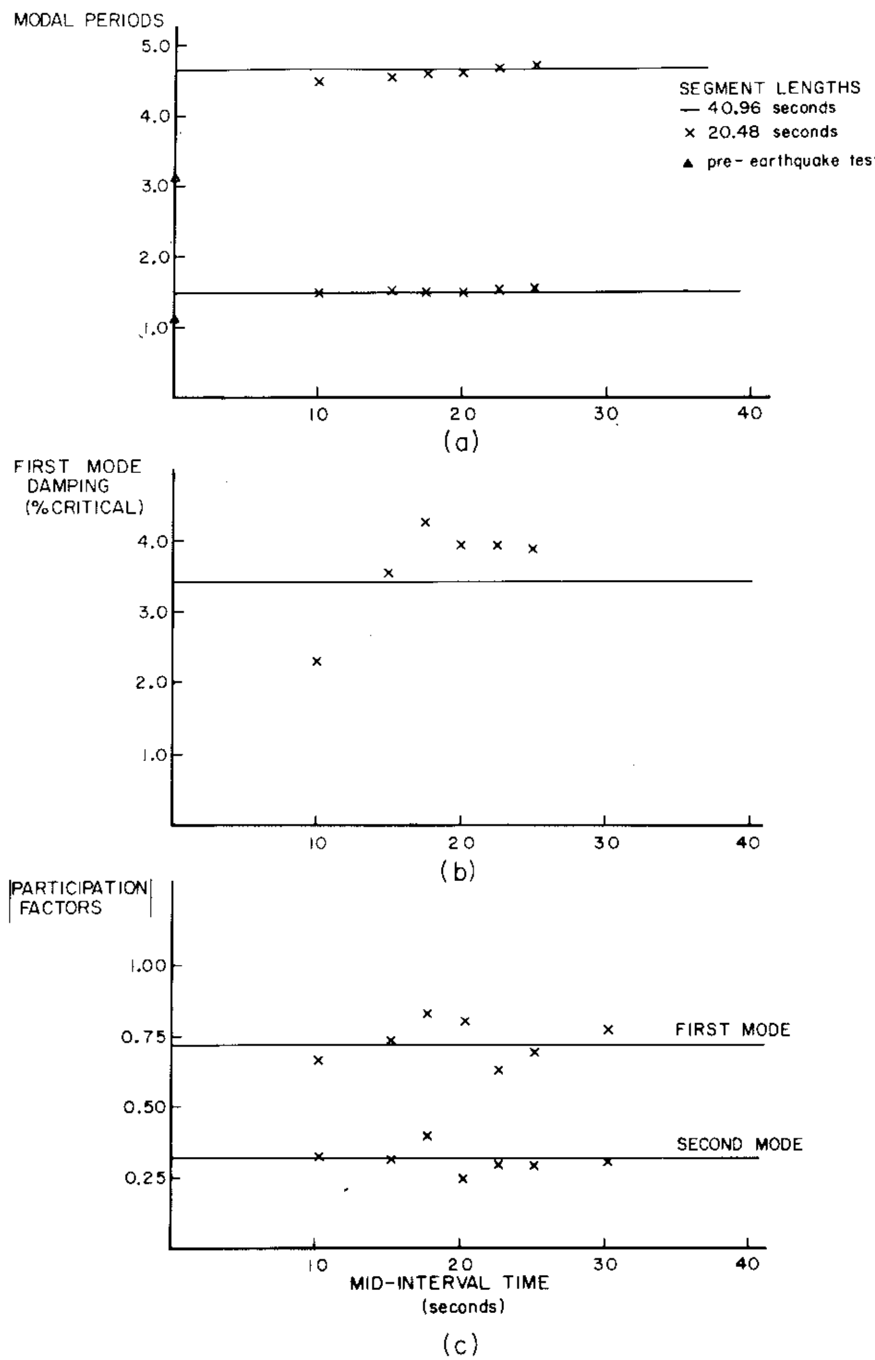


Figure 5.3 Time variation of the parameters of linear models of the Union Bank building identified from segments of the longitudinal response.

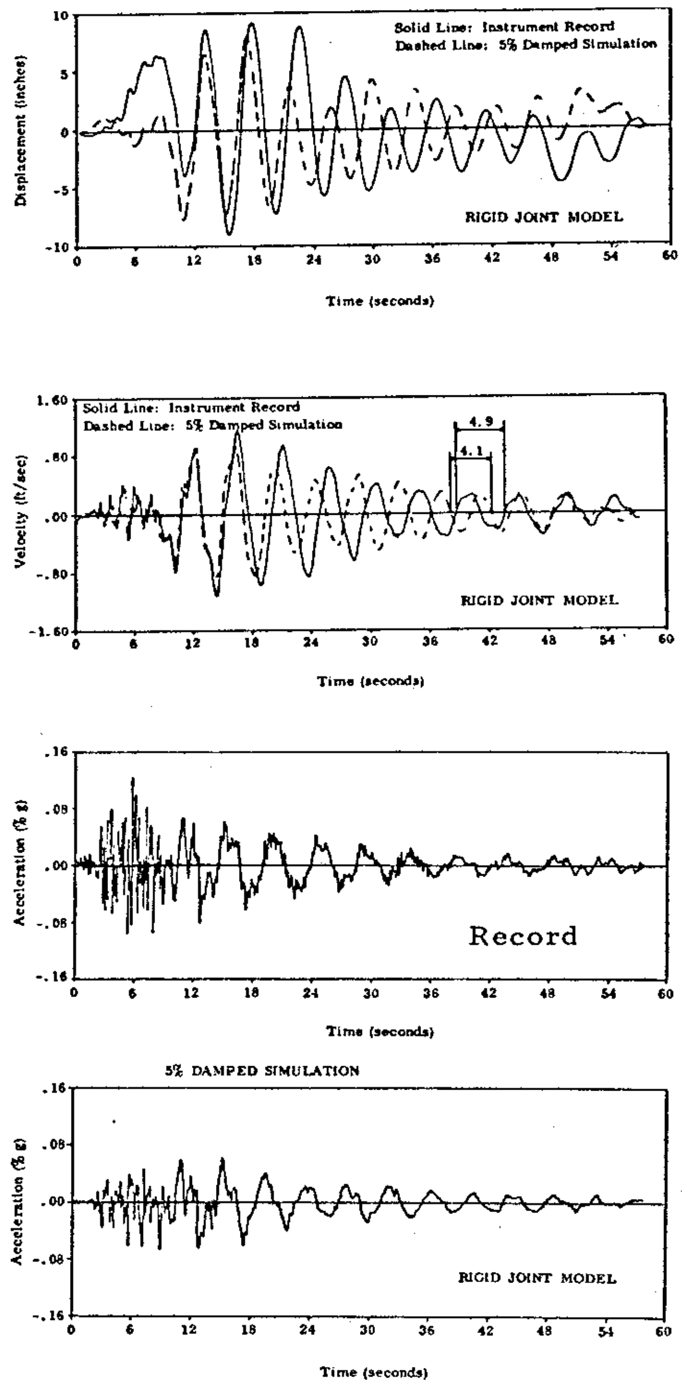


Figure 5.4 Match of the recorded responses and those calculated from the synthesized models [9] at the 19th floor of the Union Bank building during the San Fernando earthquake.

