PB 293206

SYSTEMS, SCIENCE AND SOFTWARE

SSS-R-73-1733

DYNAMIC RESPONSE OF A HIGH-RISE BUILDING TO THEORETICALLY PRODUCED EARTHQUAKE GROUND MOTION

Topical Report

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IR&D 9041-4324

June 1973

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50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF-RA-E-73-557	2.	3. Recipient's Accession No. PB 293 206
4. Title and Subtitle: Dynamic Response of A High-Rise Building to Theoretically Produced Earthquake Ground Motion, Topical Report (SSS-R-73-1733)			5. Report Date: June 1973
			6.
7. Author(s): J. Sweet			8. Performing Organization Rept. No.: SSS-R-73-1733
9. Performing Organization Name and Address: Systems, Science and Software, P. O. Box 1620, La Jolla, California 92037			10. Project/Task/Work Unit No.: IR&D 9041-4324
			11. Contract(C) or Grant(G) No. (C) (G)
12. Sponsoring Organization Name and Address: Applied Science and Research Applications (ASRA), National Science Foundation, 1800 G Street, N.W., Washington, D.C. 20550			13. Type of Report & Period Covered
			14.

15. Supplementary Notes

16. Abstract (Limit: 200 words)

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17. Document Analysis a. Descriptors

Earthquakes
Seismology
Dynamic structural response
Earthquake resistant structures
Elastic analysis
Buildings

b. Identifiers/Open-Ended Terms

Earthquake engineering

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
NTIS		39
	20. Security Class (This Page)	22. Price A03-A01

(See ANSI-Z39.18) See Instructions on Reverse OPTIONAL FORM 272 (4-77) (Formerly NTIS-35) Department of Commerce

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ACKNOWLEDGEMENTS

The author wishes to express his appreciation to Dr. J. T. Cherry for many helpful suggestions during the course of this work. In addition, thanks are also due to Mr. E. J. Halda for providing ground motion records and to Mr. K. G. Hamilton for the spectral analyses.

ABSTRACT

The structural analysis of a high-rise building subject to a theoretically produced earthquake is presented. The analysis has been performed with the building situated at three different locations relative to the earthquake epicenter. The building has also been excited using the 1940 El Centro earthquake record. The use of theoretical earthquake models provides local seismic loading levels for use in structural design. Results have been obtained using spectral analysis, modal superposition, and time-stepping.

I. INTRODUCTION

The aseismic design of civil engineering structures must take into consideration the realistic specification of expected ground motion as well as proper structural design practices. With the dynamic analysis of structures now being a standard design procedure,[1-5] the ability to predict the seismic loading at a particular building site should be advanced. This is especially important in light of recent modifications to local building codes[6] which require, along with other changes, a dynamic analysis to be performed on all "critical structures." This emphasis on dynamic analysis is sure to become more widespread in the future.

The accepted procedure in specifying the ground motion to be used in a dynamic structural analysis is to utilize a previously recorded earthquake strong motion record (the 1940 El Centro record[7] is a very popular choice). Alternatively, the ground motion may be specified using statistical techniques based on past earthquake records[8,9] or by applying purely analytical methods.[10,11] The ground motion at a particular building location is influenced by many factors. Some of the most important are: distance from earthquake epicenter, type of faulting, and local geologic configuration. These effects not only make it difficult to predict possible earthquake ground motion, their unique characteristics also make the use of ground motion records from a different region questionable. A case in point is the recent San Fernando earthquake.[12] It not only had peak accelerations at some locations (Pacoima Dam) as high as 1 g, but it also resulted from slippage on a thrust fault without a previous recent history of seismicity.

The possibility of a relatively high ground-shaking intensity in a region of low recorded seismic behavior points to the need to utilize analytical earthquake models. For, while the San Fernando fault was not extensively studied, its existence was known[12] and could have lent itself to mathematical modeling. The two-dimensional stick-slip earthquake source model developed by Cherry[13] has been used to produce the near source, free surface ground motion used in this study. This model has been incorporated into a two-dimensional finite difference computer code and treats linear or nonlinear analyses with equal accuracy.

The particular building selected for earthquake response analysis was the ten-story Bank of Nevada building. The response of this building to underground explosion-induced ground motion has been discussed in detail by Tokarz and Bernreuter.[14] In the two-dimensional plane analyzed in this study the building behaves in a moment resisting frame mode. A finite element model of this structure has been developed using the SAP[15] computer code. In addition to the aforementioned analytical ground motion loading, the building response to the 1940 El Centro record is presented for comparative purposes. The dynamic response of the building has been determined using both the modal superposition and time stepping options of the SAP code. In addition, maximum amplitudes are derived by applying response spectra techniques.

II. BUILDING DESCRIPTION

The building under consideration is the ten-story Bank of Nevada building in Las Vegas. A schematic of this building is found in Fig. 1. The two-dimensional analysis presented in this study represents the behavior of the building to seismic forces in its longitudinal direction. The three steel frames in this direction result in a moment resisting frame structural behavior. The 4-1/2 inch reinforced concrete floor slabs are sufficiently rigid to assure that the three frames displace similarly. In the transverse direction the forces are resisted primarily by two shear walls at each end of the building.

The finite element model of the building is depicted in Fig. 2. The structure is assumed fixed at the ground level and soil-structure interaction is ignored. The weights of the individual stories appearing in this figure are assumed to be concentrated at the roof and floor levels. In addition, the column-girder connections are assumed to be rigid. The area and moment-of-area values for each of the columns and girders are found in Table I. The girder stiffness was calculated using all of the reinforced concrete floor slab.

The stiffness values in Table I are determined by summing those values in the tower portion of the building and do not include all of the parking garage effects. A mathematical model that better reproduces experimental data can be derived by increasing the stiffness in the lower stories. However, the present model is utilized since it is more representative of typical high-rise buildings. A more detailed description of the Bank of Nevada building can be found in Tokarz and Bernreuter.[14]

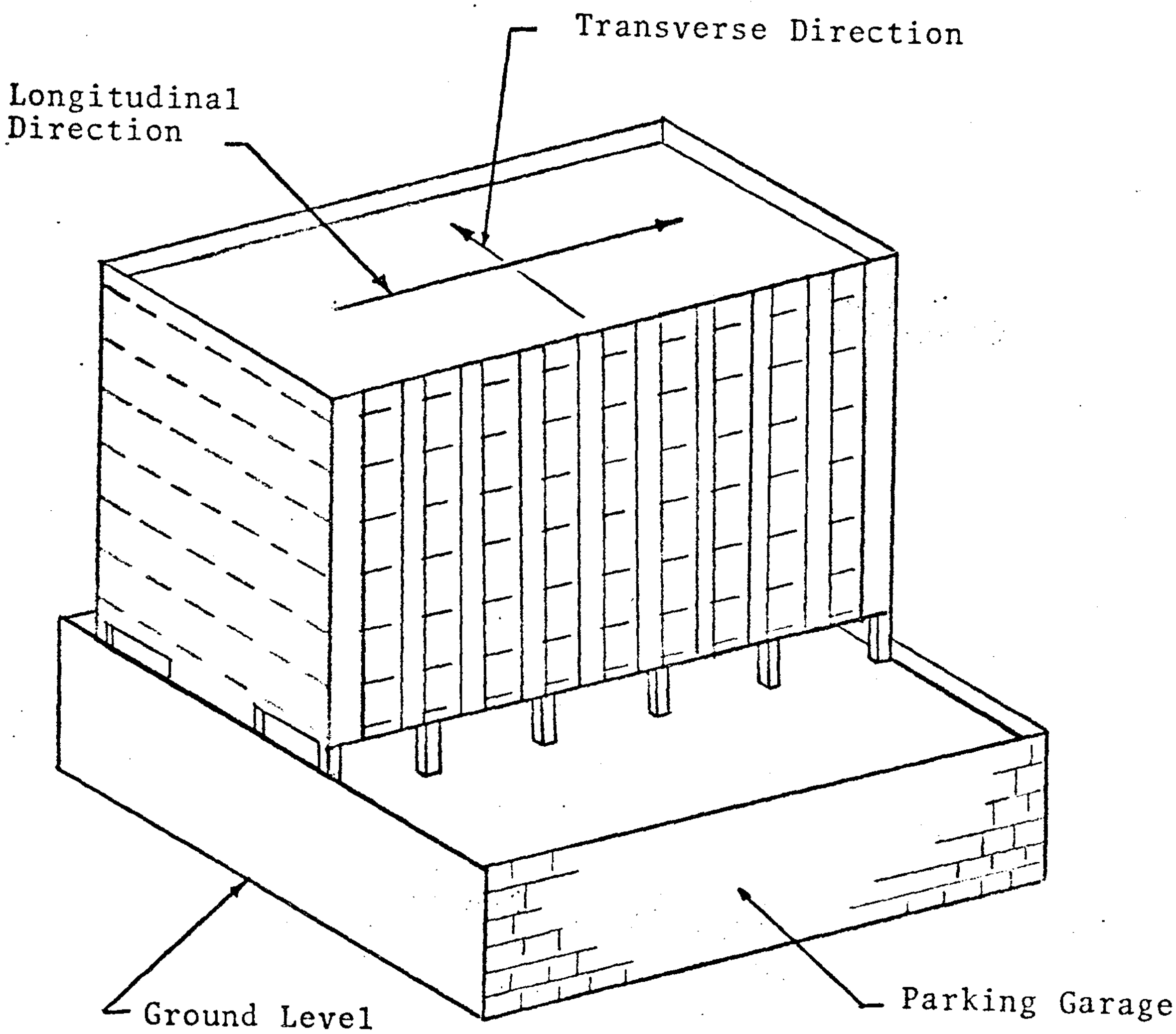

Fig. 1--Schematic of Bank of Nevada building.

