PMFSEL REPORT 92-1

PERFORMANCE OF MASONRY AND MASONRY VENEER BUILDINGS IN THE LOMA PRIETA EARTHQUAKE

by

Imran Jalil
William Kelm
and
Richard E. Klingner

Sponsored by

National Science Foundation
Grant No. BCS-9003771

ACKNOWLEDGEMENTS

The National Science Foundation (Grant No. BCS-9003771) sponsored the project which made this research and thesis possible. The cognizant NSF program official was Dr. Shih-Chi Liu. Supplemental funding was provided by the International Masonry Institute, whose technical contact was Robert Beiner. Assistance in field studies was provided by Thomas Polizzi of the Masonry Institute, San Mateo, California.

The authors also wish to thank the following individuals for their assistance during the research:

Richard Dreyer	Peter Culley & Associates, San Francisco, CA
Arnold Luft	
Philip Luke	
Anita Norton	
Boris Bresler	Wiss, Janney, Elstner Associates, Emeryville, CA
Sigmund Freeman	
Dan Eilbeck	
William Holmes	Rutherford & Chekene, San Francisco, CA
Bret Lizundia	
Randolph Langenbach	School of Architecture, University of California at Berkeley, CA
Owen O'Neil	Hugh M. O'Neil Co., Oakland, CA
Douglas Vossbrinck	Vossbrinck & Associates, Palo Alto, CA
Mr. Warren Heid	Warren B. Heid AIA & Associates, Saratoga, CA
Mr. Kenneth Simpkins	Loma Prieta Consolidated District School, Loma Prieta, CA
Mr. John Tissell	American Plywood Association, Tacoma, WA
Donald Ifland	Santa Cruz, CA
Chuck Pratt	Calstone, Santa Clara, CA
Chris Poland	Henry J. Degenkolb Associates, San Francisco, CA
James Malley	
John Kellogg	Port of San Francisco, CA
Walter Sequeira	
Robert Blake	General Engineering, San Francisco, CA

The Management Group of the Peninsula Office Building

The Technical Staff at Computers & Structures, Inc., Berkeley, CA

Special thanks go to Robert Gloyd and Nubz Newberg of the Civil Engineering Micro Lab, who went out of their way to provide computer hardware and software support. The advice of Prof. Jose Roesset, of the University of Texas at Austin, was very helpful.

DISCLAIMER

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 General

On Monday, October 17, 1989, at 5:04 p.m. local time, a strong earthquake measuring 7.1 on the Richter scale struck just south of the San Francisco Bay Area. The epicenter was approximately 16 kilometers northeast of Santa Cruz and 30 kilometers south of San Jose. The earthquake ruptured a 40-kilometer segment of the San Andreas fault. At the Corralitos station, which was less than 10 kilometers away from the fault, peak horizontal ground accelerations of 0.64g were recorded [2, 11, 32].

Although strong shaking lasted less than 15 seconds, damage was widespread: more than 60 people were killed; over 3,700 people were injured; approximately 360 businesses were destroyed; and more than 12,000 people were left homeless. The Marina District of San Francisco, approximately 100 kilometers from the epicenter, was particularly hard hit. Liquefaction effects were widely noted in this area. In addition, magnification of ground motions through the deep fills, combined with the low lateral stiffness of the street levels of many buildings in the Marina District, contributed to the heavy structural damage there [11, 21].

Modern engineered buildings, including recently designed and constructed masonry buildings, sustained little or no damage in the Loma Prieta earthquake. The most concentrated and severe damage to building structures occurred in unreinforced masonry (URM) bearing wall buildings. URM buildings, constructed of wood-frame floor and roof systems supported by thick unreinforced brick walls, were a common type of construction throughout California prior to the 1933 Long Beach earthquake [25]. This report focuses on the more recent modern masonry buildings that sustained little or no damage.

1.2 Objectives of Project

An important feature of the Loma Prieta earthquake was its effect on masonry and masonry veneer buildings. The general objectives of this project were as follows:

1) Review the overall behavior of masonry and masonry veneer buildings in the October 1989 Loma Prieta Earthquake.