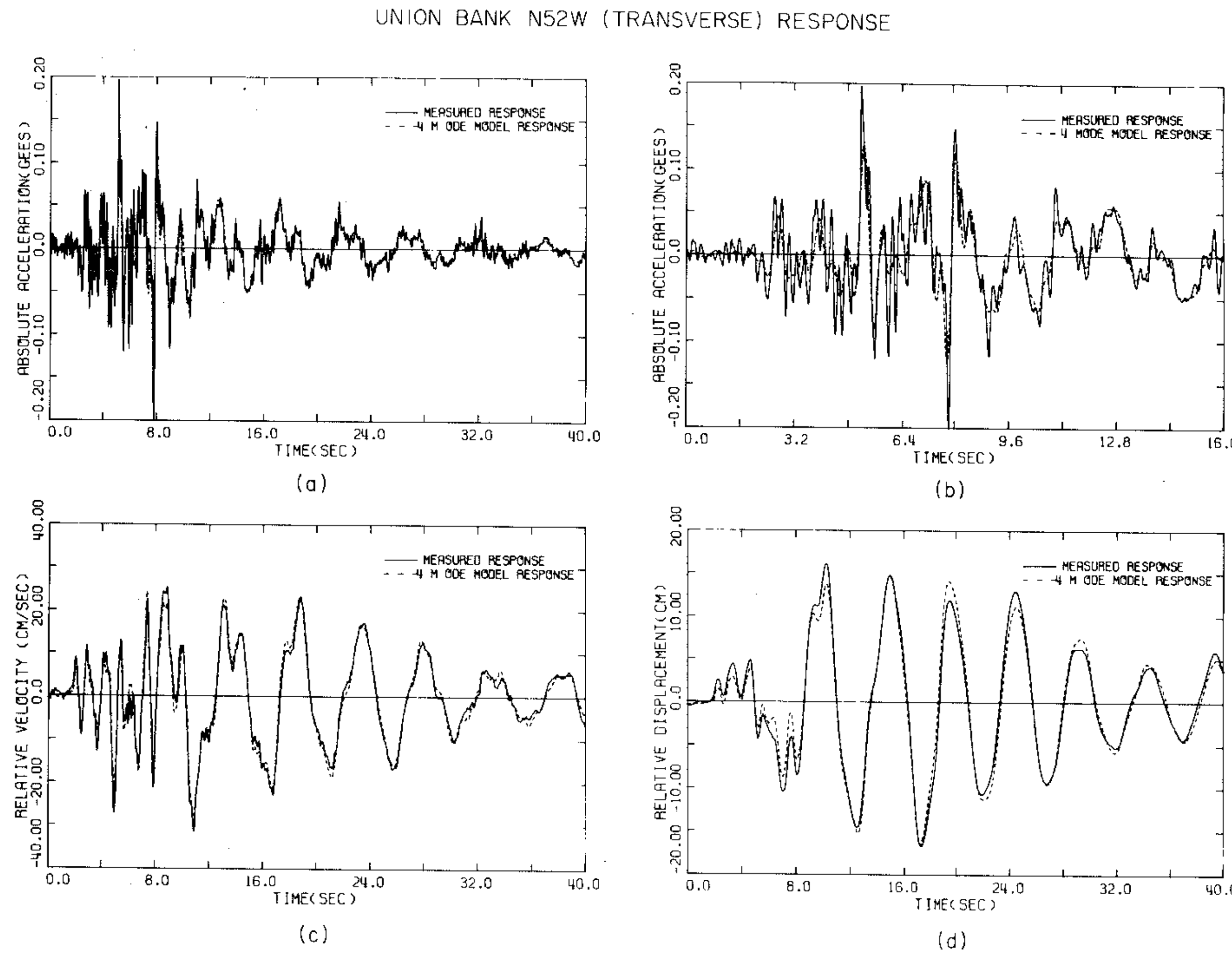


Figure 5.5 The measured and optimal four-mode model response for the Union Bank building, 19th floor transverse (N52W) component. (a) Acceleration histories. (b) First 16 seconds of (a). (c) Velocity histories. (d) Displacement histories.

5.2 KB VALLEY CENTER, 15910 VENTURA BOULEVARD

The KB Valley Center, described by Gates in the NOAA report [6], is a sixteen-story, steel-frame office tower constructed in 1970. The structure was designed in 1969 in compliance with the Los Angeles City Building Code, which for this building was virtually identical to the 1970 U.B.C. provisions. The building was situated fourteen miles from the center of the San Fernando earthquake (Fig. 1.1). The amplitudes of excitation and response were similar in the two horizontal components, with the stronger S81E component, in the longitudinal direction, having a ground acceleration of 0.15g and a response at the roof of 0.23g. Only minor nonstructural damage occurred. The repairs to partitions, mechanical equipment mounts, and seismic joints separating the tower from an adjacent four-story parking building cost approximately $3000, compared to construction costs of $4,000,000.

Periods measured in pre-earthquake ambient tests have been reported by Hart et al. [7,8], and post-earthquake periods by Mulhern and Maley [10]. Hart also estimated the effective periods and dampings from the earthquake response by Fourier analysis techniques.

Gates described trial-and-error modifications of the synthesized design models for the two directions to obtain improved matches of the measured earthquake response. The response calculations with the linear models showed that the displacements in the transverse S09W direction exceeded the design levels by 70 percent, while the longitudinal S81E displacements exceeded the design values by 120 percent. On the basis of the linear calculations, 30 percent of the members yielded in the transverse response, and 80 percent in the longitudinal direction. The ratios of the calculated stresses to the yield stresses indicated that insufficient yielding occurred to develop plastic hinges in the columns, a conclusion supported by an inspection of the structure after the earthquake.

Both components of the roof and 9th floor records have been analyzed by the frequency domain identification technique. The modal properties estimated from the earthquake response are listed in Table 5.2, along with the values determined from the earlier studies.

The study of this building allowed a comparison of the results of the systematic identifications with those of a trial-and-error parameter adjustment technique. The matches of the recorded accelerations achieved by the two methods are shown in Figures 5.6 and 5.7. Although both methods give practically acceptable results for most purposes, the systematic identification gave the better fit.

The parameter estimates from the systematic technique appeared more realistic. In particular, the parameters of the fundamental mode in the transverse direction were estimated poorly by the trial-and-error method. The fundamental period was adjusted from 3.43 seconds to 2.96 seconds. The effective period from the average of the roof and 9th floor records was listed as 3.20 seconds in the NOAA report. The values identified by the frequency domain method were 3.30 and 3.27 seconds from the two records. Hart's analysis produced a value of 3.34 seconds. Gates found a value of 20 percent for the damping of the fundamental mode, which he recognized as excessively high for this type of structure at this level of response. The high damping estimate was probably caused by difficulties in matching the response using the

TABLE 5.2: MODAL PROPERTIES OF THE KB VALLEY CENTRE

(a) Transverse S09W Direction

Source	Period (Seconds)				Damping (%)				Participation Factors						
	T_1	T_2	T_3	T_4	ζ_1	ζ_2	ζ_3	ζ_4	C_{1Roof}	C_{2Roof}	C_{3Roof}	C_{4Roof}	C_{19th}	C_{29th}	C_{39th}
Ambient Tests															
Pre-Earthquake [8]	2.18	0.73													
Post-Earthquake [10]	2.37														
Design Model [6]	3.43	1.33	0.816	0.576	2% all modes				1.51	-0.79	0.50	-0.53	0.60	0.54	0.07
Estimated From Earthquake Response															
Adjusted design model [6]	2.96	1.15	0.703	0.497	20	7.5	5	2							
Apparent effective periods [6]	3.20	1.20	0.75	0.50											
Fourier analysis [7]	3.34	1.17	0.80		11.3	6.3	7.4								
Frequency domain identification - Roof record	3.30	1.15	0.65	0.48	8.6	8.2	11.2	5.6	1.32	-0.74	0.48	-0.18			
- 9th floor record	3.27	1.16			8.8	6.4							0.47	0.61	

(b) Longitudinal S81E Direction

Source	Period (Seconds)				Damping (%)				Participation Factors						
	T_1	T_2	T_3	T_4	ζ_1	ζ_2	ζ_3	ζ_4	C_{1Roof}	C_{2Roof}	C_{3Roof}	C_{4Roof}	C_{19th}	C_{29th}	C_{39th}
Ambient Tests															
Pre-Earthquake	1.94	0.70													
Post-Earthquake	2.27														
Design Model	3.37	1.29	0.801	0.567	2% all modes				1.41	-0.68	0.41	-0.25	0.65	0.52	0.01
Estimated From Earthquake Response															
Adjusted design model	3.02	1.15	0.715	0.507	7	5	5	5							
Apparent effective periods	3.00	1.11	0.695												
Fourier analysis	3.28	1.11	0.75		8.7	6.0	5.4								
Frequency domain identification - Roof record	3.05	1.11	0.65		6.3	7.6	7.5		1.31	-0.72	0.35		0.60	0.63	
- 9th floor record	3.07	1.11			5.8	6.8									