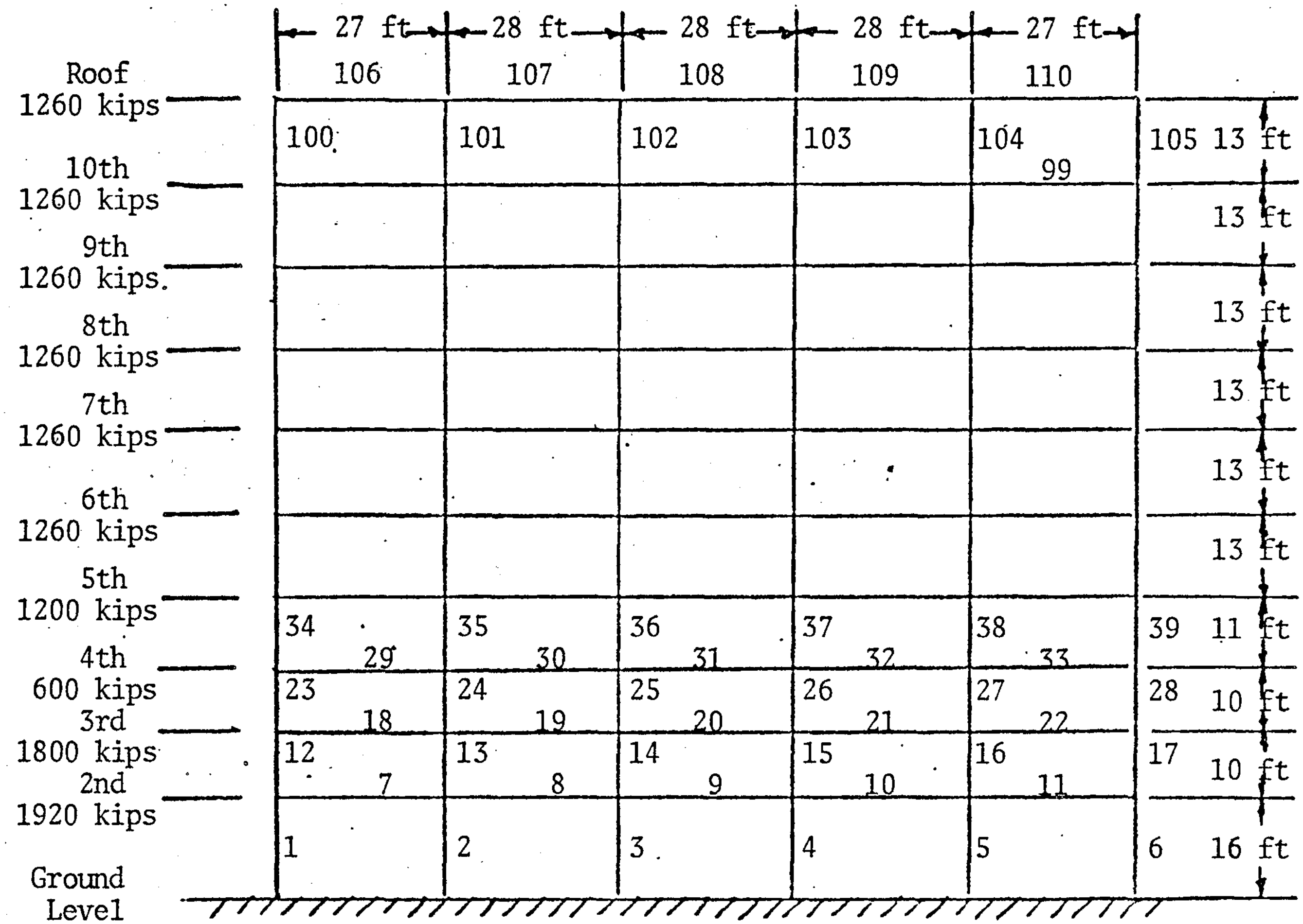

Fig. 2--Finite element model of Bank of Nevada Building, floor weights and beam lengths are depicted. Properties of beams identified by adjacent number are found in Table II. Each nodal point has two translation and one rotation degree of freedom.

TABLE I

Areas and second moments-of-area of columns and girders used in finite element model of Bank of Nevada. Beam lengths can be found in Fig. 2.

Element Number(s)	Area, ft^2	Second Moment, ft^4
1,6,12,17	2.16	0.626
2,5	1.79	0.39
7	3.79	1.62
8	2.97	1.51
9	2.29	0.99
10,11	1.26	0.47
13-16	1.79	0.39
18	3.54	1.56
19	3.48	2.69
20	3.76	1.56
21	4.71	2.05
22	3.69	1.61
23,28,34,39	1.92	0.56
29,30	1.18	0.52
31,32	2.33	1.01
33	3.54	1.56
24-27,35-38	1.58	0.33
40-44,51-55	3.00	0.86
45,50,56,61,67,72,78,83,89,94,100,105	5.56	0.68
46-49,57-60	0.66	0.13
62-66,73-77	2.96	0.83
68-71,79-82	0.51	0.10
84-88,95-99	2.83	0.73
90-93,101-104	0.37	0.07
106-110	3.00	0.97

III. THEORETICAL GROUND MOTION

The analytical earthquake model developed by Cherry[13] is consistent with the elastic rebound theory[16,17] and is essentially a release of accumulated strain energy in the vicinity of the fault. This source model has the unique feature that the rupture process is plastic-work dependent. It has been incorporated into a two-dimensional (plane strain) finite difference computer code. The choice of the finite difference approach to this problem rather than a finite element model was dictated by the highly nonlinear nature of both the faulting process and geologic material response. These nonlinearities can be treated in a more general fashion using a finite difference analog coupled with an explicit integration scheme. A more detailed description of the source model can be found in Ref. [13]. The present study will be limited to the application of the results of this earthquake model to the design and analysis of civil engineering structures.

The lateral ground acceleration produced by this earthquake model at three surface locations is found in Fig. 3. These computer generated records are from points located at the epicenter, 17.5 km away, and 35 km from the epicenter. The thrust fault that generated this motion was normal to the surface and had a length of 5 km. The faulting process was initiated 16.5 km below the surface and propagated to a point 11.5 km from the surface. From Fig. 3 it is seen that the lateral acceleration attenuates like R^{-2}, where R is the distance from the epicenter. This result agrees approximately with many of the attenuation relations summarized by Donovan.[18] In fact, using Donovan's data, the intensity of this theoretical earthquake seems to be equivalent to an earthquake of magnitude 5.5.