2. Select a small number of representative masonry and masonry veneer buildings for further study, based on their usefulness as prototypes, and on the availability of plans, specifications and strong-motion records.

3. Study their damages in detail, and compare their responses with that predicted analytically.

4. Evaluate the provisions of the 1988 Uniform Building Code (UBC) [37] as applied to these buildings.

1.3 Objectives and Scope of this Report

This report describes the investigation of the following four buildings:

Loma Prieta Community Center	Reinforced Concrete Masonry Bearing Wall
Peninsula Office Building*	Steel Frame with Masonry Veneer
2 Alhambra Street Apartment Complex	Wooden Frame with Masonry Veneer
Hotel Woodrow	Steel Frame with Masonry Veneer

This study evaluates the response of each building analytically, compares observed damage with that predicted analytically, evaluates provisions of the 1988 Uniform Building Code as applied to these buildings, and makes recommendations for the seismic design of masonry structures similar to those studied. An area map showing the locations of the four building sites is presented in Figure 1.

1.4 Organization of Report

This report is organized as follows:

Chapter 2 discusses the method used in selecting the buildings studied in this report.

Chapters 3 and 4 discuss in detail the first two buildings studied in this report. Those two chapters include a description of each building, the ground motions selected, the analytical models used, a comparison of the predicted response with observed damage, evaluation of 1988 UBC design provisions where applicable, and probable response of each building in a stronger earthquake.

*At the request of the current owner, this building's name, location, and current use have been withheld from this report.