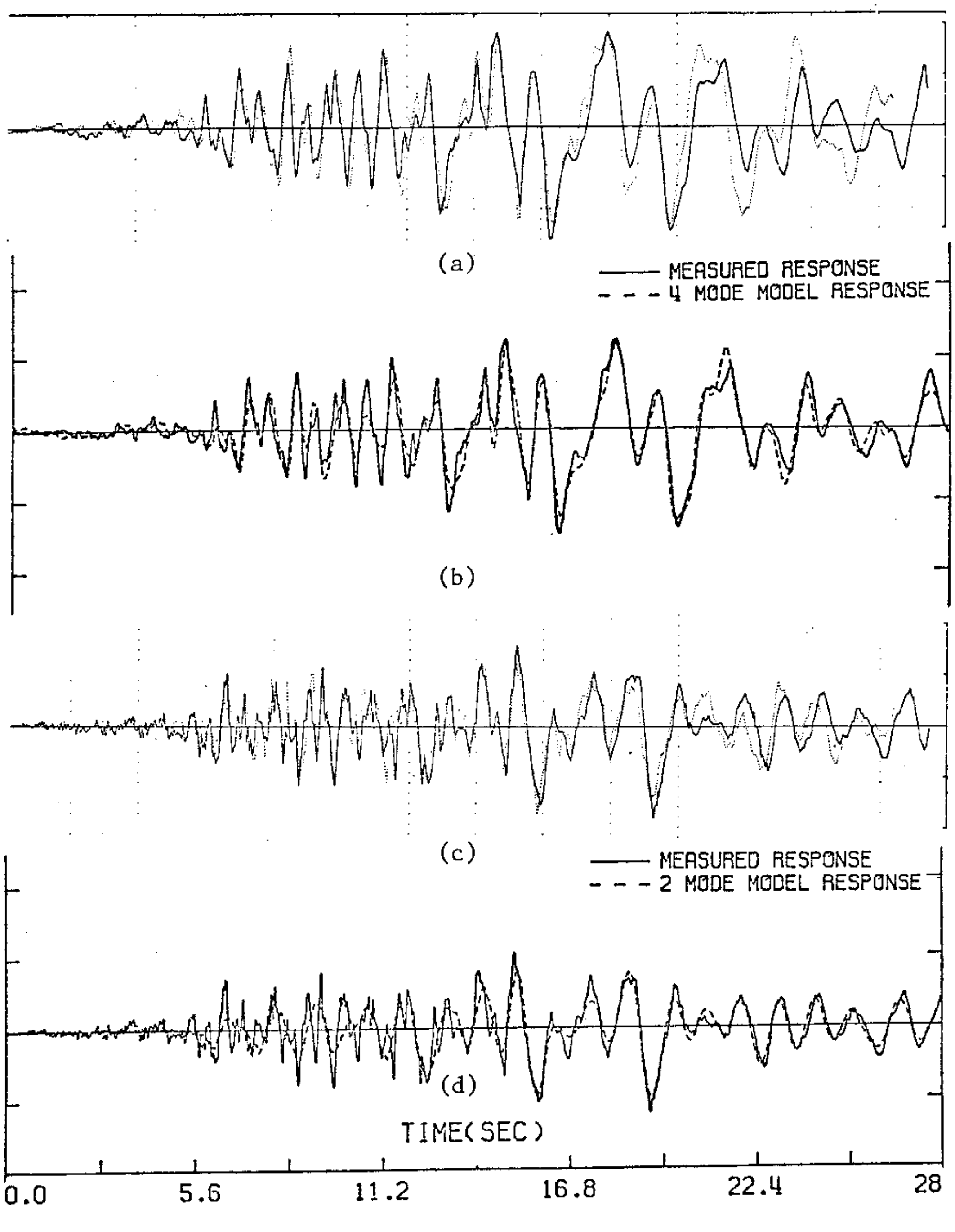


Figure 5.6. KB Valley Center, S09W acceleration responses. (a) Adjusted synthesized sixteen-mode model [6] roof response. (b) Optimal four-mode model roof response. (c) Adjusted synthesized sixteen-mode model 9th floor response. (d) Optimal two-mode model 9th floor response.

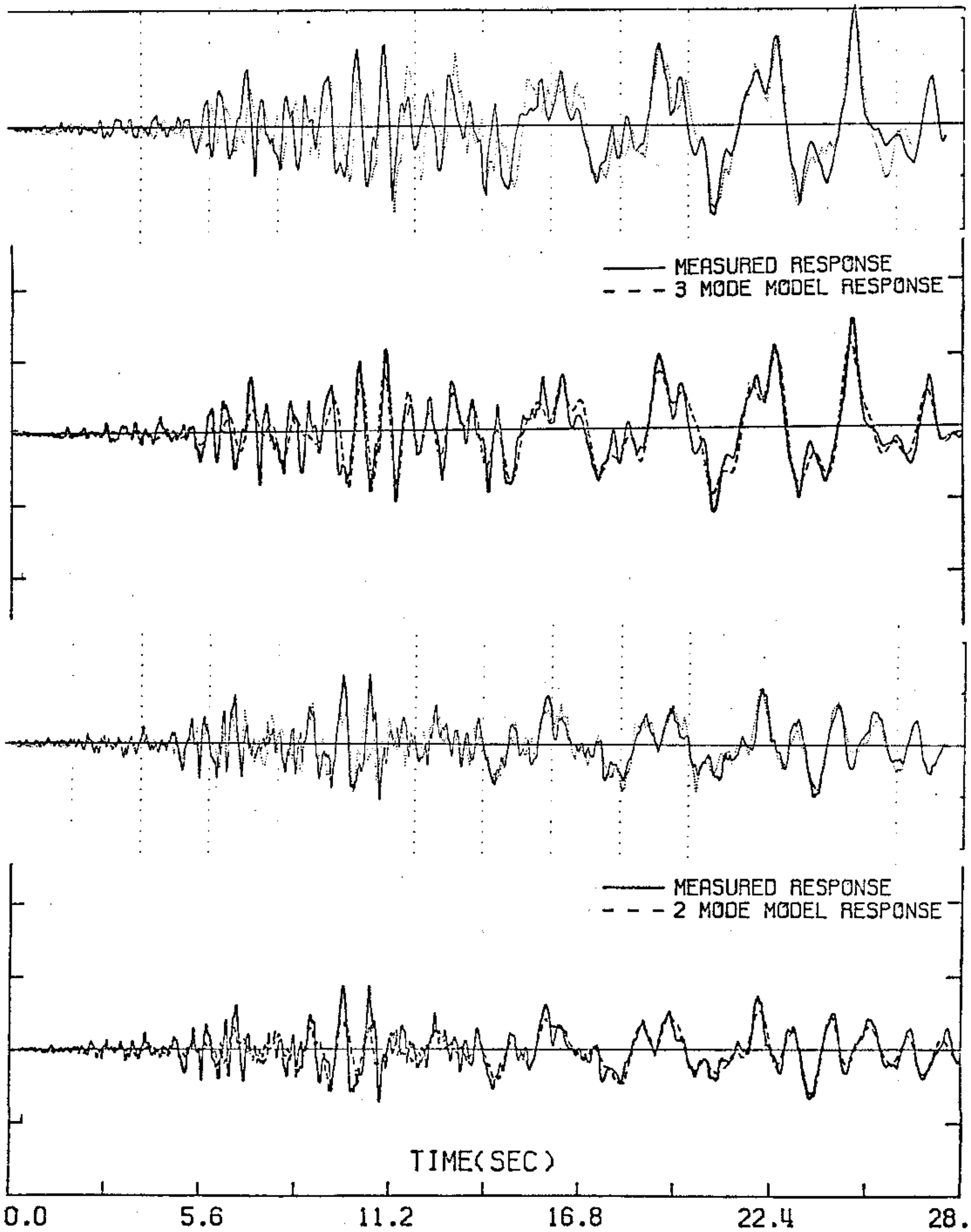


Figure 5.7 KB Valley Center, S81E acceleration responses. (a) Adjusted synthesized sixteen-mode model [6] roof response. (b) Optimal three-mode model roof response. (c) Adjusted synthesized sixteen-mode model 9th floor response. (d) Optimal two-mode 9th floor response.

incorrect period. The systematic identifications produced values of 8.6 and 8.8 percent damping from the two records, more in the range of expected values.

The identified fundamental periods increased by 50 and 60 percent over the pre-earthquake ambient vibration test values in the transverse and longitudinal directions, respectively. The corresponding increases found in the post-earthquake ambient tests were only 9 and 17 percent. The magnitude of the period lengthening during the earthquake was similar to that predicted by the original design models, which overestimated the periods in the two directions by only about 5 and 10 percent.

The need for systematic parameter identification from the earthquake records was well illustrated by the damping values of the various models. During the design, damping of 2 percent for all modes was assumed for calculating earthquake response. This value was shown to be overly conservative by the identified values of 8 to 9 percent for the first two transverse modes and approximately 6½ percent for the longitudinal modes. On the other hand, a trial-and-error parameter adjustment technique had difficulties in correctly estimating the parameters of the fundamental transverse mode, leading to an excessively high damping value of 20 percent.

5.3 KAJIMA INTERNATIONAL BUILDING, 250 EAST FIRST STREET

The Kajima International Building, described previously by Muto [11], Gates [5] and Foutch et al. [4], is a fifteen-story moment-resisting steel tower structure reaching a height of 202 feet, and with horizontal dimensions of 66 feet by 96 feet. The building, constructed in 1967, was designed under the provisions of the Los Angeles City Building Code. The maximum horizontal ground acceleration at the site, 21 miles south of Pacoima Dam (Fig. 1.1), was 0.14g with a maximum structural response of 0.21g. There was no structural damage, and only slight nonstructural damage consisting of plaster cracking. There was some evidence of impacting with the adjacent parking building.

The response acceleration histories show blips which increase the acceleration at each peak in the later part of the motion. Such blips could be caused by the building encountering increased resistance to the motion at the maximum displacement, as would occur either from impacting with an adjoining building or from the exterior spandrels coming into action and increasing the stiffness at the limits of the motion.

Frequency domain identifications have been performed using both horizontal components of the acceleration records obtained from the basement, the 8th floor and the roof. The results are listed in Table 5.3, along with the results of previous analyses of the structure.