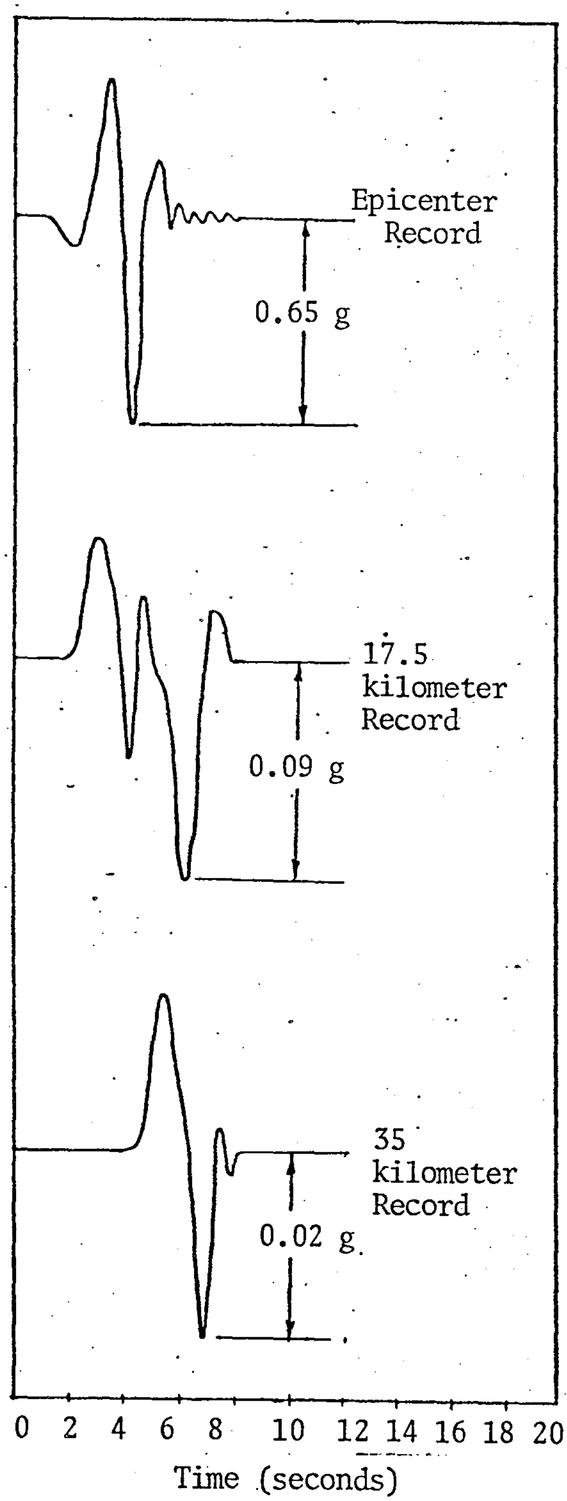

Fig. 3--Theoretical earthquake correlation records.

The two-dimensional simulation of the faulting process is, of course, a simplification of the actual three-dimensional phenomenon. It implies that a simultaneous fracturing occurs along the fault line and produces motion that appears as a line source at large distances. However, for many analyses this idealization is justified. Applications where the distance from the site in question to the fault is small compared to the fault out-of-plane dimension are especially pertinent. The results of the two-dimensional analysis will furnish an approximation of the maximum acceleration to be expected at particular locations rather than a detailed time history of the ground motion. The ability to predict ground motion of the resolution found in a typical strong motion record (see Fig. 4, for example) is many years away.

It is believed that the chief utility in this two-dimensional analytical approach to ground motion prediction is the ability to produce criteria for aseismic structural design. Some observations concerning a recent earthquake suggest this idea. The character of the San Fernando earthquake was three-dimensional in many aspects. It is believed that even if the east-west extent of the faulting was twenty times larger, the intensity in the region of strongest shaking would not be increased significantly.[12] Since this increase in faulting extent results in an earthquake that more closely resembles a two-dimensional configuration, it serves to reinforce the use of two-dimensional models for the prediction of maximum ground motion intensity.

Perhaps the most effective format for presenting design criteria is in the form of response spectra plots. These curves represent the maximum response of a single degree-of-freedom oscillator of period T for various

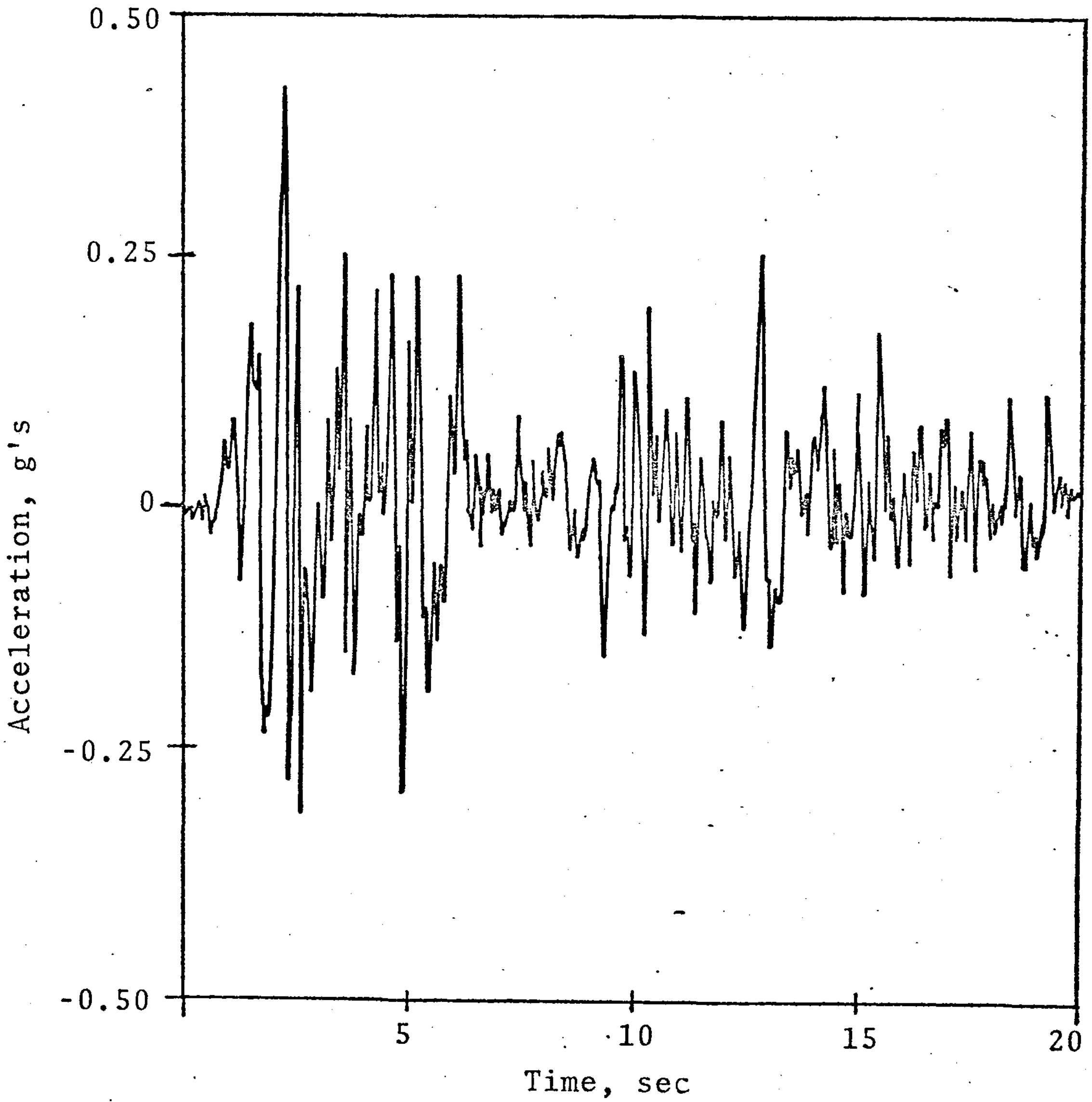

Fig. 4--Acceleration record of 1940 El Centro earthquake, NS component.

amounts of damping levels. A summary of this approach is found in Appendix B. For some structures the characteristic periods are influenced by local geologic material behavior. In the design stage, however, this soil-structure interaction effect may be approximated by appropriate adjustements to the structural periods. The response spectra for the three analytical earthquake records for damping values of 0, 2, 5 and 10 percent are found in Figs. 5 through 7. These plots have been generated using the EQA computer code.[19] It should be noted that these spectral plots have the same general behavior as the design spectra of Newmark and Hall,[20] which are derived from different considerations.