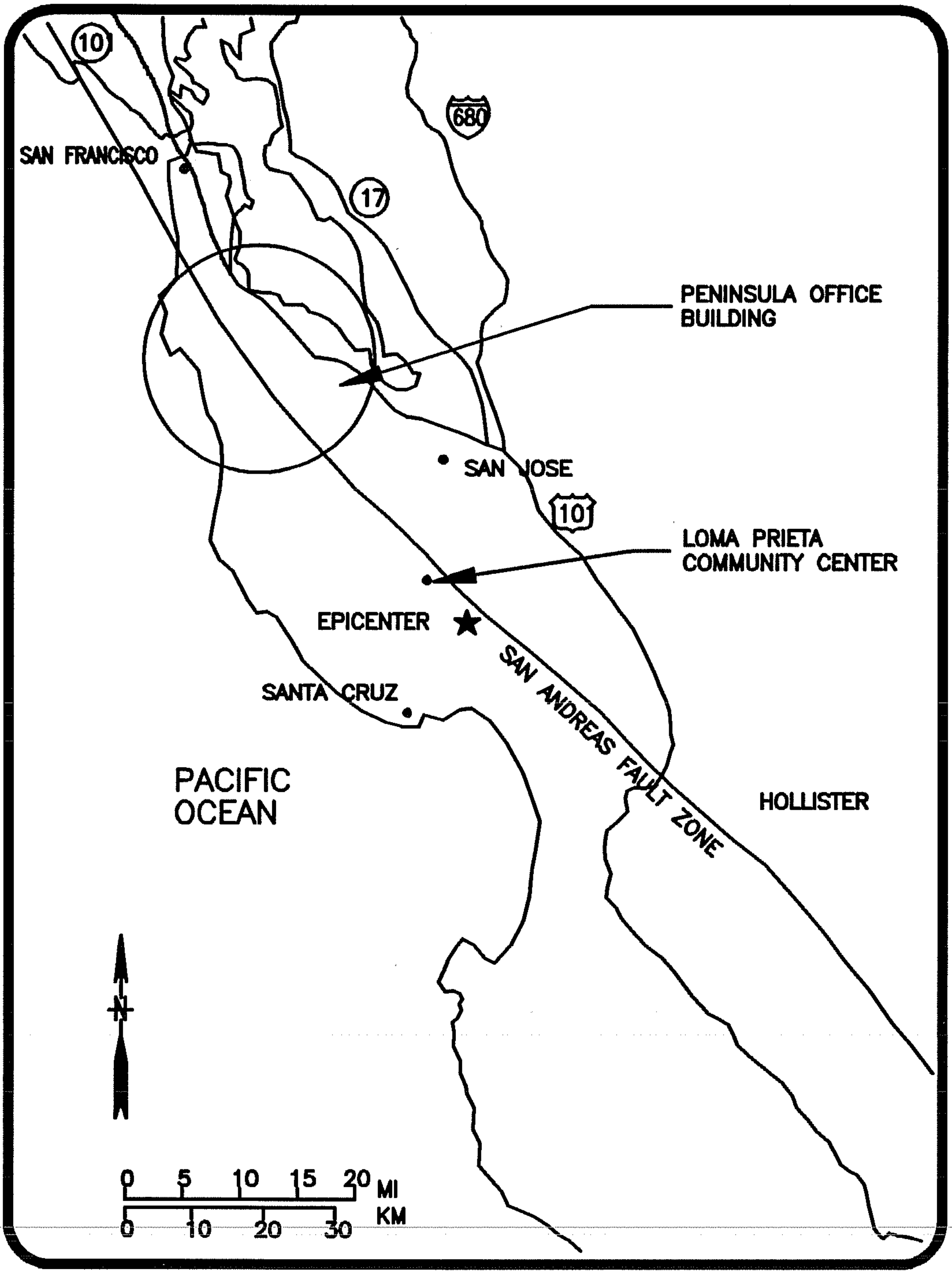

Figure 1 Area map showing building site locations.

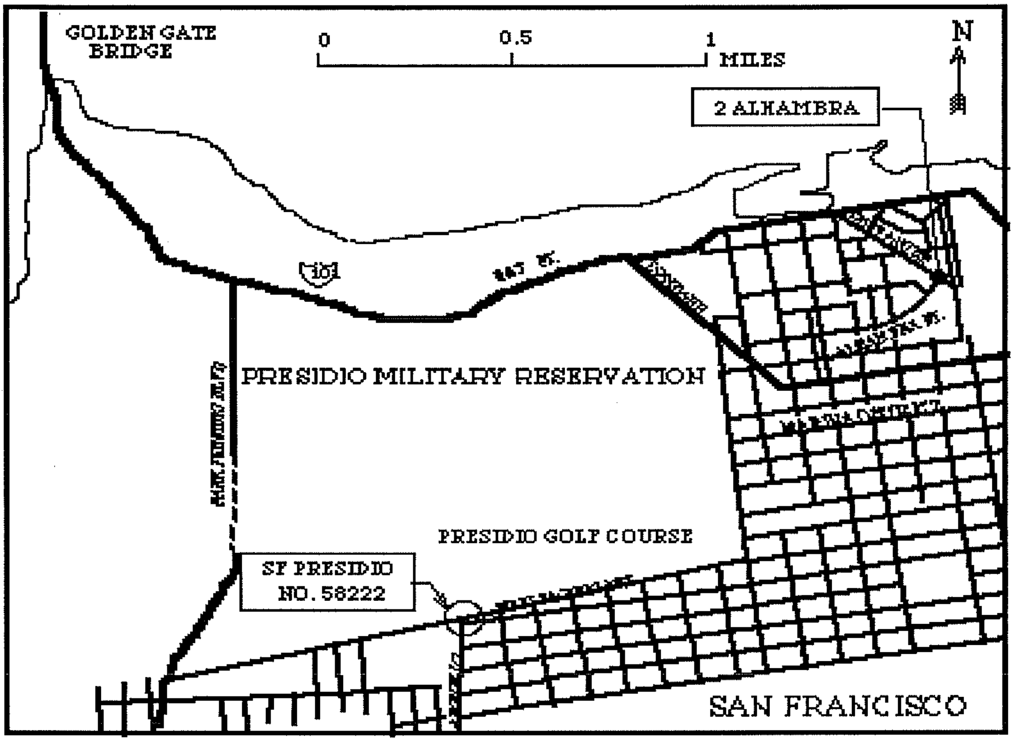

Figure 1 Area map showing building site locations (continued).