The fundamental periods showed the usual increase between pre- and post-earthquake ambient vibration test, with even longer effective values during the earthquake response. The longitudinal (N36E) fundamental period more than doubled, from 1.32 seconds in the pre-earthquake test to an average value of 2.84 seconds during the earthquake, while the increase in the transverse period was less drastic, approximately 50 percent from 1.88 seconds to 2.77 seconds. Despite these great increases the effective periods were still considerably shorter than the values of 3.31 seconds and 3.19 seconds calculated during the design for the bare structural frame. The NOAA report [5] stated that

TABLE 5.3: MODAL PROPERTIES OF THE KAJIMA INTERNATIONAL BUILDING

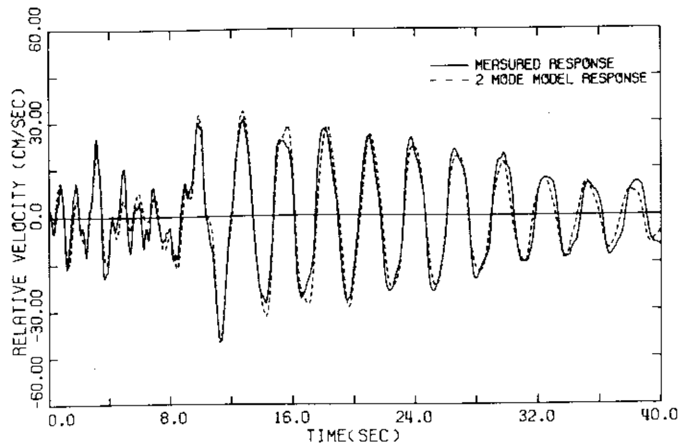
(a) Longitudinal N36E Direction

Source	Period (Seconds)			Damping (%)			Participation Factors					
	T_1	T_2	T_3	ζ_1	ζ_2	ζ_3	C_{1Roof}	C_{2Roof}	C_{3Roof}	C_{18th}	C_{28th}	C_{38th}
Ambient Tests [10]												
Pre-Earthquake	1.32											
Post-Earthquake	2.10											
Design Model [5]	3.31	1.16	0.688									
Adjusted design model [5]	2.91	1.02	0.603	5% all modes			1.33	-0.54		0.72	0.47	
Frequency Domain Identifications												
0-40.96 seconds Roof record	2.84	0.89	0.57	3.8	8.1	18.4(?)	1.10	-0.62	0.40			
8th floor record	2.84	0.90		3.8	8.2					0.54	0.60	
Equal Weighting Both Records	2.84	0.89		4.1	7.1		1.17	-0.53		0.58	0.59	
0-20.48 seconds Roof record	2.79	0.88	0.56	5.7	7.8	19.5(?)	1.23	-0.61	0.38			
20-40.48 seconds Roof floor record	2.91	0.95		3.0	2.1		1.33	-0.54				
8th floor record	2.90	0.95		2.8	2.4					0.38	0.61	

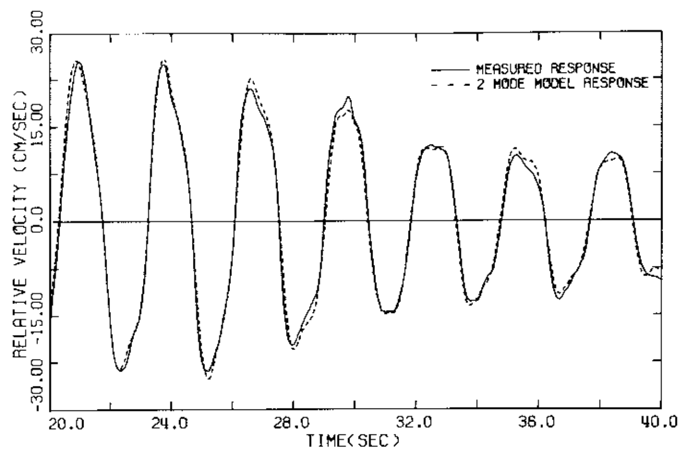
(b) Transverse N54W Direction

Source	Period (Seconds)			Damping (%)			Participation Factors					
	T_1	T_2	T_3	ζ_1	ζ_2	ζ_3	C_{1Roof}	C_{2Roof}	C_{3Roof}	C_{18th}	C_{28th}	C_{38th}
Ambient Tests												
Pre-Earthquake	1.88											
Post-Earthquake	2.15											
Design Model	3.19	1.09	0.640									
Adjusted Design Model	2.80	0.958	0.561	2% all modes			1.34	-0.52		0.81	0.46	
Frequency Domain Identification												
0-40.96 seconds Roof record	2.77	0.88	0.51	3.6	5.6	4.7	1.49	-0.56	0.31			
8th floor record	2.78	0.88		2.8	5.4					0.68	0.64	

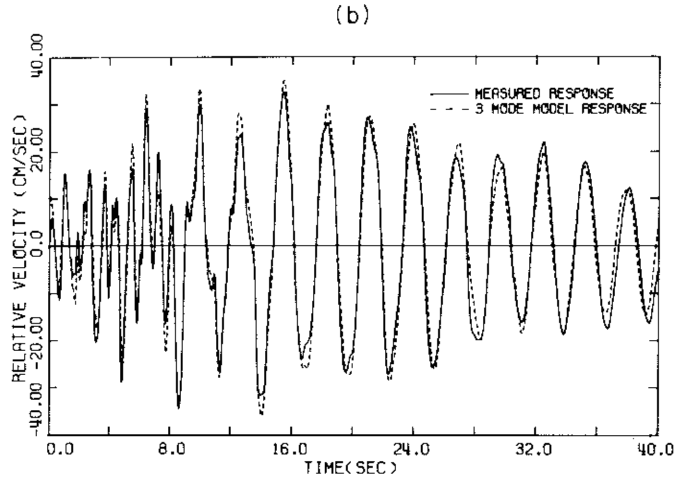
KAJIMA INTERNATIONAL BUILDING



(a)



(b)



(c)

Figure 5.8 Kajima building velocity responses. N36E component: (a) 0-40.96 seconds; (b) 20-40.48 seconds; (c) N54W component, 0-40.96 seconds.

large precast concrete spandrels provided significant nonstructural stiffness during small-amplitude vibrations; this may have accounted for much of the period shift.

The dampings associated with the period lengthenings were approximately 4 and 8 percent for the first two longitudinal modes and 3½ and 5½ percent for the transverse modes. The first longitudinal mode damping dropped from 5.7 percent over the first twenty seconds of the motion to 3.0 percent for the segment from 20 to 40 seconds. For comparison, the values selected for the adjusted synthesized models were 5 percent and 2 percent for all modes in the longitudinal and transverse directions, respectively.

Despite the period changes from the ambient tests, the linear models reproduced the response of this structure well. The estimates of the parameters from the response records at the midheight and roof locations were consistent. The normalized errors for the roof records for the full forty seconds were 0.17 and 0.10 for the longitudinal and transverse directions. The larger error in the longitudinal direction was caused by the greater period change in this component. Over the last twenty second segment the error dropped to 0.04. The quality of the matches of the response is shown by the velocity comparisons in Figure 5.8.

5.4 SHERATON-UNIVERSAL HOTEL, 3838 LANKERSHIM BOULEVARD

The twenty-story, reinforced concrete, ductile moment-resisting frame structure of the Sheraton-Universal Hotel [2] was completed in 1968. The seismic design provisions were essentially those of the 1970

Uniform Building Code. The structure was situated fifteen miles south of the center of the San Fernando earthquake and experienced a peak ground acceleration of 0.18g, with a peak response of 0.20g. Only slight damage was suffered, with repair costs amounting to only $2100 for the $7.5 million structure. Analysis with linear structural models indicated that yield stresses were not reached [2].

The results of pre- and post-earthquake ambient tests [10], the parameter values calculated from the design models [2], the values estimated from the Fourier spectra [7] and the estimates obtained from the frequency domain identification studies are given in Table 5.4. The periods listed for the design model were those calculated before the earthquake, but the ten percent damping was the rounded value chosen to best match the earthquake response.

The effective fundamental periods during the earthquake response were identified as 60 and 80 percent longer than the pre-earthquake test periods in the transverse north-south and longitudinal east-west directions, respectively. The fundamental periods of the design models were 12 percent longer than the effective period in the north-south direction and 7 percent shorter in the east-west direction. The periods identified from the first twenty seconds of the response were practically identical to those from the full forty second record length.

The 10 percent damping selected to provide a good correlation of the response of the design models with the measured response was higher than the values identified from the response. In the transverse direction the overall effective values were 7.3 and 8.3 percent for the first two modes. The effective first mode damping was estimated as

TABLE 5.4: MODAL PROPERTIES OF SHERATON-UNIVERSAL HOTEL

(a) Transverse NS Direction

Source	Period (Seconds)			Damping (%)			Roof Participation Factor		
	T_1	T_2	T_3	ζ_1	ζ_2	ζ_3	C_1	C_2	C_3
Ambient Tests									
Pre-Earthquake [10]	1.22								
Post-Earthquake [10]	1.35								
Earthquake Response									
Design model [2]	2.21	0.79	0.48	10% all modes			1.44	-0.74	
Fourier analysis [7]	2.13	0.66	0.25	4.9	5.0	4.3			
Identification 0-40.96 sec	1.98	0.55		7.3	8.3		1.41	-0.29	
0-20.48 sec	1.96	0.56	0.44	8.4	6.2	1.0	1.42	-0.23	-0.20

(b) Longitudinal EW Direction

Source	Period (Seconds)			Damping (%)			Roof Participation Factor		
	T_1	T_2	T_3	ζ_1	ζ_2	ζ_3	C_1	C_2	C_3
Ambient Tests									
Pre-Earthquake	1.26								
Post-Earthquake	1.47								
Earthquake Response									
Design model	2.15	0.74	0.45	10% all modes			1.30	-0.53	
Fourier analysis	2.27	0.72	0.30	4.1	8.4	8.4			
Identification 0-40.96 sec	2.24	0.67		6.2	11.6		1.16	-0.65	
0-20.48 sec	2.23	0.67	0.27	7.5	8.9	7.9	1.34	-0.52	-0.21

6.2 percent in the east-west direction, but this was associated with a participation factor of 1.16 compared to 1.30 for the design model. Scaling by the ratio of the participation factors produced a damping value of 6.8 percent.