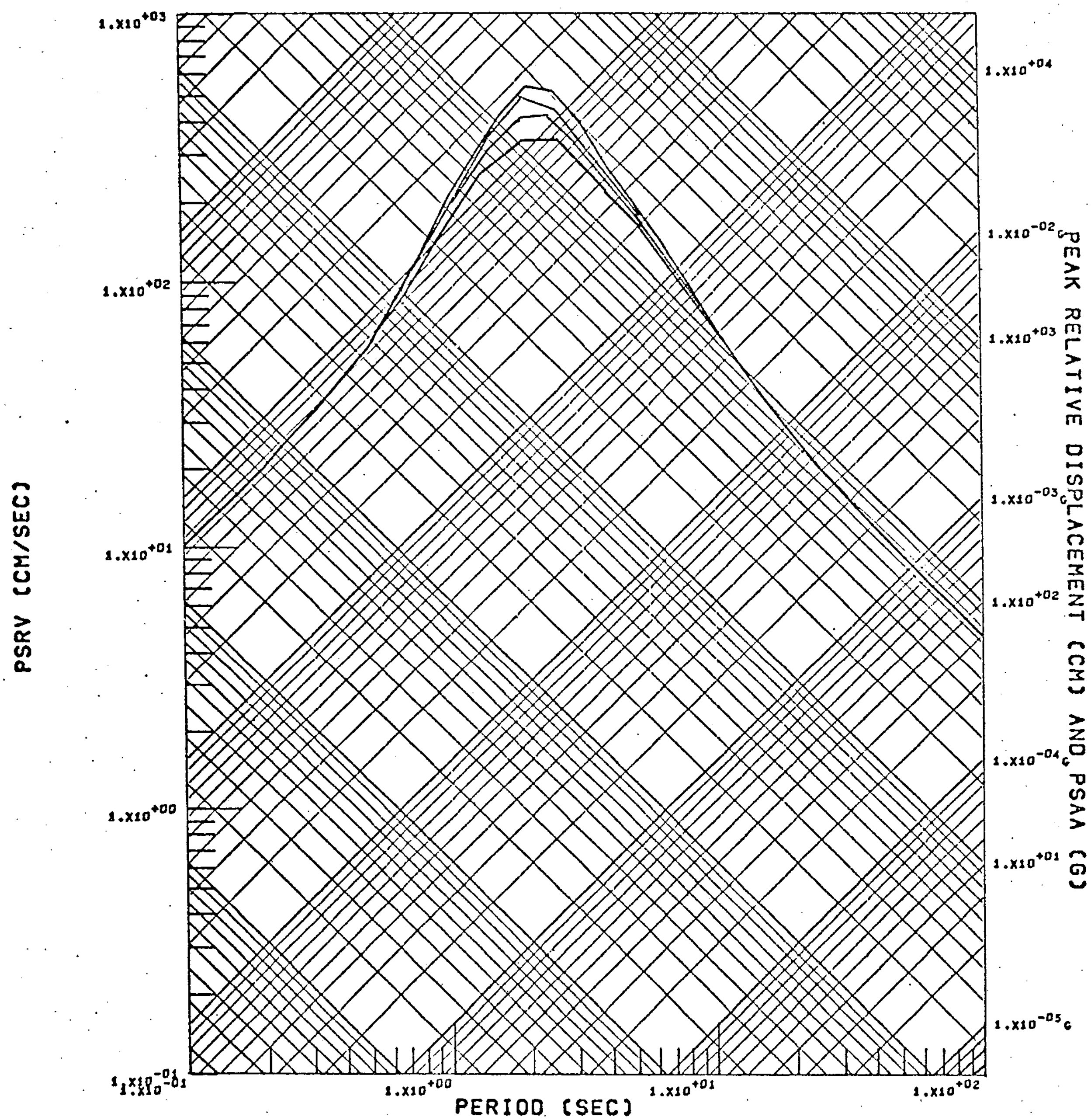

Fig. 5--Response spectra of epicenter theoretical earthquake record. Damping values considered are 0, 2, 5, and 10 percent.

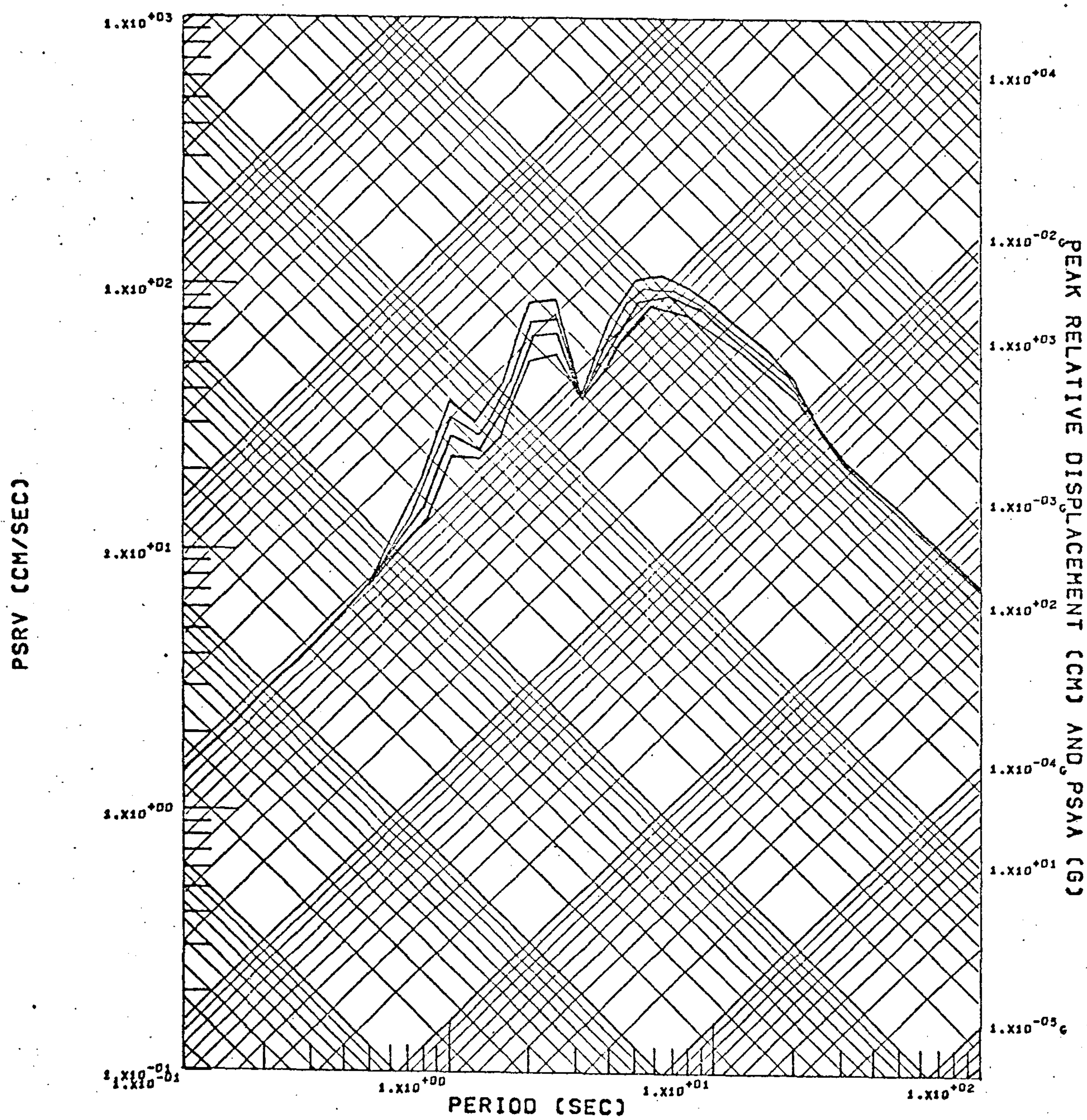

Fig. 6--Response spectra of 17.5 km theoretical earthquake record. Damping values considered are 0, 2, 5, and 10 percent.