In common with the other structures studied in this chapter, the linear models provided good matches of the measured responses. The normalized errors were 0.14 for both directions.

5.5 1900 AVENUE OF THE STARS

The building at 1900 Avenue of the Stars [13] is a steel moment-resisting frame structure rising twenty-seven stories above ground (Fig. 5.9). The 371 feet high tower has horizontal dimensions of 208 feet by 108 feet. The building, located twenty miles from Pacoima, experienced ground accelerations of 0.08g and a roof response of 0.14g during the San Fernando earthquake.

Table 5.5 contains the periods measured in vibration tests, estimated from the Fourier spectra of the earthquake, and identified by the frequency domain method. The periods effective during the earthquake response were 30 percent longer than those measured in the pre-earthquake tests. This lengthening was less than for most of the other structures studied, in line with the smaller excitation and response. The dampings were about 5 percent in the N44E direction, and 2.2 and 5.5 percent for the first two S46E modes.

The identifications were performed over a narrow frequency band of 0.07 to 0.90 Hz, including only the first two modes. The frequency domain fits over these intervals were very good, producing

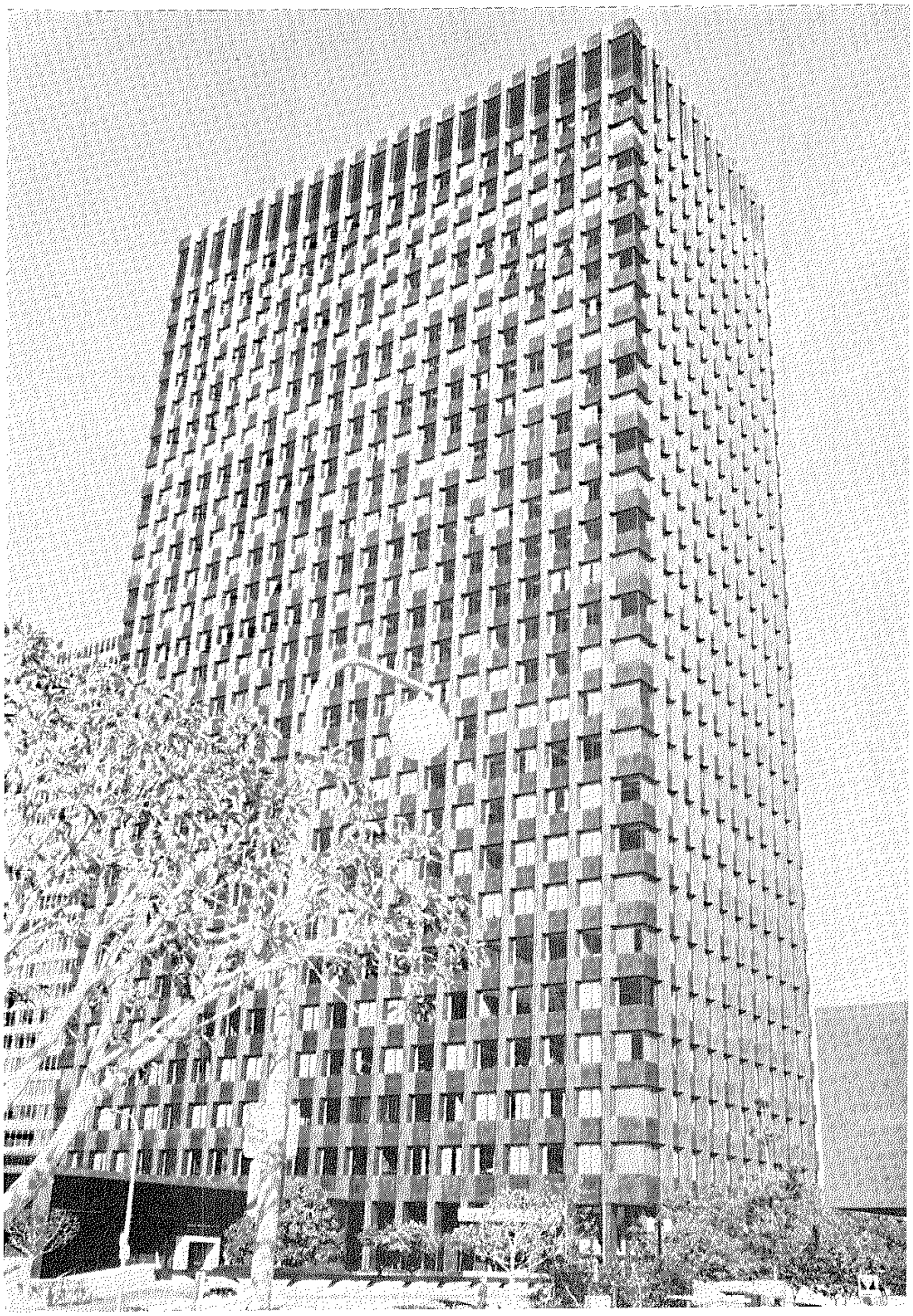


Figure 5.9 The 1900 Avenue of the Stars building.

TABLE 5.5: MODAL PROPERTIES OF 1900 AVENUE OF THE STARS STRUCTURE

(a) N44E Direction

Source	Period (Seconds)			Damping (%)			Roof Participation Factor		
	T_1	T_2	T_3	ζ_1	ζ_2	ζ_3	C_1	C_2	C_3
Ambient Tests									
Pre-Earthquake [10]	3.3								
Post-Earthquake [10]	3.66	1.17	0.68						
Earthquake Response									
Fourier spectra [7]	4.27	1.42		5.2	2.0				
Frequency domain identification	4.37	1.45		4.4	5.2		1.27	-0.59	

(b) S46E Direction

Source	Period (Seconds)			Damping (%)			Roof Participation Factor		
	T_1	T_2	T_3	ζ_1	ζ_2	ζ_3	C_1	C_2	C_3
Ambient Tests									
Pre-Earthquake	3.3								
Post-Earthquake	3.54	1.21	0.72						
Earthquake Response									
Fourier spectra [2]	4.26	1.41		6.5	2.5				
Frequency domain identification	4.24	1.47		2.2	5.5		1.07	-0.56	

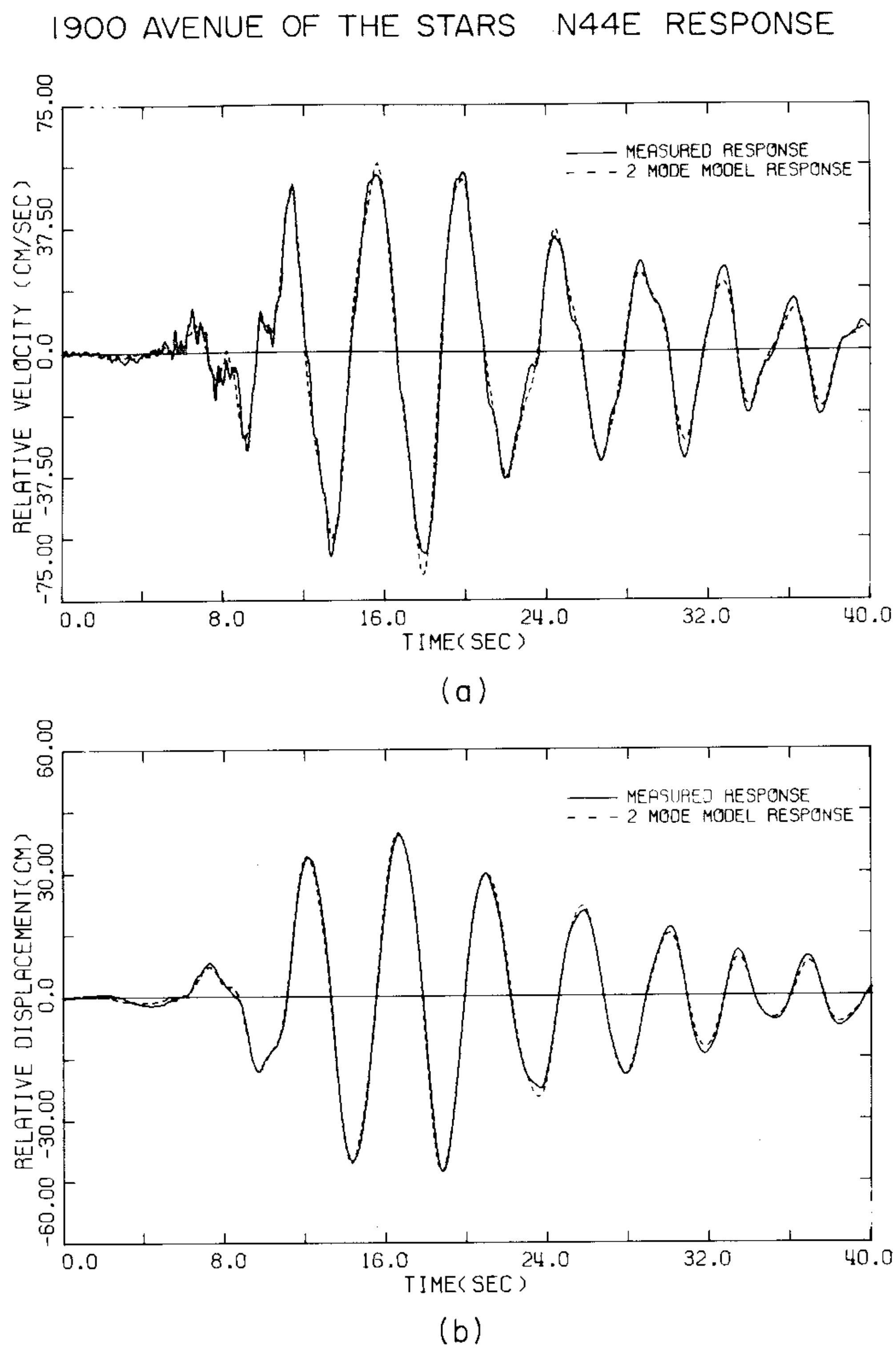


Figure 5.10 1900 Avenue of the Stars, N44E component. Comparison of the measured and optimal two-mode model response (a) Relative velocity. (b) Relative displacement.