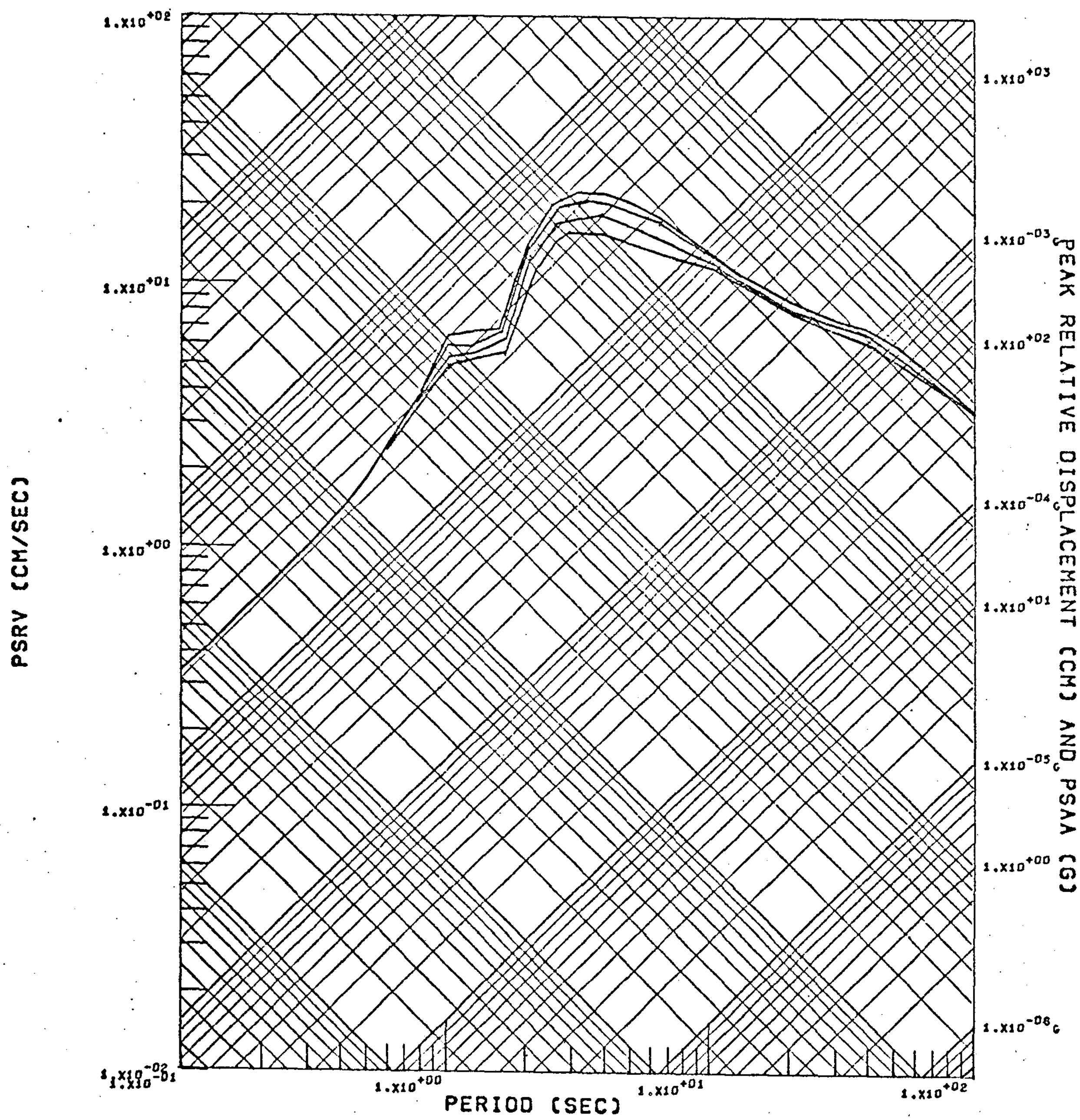

Fig. 7--Response spectra of 35 km theoretical earthquake record. Damping values considered are 0, 2, 5, and 10 percent.

IV. CALCULATIONAL RESULTS

The structural response of the Bank of Nevada building was analyzed using the finite element computer code, SAP.[15] The dynamic response of the lumped mass representation was determined using both time-stepping and modal superposition. In addition, maximum structural response was approximated by applying response spectra techniques. Brief summaries of modal analysis and spectral techniques are found in Appendicies A and B, respectively. For all of the analyses the seismic input was assumed to be lateral acceleration applied to the base of the building.

4.1 TIME-STEPPING APPROACH

The finite element model of the building assumes the following mathematical form:

$$M\ddot{u} + C\dot{u} + Ku = f(t) \tag{1}$$

where

M = mass matrix
C = damping matrix
K = stiffness matrix
u = relative nodal displacement vector
$f(t)$ = seismic forcing function

The mass matrix is derived using the story weights of Fig. 2 and the stiffness matrix is determined using the beam properties found in Table I. Structural damping is usually specified in terms of its modal value, which can only be thought of as an approximation to the highly nonlinear nature of structural dissipation. The damping matrix is assumed to be of the following form:

$$C = \alpha M + \beta K$$

where α and β are scalar parameters. The damping matrix, C, is related to modal damping through the following relationships:[15]

$$\alpha = \lambda\omega$$

$$\beta = \frac{\lambda}{\omega}$$

Namely, if α and β are determined from the above equations a modal damping equal to λ will be specified in a particular mode of frequency ω. The modal damping in other modes of frequency ω_i are given by

$$\lambda_i = \left(\frac{\omega}{\omega_i} + \frac{\omega_i}{\omega}\right)\frac{\lambda}{2}$$

The frequency ω is usually chosen to be that of the fundamental mode. For the present study the modal damping was taken to be 5 percent[14] and with the fundamental period for this structure equal to 1.7 sec, α and β equal 0.184 and 0.0136, respectively. Specific time responses will be discussed in Section 4.4.

4.2 MODAL ANALYSIS

The differential equations describing the motion of the structure, Eq. (1), may be reformulated using modal analysis. After calculating the eigenvectors and eigenvalues (i.e., mode shapes and natural frequencies) of the undamped system the equations of motion may be uncoupled into the principal modes of the system. Adding modal damping, the equations of motion then become

$$\ddot{\eta}_i + 2\lambda_i\omega_i\dot{\eta}_i + \omega_i^2\eta_i = -\gamma_i\ddot{g}(t)$$

where

η_i = modal coordinate of i^{th} mode

λ_i = fraction of critical damping

γ_i = modal participation factor

$\ddot{g}(t)$ = lateral acceleration of base

The displacement history of the nodes of the structure is calculated from the modal coordinates using

$$u = \phi\eta$$

where ϕ is a matrix whose columns are the eigenvectors. A more detailed description of modal analysis is found in Appendix A.

The mode shapes and periods for the first four modes of the structural model of the Bank of Nevada are found in Fig. 8. All mode shapes have been normalized to 1.0 at their maximum values. These modes are all predominately lateral deflections with the vertical displacements being of a lower order of magnitude. The calculated period of the fundamental mode of 1.7 sec agrees very well with the value of 1.72 sec given by Tokarz and Bernreuter[14] for their model L5. A modal damping of 5 percent is used in all calculations.

4.3 SPECTRAL ANALYSIS

A response spectrum as used in this study is defined as the variation with undamped period of the maximum response of a single degree-of-freedom oscillator for a particular ground motion. The relative displacement spectrum, S_D, is then the behavior of the maximum relative displacement versus period. It is convenient to introduce pseudo relative velocity (PSRV) and pseudo absolute acceleration (PSAA) defined by the following relations:

$$\text{PSRV} = \omega S_D$$

$$\text{PSAA} = \frac{S_D}{\omega^2}$$

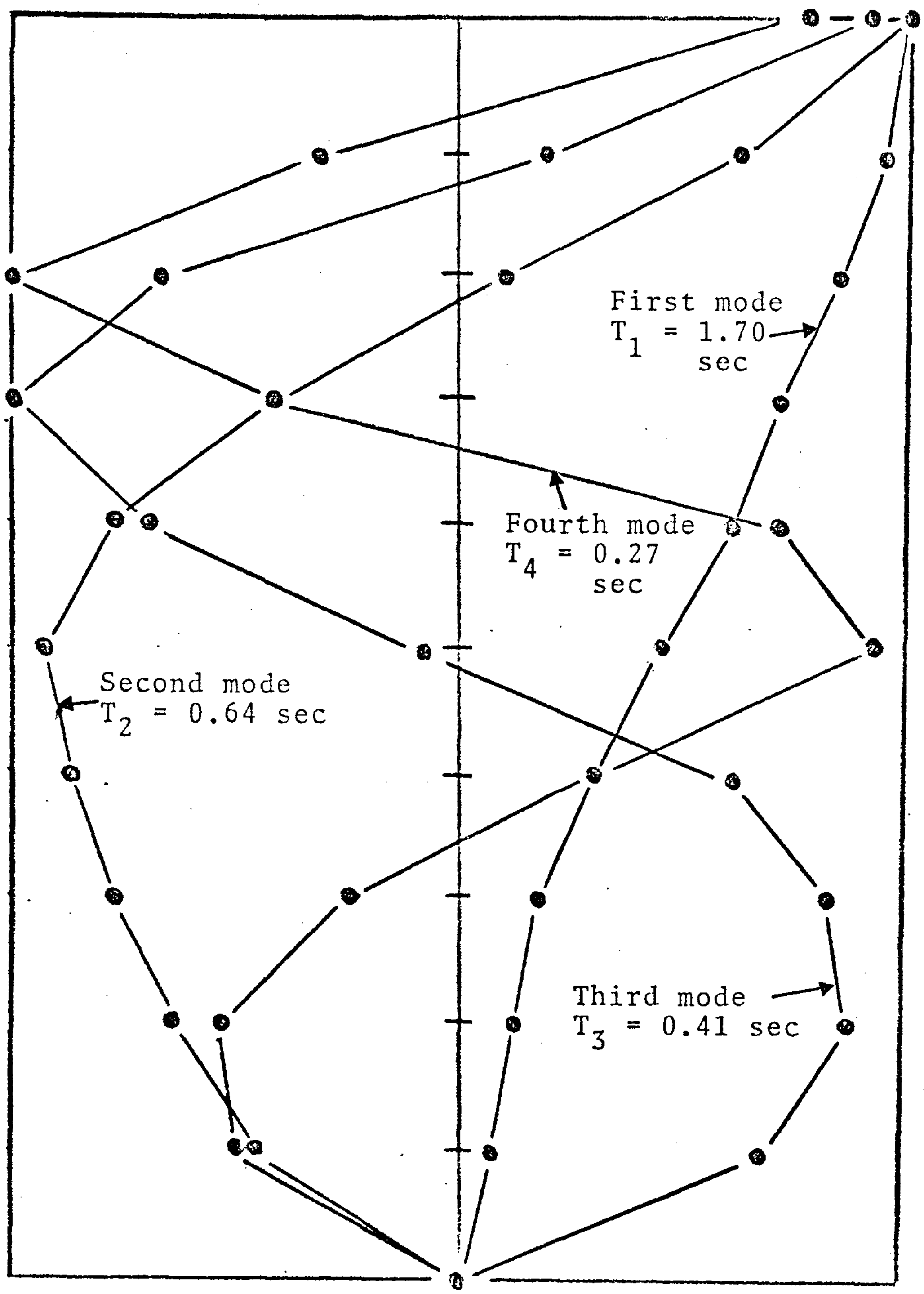

Fig. 8--Mode shapes and frequencies for first four normal modes of the Bank of Nevada building.

where ω is the natural frequency (2π/period). For damping less than 10 percent these spectra are very nearly equal to the actual velocity and acceleration spectra. The use of these pseudo values allows all of the spectral characteristics to be presented on a single plot. Examples of these tripartite logarithmic plots are found in Figs. 5 through 7.

Given the values of damping and period for mode i, the maximum displacement of this mode is determined from the spectral plots and is defined as S_{Di}. The maximum displacements are then determined using a root-mean-square contribution of each mode. The pertiment relationships are outlined in Appendix B.

4.4 CALCULATION SUMMARY

The numerical calculations associated with this study determine both stresses and displacements. Because the main emphasis is to present a particular seismic design technique rather than a detailed stress analysis, only the displacement temporal variation will be discussed. In addition to the three theoretical earthquake records discussed in Section III, the building is also excited using the 1940 El Centro record[7] of Fig. 4. Its response spectra are found in Fig. 9 for various amounts of damping.

A comparison between the time stepping and modal superposition calculational approaches is found in Fig. 10. Depicted is the displacement history of the roof due to the El Centro seismic loading. This excellent agreement serves to confirm both the normal mode concept and the calculational procedure used in the SAP code. The minor differences in peak displacements are probably due to the slightly different damping descriptions.

A typical building displacement configuration at a particular instant of time during the dynamic analysis is

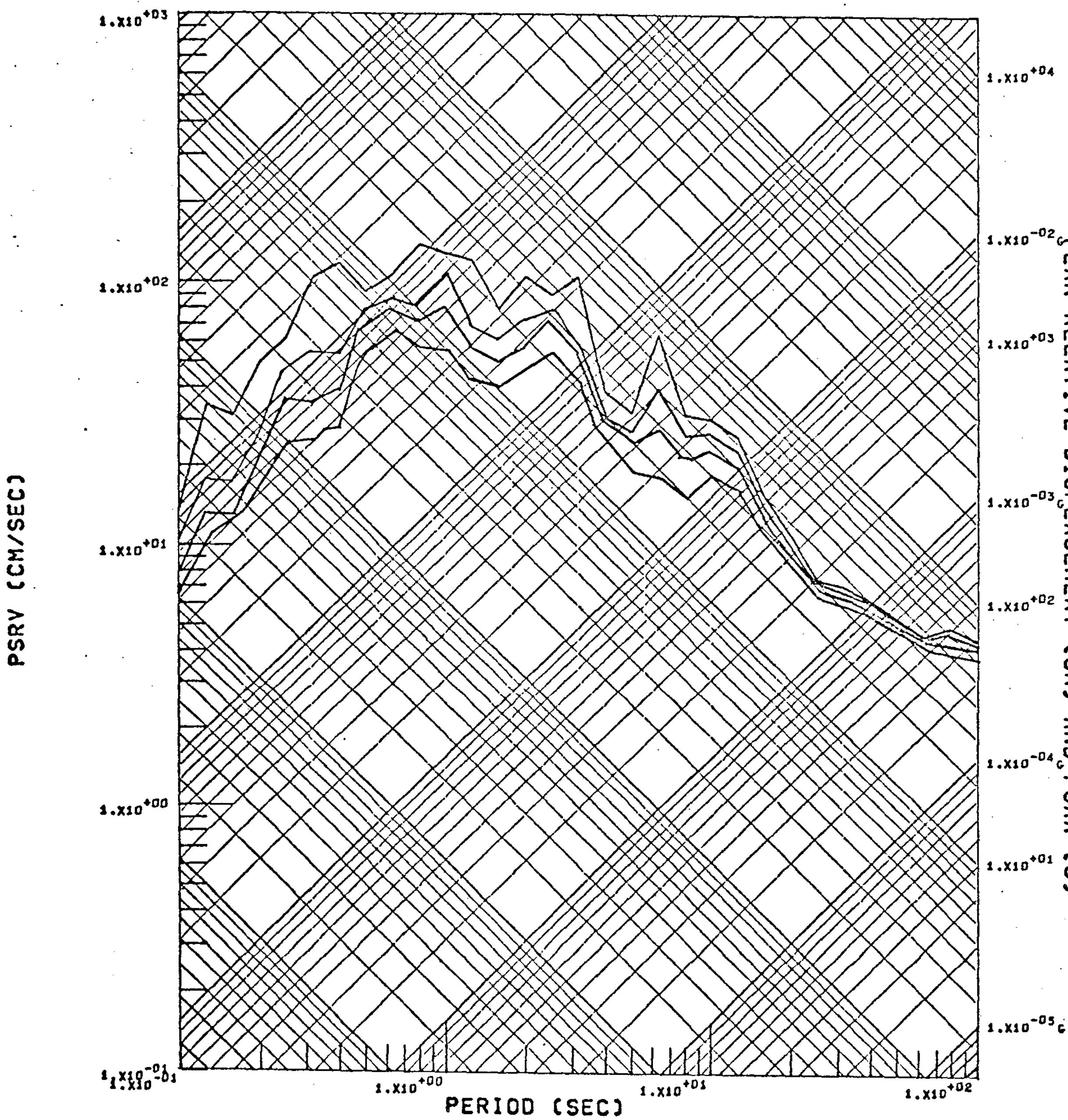

Fig. 9--Response spectra of 1940 El Centro NS earthquake record. Damping values considered are 0, 2, 5 and 10 percent.