measures-of-fit of 0.047 and 0.044 for the N44E and S46E directions, respectively. There was obvious higher frequency content in the acceleration transforms which was also evident in the matches of the acceleration time histories. This high frequency content was unimportant in the velocity and displacement responses. The time histories in the N44E direction, in particular, were very well matched, with the displacement fit virtually perfect (Fig. 5.10). The excellent match of the roof displacement for this high-rise structure by a two-mode model illustrates yet again the extreme dominance of the low modes in the earthquake response of this type of structure.

5.6 SUMMARY

In this chapter the responses of several high-rise buildings to the San Fernando earthquake have been considered. These records were typical of many obtained during the earthquake, and were of moderate amplitude motions suitable for analysis with linear models.

The buildings ranged in height from the 16-story KB Valley Center to the 42-story Union Bank building. All but the reinforced concrete Sheraton-Universal Hotel were steel-framed structures. The ground accelerations experienced were typically about 0.15g, with responses generally between 0.20 and 0.25g. None of the buildings suffered structural damage, although all exhibited modal periods and dampings significantly greater than in vibration tests.

The systematic identification technique provided reliable estimates of the parameters of several modes of the structures, not just the fundamental. For most of the buildings, the parameters of the three lowest modes were estimated. The parameters of four modes were estimated from both components of the midheight response of the Union Bank building. For the transverse direction, this provided information about the first, second, fourth and sixth modes. Nodes at the third and fifth modes near the measurement location were indicated by their absence from the identified model.

The identification studies confirmed that the type of moderate response recorded in these structures could be reproduced well by linear models with only a few modes, apart from high-frequency content in some of the acceleration records, and obviously spurious long-period content in some of the displacement histories calculated from the measured accelerations. The optimal models identified for these structures from the acceleration transforms provided good matches of the detailed velocity and displacement histories, not just matches of the largest amplitude portion of the response as obtained for some records studied in previous chapters. For the 27-story building at 1900 Avenue Avenue of the Stars, in which the excitation of 0.08g and the response of 0.14g were about half those of the other buildings, the velocity and displacement matches obtained with a two-mode model were virtually perfect for the N44E component.

For these buildings, the models synthesized during design predicted the periods effective during the earthquake response reasonably well, generally within ten percent. The periods derived from the design models were certainly much closer to the periods identified from the response than were those measured in ambient tests before the earthquake. The earthquake periods ranged from a third longer to over double the pre-earthquake ambient periods, while the worst estimate provided by the design models was 15 percent too long.

Several of the buildings considered in this chapter were studied by Hart [7,8] using Fourier spectra and transfer function analyses. It was expected that the fundamental periods estimated by these techniques would be in near-perfect agreement with those from the least-squares frequency domain match. Somewhat surprisingly, Hart's values for the fundamental periods in the transverse direction for the Sheraton-Universal Hotel and in the longitudinal direction for the KB Valley Center were both about seven percent longer than those identified in this study. The more common problems in Hart's type of analysis are estimating the dampings and selecting the peaks corresponding to the higher mode periods from several candidates. The uncertainty in the higher mode periods was illustrated by the difference of typically 15 to 20 percent between the values estimated by Hart and by the present systematic frequency-domain identification technique. Most of the damping estimates obtained by Hart varied considerably from those obtained here, even for the fundamental modes.

The values of the first mode damping identified from the earthquake response of these structures ranged from 2.2 percent in the S46E direction for the 1900 Avenue of the Stars building, which experienced excitation and response amplitudes about half those of the other structures, to 8.8 percent in the transverse direction for the KB Valley Center. The Union Bank and Kajima Buildings exhibited dampings around 4 to 5 percent, and the reinforced concrete Sheraton-Universal Hotel had values of 6 and 7½ percent in the two directions. This range of values is believed to be representative of that expected in tall framed structures during response to strong, but not structurally damaging, earthquake excitation.

REFERENCES

1. Beck, J.L., "Determining Models of Structures from Earthquake Research", Earthquake Engineering Research Laboratory Report No. EERL 78-01, California Institute of Technology, Pasadena, California, June 1978.

2. Blume, John A. and Associates, "Sheraton-Universal Hotel, 3838 Lankershim Boulevard, Los Angeles", in 'San Fernando, California, Earthquake of February 9, 1971', L.M. Murphy (ed.), Vol. I, Part A, 307-326, U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration (NOAA), Washington, D.C., 1973.

3. California Institute of Technology, "Strong Motion Accelerograms", Earthquake Engineering Research Laboratory, Pasadena, California.

4. Foutch, D.A., G.W. Housner and P.C. Jennings, "Dynamic Responses of Six Multistory Buildings during the San Fernando Earthquake", Earthquake Engineering Research Laboratory Report No. EERL 75-02, California Institute of Technology, Pasadena, California, October 1975.

5. Gates, W.E., "Kajima International Building, 250 E. First Street, Los Angeles", in 'San Fernando, California, Earthquake of February 9, 1971', L.M. Murphy (ed.), Vol. I, Part B, 509-39, U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration (NOAA), Washington, D.C., 1973.

6. Gates, W.E., "K.B. Valley Center, 15910 Ventura Boulevard, Los Angeles", in 'San Fernando, California, Earthquake of February 9, 1971', L.M. Murphy (ed.), Vol. I, Part 8, 449-80, U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration (NOAA), Washington, D.C., 1973.

7. Hart, G.C., R.M. DiJulio and M. Lew, "Torsional Response of High-Rise Buildings", Journal of the Structural Division, Proc. ASCE, Vol. 101, ST2, 397-410, February 1975.

8. Hart, G.C. and R. Vasudevan, "Earthquake Design of Buildings: Damping", Journal of the Structural Division, Proc. ASCE, Vol. 101, ST1, 11-29, January 1975.

9. Martin, A.C. and Associates, "Union Bank Square, 445 South Figueroa Street, Los Angeles", in 'San Fernando, California, Earthquake of February 9, 1971', L.M. Murphy (ed.), Vol. I, Part B, 575-546, U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration (NOAA), Washington, D.C., 1973.

10. Mulhern, M.R. and R.P. Maley, "Building Period Measurement Before, During, and After the San Fernando Earthquake", in 'San Fernando, California, Earthquake of February 9, 1971', L.M. Murphy (ed.), Vol. I, Part B, 725-33, U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration (NOAA), Washington, D.C. 1973.

11. Muto, K., "Strong Motion Records and Simulation Analysis of K11 Building in San Fernando Earthquake," Muto Institute of Structural Mechanics, Report 71-2-1, Tokyo, Japan, September 1971.

12. Trifunac, M.D., "Ambient Vibration Test of a Thirty-Nine Story Steel Frame Building", Report EERL 70-02, California Institute of Technology, Pasadena, California, July 1970.

13. Trifunac, M.D. and B. Turner, "Accelerograph Site Data: San Fernando Earthquake of 1971. Volume 3", Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, California 1976 (unpublished report).

14. Udwadia, F.D. and M.D. Trifunac,"Ambient Vibration Tests of Full-Scale Structures",Proc. 5th World Conf. on Earthquake Engineering, Vol. 2, 1430-39, Rome, Italy,1974.

VI: CONCLUSIONS

A systematic technique has been developed in this thesis for identifying the parameters of linear models of structures from their recorded earthquake excitation and response. The main objective of the work has been to determine the values of the modal parameters of linear mathematical models of structures which best reproduce the responses recorded in a variety of buildings during different levels of earthquake motion.

The identifications have been performed by analysis in the frequency domain because of a desire to improve unsatisfactory results obtained by methods using transfer functions. The natural approach in the frequency domain is to estimate the modal parameters of the structural model. This proves convenient because earlier work by Beck [1] has shown that the parameters of linear models of structures which can be estimated uniquely and accurately from the earthquake base motion and response are indeed the parameters of the dominant modes in the response, rather than the elements of the damping and stiffness matrices.

The identification technique produces the values of the modal parameters which achieve a least squares match over a specified frequency band between the unsmoothed, complex-valued, finite Fourier transform of the acceleration response recorded in the structure and that calculated for the model. The identification can be performed using either the full lengths of the earthquake records or portions of the records. Using segments of the records permits tracing the variation of the effective linear parameters during the response.