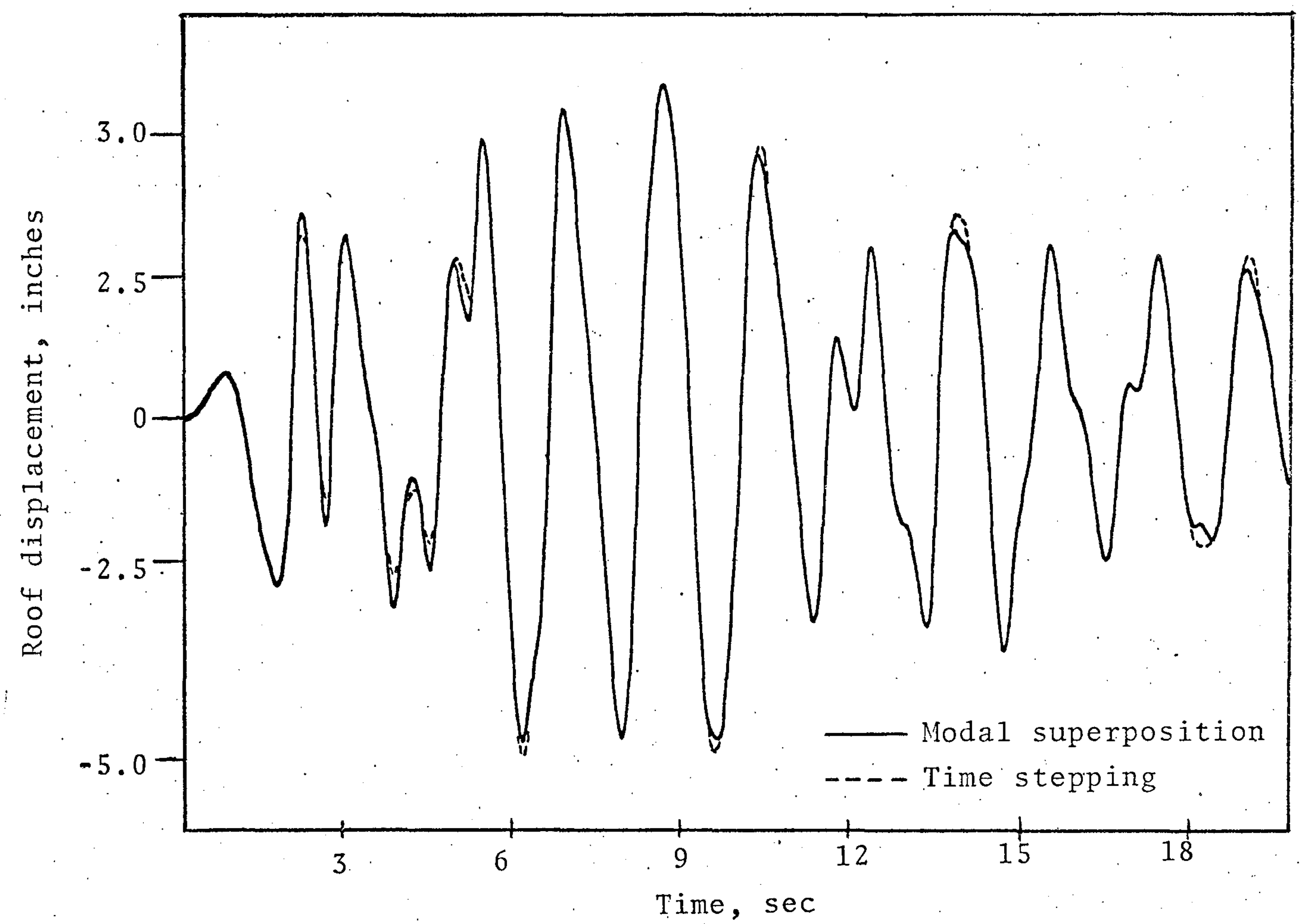

Fig. 10--Time response of roof of Bank of Nevada subject to 1940 El Centro NS ground motion. Comparison between modal superposition and time-stepping techniques.

presented in Fig. 11. The displacements in this figure are multiplied by 7.5 for graphical purposes. This specific example is from the response due to the 17.5 km theoretical ground motion. Figure 11 is actually a single frame of a computer-generated movie depicting the building motion. This movie capability has been made an operational feature of the SAP code during the course of this study.

The spectral analysis technique circumvents many of the details of a dynamic analysis and presents the maximum displacements (or stresses) experienced during a particular ground motion excitation. The resulting maximum roof displacement calculated using a RMS modal summation for the three theoretical and the El Centro ground motion excitations are presented in Table II. Appearing also, for comparison, are the maximum values from the temporal modal superposition analysis. These later values are essentially identical to the time-stepping values. The spectral calculations agree very well with their time history counterparts for all cases considered. These results help explain why spectral approaches are so valuable for both design and analysis applications. It is interesting to note that even though the 17.5 km theoretical and El Centro time records are quite dissimilar, the maximum displacements experienced by the building for these two earthquakes are relatively close. This fact becomes apparent when the spectral plots at 5 percent damping are examined. For, at the fundamental period of the Bank of Nevada, 1.7 sec, these spectra are equal.

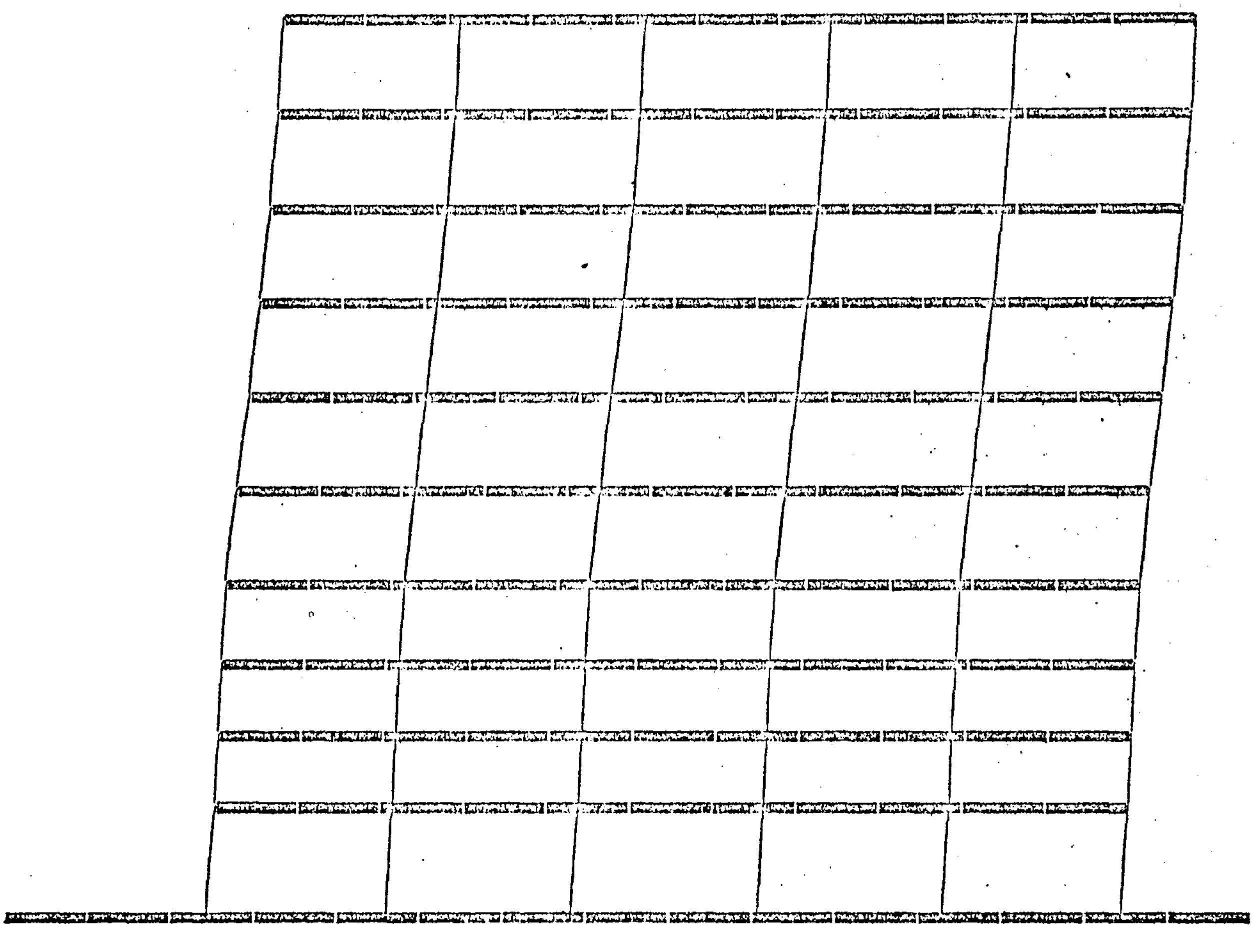

Fig. 11--Typical displacement configuration of Bank of Nevada subjected to epicenter theoretical earthquake record. Displacements magnified by 7.5.

TABLE II

Earthquake Record	Maximum Roof Displacement, feet	
	Time Integration	Response Spectra
El Centro	0.545	0.597
Theoretical, epicenter	4.560	4.376
Theoretical, 17.5 km	0.525	0.564
Theoretical, 35 km	0.096	0.109

Maximum roof displacement of Bank of Nevada for El Centro and theoretical earthquakes. Comparison between time integration and response spectra results.

V. SUMMARY AND CONCLUSIONS

A knowledge of the expected seismic loading at a particular site is an extremely important input to the design of civil engineering structures. There exists today a critical need to provide a rational approach to the specification of this ground motion. In the present study it has been postulated that one approach to this specification is through the use of theoretical earthquake models. It is felt that the use of two-dimensional simulations will provide an approximation to the maximum ground motion intensity to be expected for many types of fault systems. The resulting theoretical ground motion records can then be used to derive response spectra for utilization in structural design.

A particular high-rise building, the Bank of Nevada, has been analyzed for both theoretical and measured ground motion excitation. A dynamic two-dimensional finite element model has been utilized using time-stepping, modal superposition, and spectral approaches. The results confirm the utility of the use of response spectra for design and analysis of structures.

Further extensions to this study can, and should, be pursued. It is a straightforward task, for example, to include detailed geologic descriptions. The use of the two-dimensional finite difference code to model nonlinear geologic material behavior and layered configurations has been accomplished previously.[21-23] Also, while the present building model has been two-dimensional, the SAP code can treat three-dimensional structural models quite easily.

APPENDIX A

MODAL SUPERPOSITION

The concept of normal modes is best introduced by first considering the free vibration problem

$$M\ddot{u} + Ku = 0$$

where u is a $n \times 1$ displacement vector, M is the $n \times n$ mass matrix, and K is the $n \times n$ stiffness matrix. Assuming a solution of the following form

$$u = u_0 e^{i\omega t}$$

yields the well known eigenvalue problem

$$(K - \omega^2 M)u_0 = 0$$

The n eigenvalues, ω_i^2, and n eigenvectors, u_{0i}, can be determined using standard techniques.[24]

Considering now the forced vibration problem

$$M\ddot{u} + Ku = f(t),$$

where $f(t)$ is the forcing function vector; the normal coordinates, η_i, are introduced using the following coordinate transformation:

$$u = \phi\eta$$

where ϕ is the $n \times n$ matrix whose columns are the eigenvectors u_{0i}. Substituting this expression into the equations of motion and premultiplying by ϕ^T yields

$$\phi^T M \phi \ddot{\eta} + \phi^T K \phi \eta = \phi^T f(t)$$

It follows from eigenvalue theory that $\phi^T M \phi$ is a diagonal matrix.[24] In fact, this matrix is usually normalized to equal the identity matrix, i.e.,

$$\phi^T M \phi = I$$

Therefore, using the fact that

$$K u_0 = \omega^2 M u_0$$

when $\phi^T K \phi$ is also a diagonal matrix whose nonzero terms equal ω_i^2. Using these relationships, the differential equation describing the behavior of the ith normal coordinate is given by:

$$\ddot{\eta}_i = \omega_i^2 \eta_i = \sum_{j=1}^{n} \phi_{ji} f_j(t)$$

When the forcing function can be expressed as

$$f_i(t) = -\ddot{g}(t) m_i ,$$

where m_i equal the diagonal mass matrix terms, the above differential equation becomes

$$\ddot{\eta}_i + \omega^2 \eta_i = -\gamma_i \ddot{g}(t)$$

The term γ_i is called the modal participation factor and equals

$$\gamma_i = \sum_{j=1}^{n} m_j \phi_{ji}$$

This term assumes a slightly different form if the aforementioned normalization is not invoked.[14]

APPENDIX B

SPECTRAL RESPONSE

The response spectrum is a plot of the maximum relative displacement versus period for a single degree-of-freedom oscillator subject to a prescribed support acceleration and, at a particular level of damping. This equation of motion is given by

$$\ddot{u} + 2\omega\lambda\dot{u} + \omega^2 u = -\ddot{g}$$

where

u = relative displacement

ω = undamped natural frequency (2π/period)

λ = fraction of critical damping

$\ddot{g}$ = base acceleration

The solution is given by

$$u(t) = \frac{-1}{\omega(1-\lambda^2)^{1/2}} \int_0^t g(\tau)e^{-\lambda\omega(t-\tau)} \sin\left[\omega(1-\lambda^2)^{1/2}(t-\tau)\right] d\tau$$

The displacement spectrum is thus defined by

$$S_D = |u(t)|_{max}$$

Similar relations can also be defined for velocity and acceleration. However, for $\lambda < 0.1$ the following pseudo spectra nearly equal their exact counterparts:

PSRV = ωS_D, pseudo relative velocity

PSAA = $\frac{S_D}{\omega^2}$, pseudo absolute acceleration

These relationships allow all three quantities to be plotted on a single tripartite logarithmic plot as is seen in Fig. 9. Using the results of Appendix A, the contribution to the maximum displacement of mass node j due to mode i is given by

$$u_{j,i}^{max} = \gamma_i \phi_{ji} S_{Di}$$

where S_{Di} is S_D evaluated at ω_i. Because the contributions from all modes do not occur at the same instant of time, the total maximum displacement at a mass node is usually approximated using the root-mean-square summation. Similar relationships can be derived for stresses.[14]

VI. REFERENCES

1. Bielak, J., "Earthquake Response of Building-Foundation Systems," Ph.D. Thesis, California Institute of Technology, 1971.

2. Jennings, P. C., R. B. Matthiesen, and J. B. Hoerner, "Forced Vibration of a Tall Steel-Frame Building," Earthquake Engineering and Structural Design, Vol. 1, pp. 107-132, 1972.

3. Weaver, W., G. E. Brandow, and K. Höeg, "Three Dimensional Soil-Structure Response to Earthquakes," BSSA, Vol. 63, No. 3, pp. 1041-1056, 1973.

4. Wood, J. H., "Analysis of the Earthquake Response of a Nine-Story Steel Frame Building During the San Fernando Earthquake," California Institute of Technology Earthquake Engineering Research Laboratory, Report No. EERL 72-04, 1972.

5. "Response of an Eleven-Story Reinforced Concrete Masonary Building to Earthquake Ground Motion," Agbabian-Jacobsen Associates, R-3200-2211, 1972.

6. "Dynamic Analysis Now Required for Structures in Los Angeles," Engineering News Record, p. 14, May 3, 1973.

7. Brady, A. G., D. E. Hudson, and M. D. Trifunac, "Strong-Motion Earthquake Accelerograms, Digitized and Plotted Data," California Institute of Technology, EERL-71-50, 1971.

8. Jennings, P. C., G. W. Housner, and N. C. Tsai, "Simulated Earthquake Motions for Design Purposes," Proc. 4th World Conference on Earthquake Engineering, Santiago, Chili, 1969.

9. Ruiz, P., and J. Penzien, "Stochastic Seismic Response of Structures," J. of Engr. Mech. Div., Proceedings of the ASCE, pp. 441-456, April 1971.

10. Newmark, N. M., A. R. Robinson, A. H. S. Ang, L. A. Lopez, and W. S. Hall, "Methods for Determining Site Characteristics," Proceedings International Conference of Microzonation for Safer Construction Research and Application, Seattle, Washington, 1972.

11. Perkins, D. M., S. T. Harding, K. W. Campbell, and A. F. Espinosa, "Studies of Site Amplification in San Fernando," Proceedings International Conference of Microzonation for Safer Construction Research and Application, Seattle, Washington, 1972.

12. Housner, G. W. and P. C. Jennings, "The San Fernando California Earthquake," Earthquake Engineering and Structural Dynamics, Vol. 1, pp. 5-31, 1972.

13. Cherry, J. T., "Calculations of Near Field, Earthquake Ground Motion," Systems, Science and Software, Report No. SSS-R-73-1759, June 1973.

14. Tokarz, F. J., and D. L. Bernreuter, "Comparison of Calculated and Measured Response of a High-Rise Building Ground Motions Produced by Underground Nuclear Detonations," Lawrence Radiation Laboratory, TID-4500, UC-35, December 1970.

15. Wilson, E. L., "SAP-A General Structural Analysis Program," University of California at Berkeley, UCSESM-70-20, September 1970.

16. Reid, H. F., "The Mechanics of Earthquakes: The Elastic Rebound Theory, Regional Strain," Bulletin National Research Council, No. 90, 1933.

17. Benioff, H., "Earthquake Source Mechanisms," Science, 143, March 1964.

18. Donovan, N. C., "Earthquake Hazards for Buildings," Proc., Building Practices for Disaster Mitigation, Boulder, Colorado, 1972.

19. Hamilton, K. G., "EQA - A Computer Code to Calculate Seismic Response Spectra," Systems, Science and Software Internal Report.

20. Newmark, N. M., and W. J. Hall, "Procedures and Criteria for Earthquake Resistant Design," Proceedings, Building Practice for Disaster Mitigation, Boulder, Colorado, 1972.

21. Cherry, J. T., J. Sweet, and E. J. Halda, "Prediction of Surface Vertical and Horizontal Displacements in Non-Homogeneous Media Resulting from Surface Nuclear Explosions," Systems, Science and Software, Report No. SSS-R-73-1517, January 1973.

22. Sweet, J., "Two-Dimensional Stress Wave Models for Porous Media," Systems, Science and Software, Report No. SSS-R-72-1315, October 1972.

23. Cherry, J. T., A. J. Good, and K. G. Hamilton, "Free Field Calculations for Seismic Predictions," Systems, Science and Software, Report No. SSS-R-73-1605, April 1973.

24. Crandall, S. H., Engineering Analysis, McGraw-Hill, 1956.