TABLE 6.1: SUMMARY OF IDENTIFICATION STUDIES

Structure and Component		Stories	Construction	Miles From Pacoima Dam*	Max. Acc. (%g)		Fundamental Period (Sec)				1st Mode Damping (%)		Response Type
					Ground	Response	Pre-Eq.	Post-Eq.	Eq.	Eq/Pre-Eq.	Eq.	Tests	
JPL 180 - Borrego Mountain	S82E	9	Steel Frame	15	0.7	3.1	0.91		1.09	1.2	2.9		A
	S08W				0.7	2.3	0.88		1.15	1.3	2.7		
- Lytle Creek	S82E				1.5	2.5	0.91		1.02	1.12	4.7		A
	S08W				2.4	3.7	0.88		1.13	1.3	3.5		
- San Fernando	S82E				21	37	0.91	1.01	1.26	1.4	3.8		C
	S08W				14	21	0.88	1.16	1.42	1.6	6.4		
Millikan - Lytle Creek	NS	9	R.C. Shearwall	19	1.9	5.4	0.53		0.52	0.98	2.9	1-2	A
	EW				1.9	3.5	0.69		0.71	1.03	2.2	0.7-1.7	
- San Fernando	NS				20	31	0.53	0.54	0.62	1.17	6.4		C
	EW				18	34	0.69	0.79	0.98	1.4	7.0		
1900 Avenue of Stars	N44E	27	Steel Frame	20	8	14	3.3	3.6	4.37	1.3	4.4		A-B
	S46E				8	11	3.3	3.6	4.24	1.3	2.2		
Union Bank	S38W	39	Steel Frame	21	12	12	3.11	3.7	4.63	1.5	4.9	1.5	B-C
	N52W				15	20	3.53	4.1	4.71	1.3	4.1	1.7	
KB Valley Center	S09W	18	Steel Frame	14	13	22	2.18	2.37	3.30	1.5	8.6		B
	S81E				14	22	1.94	2.27	3.05	1.6	6.3		
Kajima	N36E	15	Steel Frame	21	9	19	1.32	2.10	2.84	2.2	3.8		B
	N54W				12	17	1.88	2.15	2.77	1.5	3.6		
Sheraton-Universal	NS	19	R.C. Frame	15	16	11	1.22	1.4	1.98	1.6	7.3		B
	EW				15	18	1.26	1.5	2.24	1.8	6.2		
Holiday Inn - Marengo	S52W	7	R.C. Frame	22	13	41	0.49	0.63	1.17	2.4	5.0(?)		D
	N38W				12	23	0.53	0.64	1.06	2.0	17.8		
Holiday Inn - Orion	NS	7	R.C. Frame	8	25	38	0.48	0.68	1.42	3.0	19.2		D
	EW				13	31	0.52	0.72	1.20	2.3	17.3		
Bank of California	N11E	12	R.C. Frame	14	22	28	(0.85)	1.70	2.35	2.8	12.1		D
	N79W				15	24	(1.33)	1.60	3.01	2.3	10.0		

*Taken as the center of energy release in the San Fernando earthquake.

Response types:
- A - Small amplitude, excellent matches, close to vibration test periods.
- B - Moderate amplitude, good to excellent matches, periods changed from vibration tests.
- C - Variation of linear parameters with time, strongest response matched by time-invariant modes.
- D - Minor structural damage, limit of linear models.

There are several important features of the method used in the identification. The first is that the effect of the initial and final conditions of the segment of record used for the identification are considered, leading to the inclusion of the differences between the initial and final modal velocities and displacements as parameters of the model to be estimated from the records, along with the natural periods, dampings and effective participation factors. The inclusion of these parameters is particularly important for segment-by-segment analysis. Often the response in later parts of the record is mainly decay in free vibration of the motion caused by earlier excitation, so the initial conditions must be determined to obtain accurate estimates of the other parameters.

Many of the previously used techniques for identifying structural parameters use only the measured response of the structure and ignore the excitation. The present method uses the recorded base motion as the excitation for the model. This makes the technique more efficient and produces more accurate estimates of the parameters and matches of the response than methods which use the response records alone, along with assumed characteristics of the excitation. These methods generally require longer segments of the records to perform the identification and often require assumptions about the nature of the motion which are erroneous for earthquake excitation and response. Again, this feature is important for segment-by-segment analysis, as the use of shorter segments allows more nearly "instantaneous" values of the parameters to be estimated.

Finally, it is noted that the method uses the entire Fourier transform, both amplitude and phase, rather than just the modulus. In addition, smoothing of the transforms is not required and would, in fact, reduce the accuracy of the identified parameters. Smoothing reduces the amplitudes and increases the bandwidths at the modal frequencies, leading to overestimations of the damping values.

Many of the identifications given in previous chapters have included sensitivity analyses which provide information on the relative accuracy of the estimates of the different parameters. For the levels of damping exhibited in the earthquake response of structures, the measure-of-fit is much more sensitive to the periods than the dampings and participation factors. This means that the periods are the easiest parameters to identify, as a rule. Generally, the accuracy of all the estimates decreases for higher modes, but this depends partly on the degree of excitation of the modes and the nearness of the recording station to a node. The parameters of a strongly excited higher mode may be estimated more accurately than those of a less responsive lower mode. Similarly, it helps to have records from stations near maxima of mode shapes, as proximity to a node of a mode diminishes the contribution of that mode to the total response at the station.

One unfortunate feature which is revealed by both the estimated parameter values and the sensitivity analyses is that the estimates of the dampings and participation factors are often coupled. Their ratio, which determines the modal amplifications, is estimated much more reliably than the values of the individual parameters. This coupling, which appears to be an inherent feature of earthquake response, is disappointing, as the values of the dampings for different levels of response are perhaps the results of the identifications which are most required. Consequently, several strategies have been applied to overcome the interactions between these estimates.

When response records are available from only one location in a structure, the participation factors can be constrained to their initial estimates and the identification performed with respect to the other parameters. Depending on the amount of information available about the structure, the estimates of the participation factors can be derived from synthesized design models, calculated from mode shapes measured in vibration tests, or assumed on the basis of a uniform shear beam or some other simple model. The justification for this strategy is that the participation factors appear much less sensitive to changes in the structure than the periods and dampings. In particular, if the period changes are caused by proportional stiffness changes over the structure, the participation factors remain unaltered. The loss of information about the participation factors inherent in the constrained identification approach is thought to be of little practical significance. The records for which the participation factors may be expected to change most are those with the greatest changes in the other structural properties as indicated by the amount of period shift. These records generally produce unrealistic values of the participation factors in unconstrained identifications because of problems of matching segments of the nonlinear response with time-invariant, linear models. Consequently, it is felt justifiable to sacrifice unrealistic information about the participation factors to attempt to achieve better estimates of the effective dampings. The damping values found from the constrained identifications are generally in agreement with those obtained simply by scaling damping estimates from unconstrained identifications by the ratio of the initial to the identified values of the participation factors.

When response records are available from two locations in the structure, a simultaneous identification may be performed. The physical constraint that the records come from the same structure implies that only one damping value per mode can be estimated from the two records. Sensitivity analyses show that the coupling is generally much reduced by this approach. Consequently, more reliable estimates of the dampings and participation factors result. This approach appeared very promising, but it was used mainly for the records in which nonlinear behaviour was most pronounced, so it was not always successful because linear models were incapable of providing accurate matches of the responses. It appears, as expected, to be the appropriate method to use for moderate amplitudes of response which can be reproduced well by linear models.

In fulfilling the aim of investigating the response records from a variety of structures over a range of excitation levels, ten structures have been considered in this study (Tables 1.1 and 6.1). Most of the records are from the San Fernando earthquake. For two of the structures, Millikan Library and JPL Building 180, response records from more distant earthquakes with maximum ground accelerations about a tenth of those recorded at the same sites during the San Fernando earthquake have also been analyzed. Another comparison of the effects of different excitations on the structural response was provided by the two Holiday Inn buildings which are virtually identical structures. The buildings ranged in height from the seven-story Holiday Inn to the forty-two story Union Bank. Maximum ground accelerations ranged from 0.007g in JPL Building 180 during the Borrego Mountain earthquake to 0.25g in the Orion Avenue Holiday Inn during the San Fernando earthquake, with associated responses from 0.023g to approximately 0.4g.

In Table 6.1, the level of the responses has been indicated by the maximum ground and response accelerations and the ratio of the effective fundamental period during the earthquake to that in pre-earthquake ambient tests. For all but the lowest amplitude earthquake responses, the effective periods were considerably longer than the vibration test values. The post-earthquake ambient periods are also listed, to indicate that they reflect only a portion of the change that occurred in the earthquake. The first mode dampings identified from the earthquake records are given, together with the few values available from vibration tests. The vibration test dampings are in many cases only a small fraction of the damping effective in the earthquake response. At the very low vibration levels of ambient tests, the structural characteristics seemed to be influenced markedly by components which have little effect at the amplitudes of earthquake response. Accordingly, the period and damping values determined from ambient tests are not easily extrapolated to the much larger amplitudes of strong earthquake response.

In the final column of Table 6.1, the responses are classified into four categories, according to the nature of the effective linear models obtained in the identification studies. The types of response of each of these classes is summarized below.

Class A response was of small amplitude, with ground accelerations less than 0.03g and responses below 0.06g. The effective fundamental periods were close to those determined in vibration tests. Only the Borrego Mountain and Lytle Creek responses fell into this category. The response was reproduced very well by linear models for this level of vibration. Fundamental mode dampings were less than 5 percent of critical, generally around 3 percent.

Class B response was of moderate amplitude, typical of that of many of the instrumented buildings which suffered no structural damage during the San Fernando earthquake. Maximum ground accelerations were typically 0.15g, with responses around 0.20g. Linear models reproduced the response very well, but the effective periods of earthquake response were significantly different from those measured in vibration tests, typically 50 percent longer. The identified values of the first mode dampings varied from 2.2 percent to 8.6 percent, with an average value of 5 percent. The changes from the parameter values effective during the vibration tests to those exhibited during the earthquake response occurred rapidly, often in too short a time to allow the variation to be traced by segment-by-segment analysis. The optimal models identified for these structures provided good matches of the details of the velocity and displacement histories, not just matches of the largest amplitude portion of the response as obtained for the stronger motion of the class C records. The responses considered in chapter V fell into class B.

Class C responses were those in which the variation of the effective linear parameters as a function of time could be traced. Typically the periods lengthened from the vibration test values at the beginning of the motion to the longest values effective at the time of maximum response, and then remained fairly constant. The dampings were highest during the maximum response, and then became smaller later in the motion. The parameter values identified from the whole response records generally reflected the values effective during the largest amplitudes of response. Presumably, this is a consequence of the quadratic measure of difference which emphasizes the largest amplitudes. Consequently, the overall model response histories matched the strongest portions of the measured motions reasonably well, but, because of the over-estimation of the damping for the later parts of the records, the responses of the models decayed faster than the measured responses. The typical maximum ground accelerations for these records were 0.20g, with responses of 0.30g. The amount of period increase was similar to the class B responses, while the average first mode damping was near 6 percent. The responses during the San Fernando earthquake of JPL Building 180 and Millikan Library were in category C.

The final category, class D, includes those structures which suffered minor structural damage. This type of response was the limit of applicability of linear models. A single linear model was unable to reproduce the entire response history for this type of structural behavior, although segment-by-segment analysis produced some good matches and allowed the changes in the parameters to be traced approximately. The markedly nonlinear behavior led to difficulties in estimating the dampings and participation factors at this level of response. For the structures studied which fell into this class, the effective overall fundamental periods ranged from double to triple the test periods. The maximum dampings were typically in the 15 to 20 percent range. The two Holiday Inns and the Bank of California comprised this class.

The identification studies have shown that linear models with the appropriate parameters can reproduce the detailed histories in type A and B responses, and the highest amplitude portion of the motion in type C responses. Only the type D motions, in which structural damage had occurred, could not be represented satisfactorily by time-invariant linear models.

Although this summary has concentrated on the fundamental mode periods and dampings for the different categories, it must be emphasized that the properties of several modes can be estimated by the systematic identification technique. It is an important result of the identification studies that only a few modes were required to reproduce the measured responses. The only contribution of modes higher than about the sixth in any of these responses was high frequency content in some of the acceleration records, which was not apparent in the smoother velocity and displacement responses. For most of the records, the responses were reproduced very well by models containing only two or three modes. It is this need to consider only a very few modes which makes the modal approach attractive for analyses used in design.

One of the aims of the identification studies was to compare the optimal linear models with the synthesized models used either during the design of the structures or in post-earthquake analyses. It was also interesting to compare the results of the systematic identifications with those from transfer function approaches.

The models identified by systematic identification techniques produced matches of the detailed response histories markedly superior to those obtained by most synthesized design models and generally considerably better even than those given by models adjusted by trial-and-error. Methods using trial-and-error adjustment of parameters often match response amplitudes reasonably well and are therefore useful for many purposes. However, the calculated responses have difficulty staying in phase with the measured motions. The importance of the improved matches obtained by systematic techniques are that they confirm the validity of linear models in cases where the matches achieved by other methods show enough discrepancies to raise the possibility that nonlinear mechanisms may be required to represent accurately the structural behavior.

The better matches achieved by the systematic methods are generally associated with greater and more reliable information about the structure. For example, the present study resolved a problem with a questionably high damping value of 20 percent obtained for the fundamental transverse mode of KB Valley Center in a trial-and-error parameter adjustment analysis [2]. The systematic technique showed that the period had also been estimated poorly in the earlier study. When the correct period was used, the estimated damping fell to 8½ percent.

For design purposes, the synthesized models provided periods sufficiently close to the values identified from the records, generally within ten percent, that the maximum response quantities calculated from reasonably smooth design spectra would be essentially the same. There are problems, however, in selecting the dampings. The identification studies showed that the approximate range of damping values for the non-damaging response of tall steel-framed tower structures is four to eight percent, with an average near five percent for the undamaged structures considered in this study. Values as low as two percent appear appropriate only for very low amplitude responses such as those recorded in the Lytle Creek and Borrego Mountain earthquakes. This result is only for one type of building at one level of response and more studies are required to derive a correlation of the expected damping to the properties of the ground excitation and the structural properties of the building.

There are several interesting and important areas for the further development and application of techniques for structural identification from earthquake records. There are several possibilities for study with linear models, besides the straightforward application of the present techniques to more records to obtain further information about dampings and periods.

Since the beginning of the present study, the Santa Barbara earthquake of August 13, 1978 [5,6] and the very recent Coyote Lake earthquake of August 6, 1979 [4] have provided records from several locations in the same structure. These records offer the opportunity to obtain more detailed mode shape determinations than are possible from the previously available records, and also to compare the response for different locations on the same floor.

There are also records available from dams and bridges which may require extension of the models to allow multiple inputs. These records may also be fruitful for examination by non-planar models.

For the Ferndale earthquake of June 7, 1975, there are records available at the Humboldt Bay Power Plant from the buried base of the caisson structure, in the caisson on the surface, and at a "free-field" surface site [8]. These records are worthy of analysis to investigate soil-layer amplification (or attenuation) effects and soil-structure interaction.

The field of structural identification with nonlinear models requires much more research. The Orion Avenue Holiday Inn and the Bank of California records that have been considered in this study require analysis using nonlinear models. At present, the functional form of the equations of motion which may be appropriate are unknown, but it seems that both hysteretic and degrading stiffness effects must be included for realistic results. Some research to determine the appropriate models for the nonlinear behavior of structures has been done at Berkeley in the identification of models for simple structural frames subjected to earthquake-like excitations on a shaking table [3,7]. The experience from the shaking table studies should prove valuable when actual earthquake records become available for substantially inelastic motions, approaching structural failure.

In summary, systematic identification studies have shown that the behavior of structures during earthquake response can be represented very well by linear models for amplitudes up to the onset of structural damage. The use of systematic techniques allows optimal models to be identified which generally reproduce the response much better than models derived by trial-and error methods. The close duplications of the measured responses allows more model parameters to be found and gives more confidence in the accuracy of the estimates of the parameters of the dominant modes.

REFERENCES

1. Beck, J.L., "Determining Models of Structures From Earthquake Records", Earthquake Engineering Research Laboratory Report No. EERL 78-01, California Institute of Technology, Pasadena, California, June 1978.

2. Gates, W.E., "KB Valley Center, 15910 Ventura Boulevard, Los Angeles", in 'San Fernando, California, Earthquake of February 9, 1971', L.M. Murphy (ed.), Vol. I, Part B, 447-80, U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration (NOAA), Washington, D.C., 1973.

3. Kaya, I. and H.D. McNiven, "Investigation of the Nonlinear Characteristics of a Three-Story Steel Frame Using System Identification", Earthquake Engineering Research Center Report UCB/EERC 78-25, University of California, Berkeley, California 1978.

4. Mattheisen, R.B., "Preliminary Observations on the August 6, 1979 Coyote Lake, California, Earthquake", presented at the 2nd U.S. National Conf. on Earthquake Engineering, Stanford University, Palo Alto, California, August 22-24, 1979.

5. Miller, R.K., and S.F. Felszeghy, "Engineering Features of the Santa Barbara Earthquake of August 13, 1978", University of California, Santa Barbara, Report UCSB-ME-78-2, December 1978.

6. Porter, C.D., J.T. Ragsdale and R.D. McJunkin, "Processed Data from the Partial Strong-Motion Records of the Santa Barbara Earthquake of 13 August, 1978. Preliminary Results 1979", California Division of Mines and Geology, Preliminary Report 23.

7. Stanton, J.F. and H.D. McNiven, "The Development of a Mathematical Model to Predict the Flexural Response of Reinforced Concrete to Cyclic Loads, Using System Identification", Earthquake Engineering Research Center Report UCB/EERC 79-02, University of California, Berkeley, California 1979.

8. Valera, J.E., H.B. Seed, C.F. Tsai and J. Lysmer, "Soil-Structure Interaction Effects at the Humboldt Bay Power Plant in the Ferndale Earthquake of June 7, 1975", Earthquake Engineering Research Center Report UCB/EERC 77-02, University of California, Berkeley, California, January 1977.