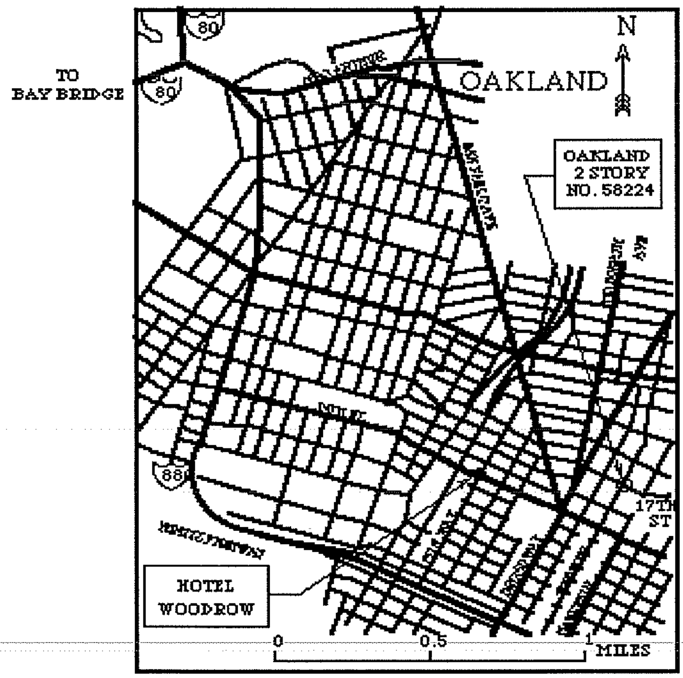

Figure 1 Area map showing building site locations (continued).

Chapter 5 and 6 discuss in detail the last two buildings studied in this report. These two chapters include a description of each building, the ground motions selected, the analytical models used and a comparison of the predicted response with observed damage.

Chapter 7 presents a summary of the findings and conclusions of this investigation, and recommendations for design and further research.

Appendices A and B present design criteria, design calculations, computer results and computer data files. Appendix C contains a soil response program developed to calculate the modification of earthquake records from rock recording sites to the soft soil of the Marina District.

CHAPTER 2

SELECTION OF BUILDINGS TO BE INVESTIGATED

As initially outlined in the research proposal presented to the National Science Foundation, all types of masonry and masonry veneer buildings, including unreinforced masonry, were to be investigated. However, as part of the overall NSF/USGS Loma Prieta program, a grant similar to that supporting this project was also awarded for another project specifically intended to study the behavior of unreinforced bearing wall masonry. Therefore, this investigation concentrated on other types of masonry:

1) modern reinforced bearing wall construction

2) modern anchored or adhered veneer construction

3) older unreinforced masonry infills with steel or concrete frames

4) wood-framed apartment buildings with masonry veneer

The following criteria were used for preliminary building selection:

1) The building construction should be one of the four types listed above.

2) Ground motion records should be available, either at the building site or reasonably close to it.

3) The building should be undamaged or lightly damaged. This would permit more precise estimation of the force levels and deflections during the earthquake.

4) Plans of the building should be available, and the owner and engineer should be amenable to having their building studied.

Using the above criteria, a preliminary list of potential buildings was developed by the Principal Investigator before and during a trip to the Bay Area in June 1990. During this trip the following engineering and masonry industry representatives were contacted:

Boris Bresler Sigmund Freeman Dan Eilbeck	Wiss, Janney, Elstner Associates, Emeryville, CA
Arnold Luft Richard Dreyer Philip Luke	Peter Culley & Associates, San Francisco, CA
William Holmes	Rutherford & Chekene, San Francisco, CA
Douglas Vossbrinck	Vossbrinck & Associates, Palo Alto, CA
Thomas Polizzi	Masonry Institute, San Mateo, CA
Donald Ifland	Santa Cruz, CA
Chuck Pratt	Calstone, Santa Clara, CA
Chris Poland James Malley	Henry J. Degenkolb Associates, San Francisco, CA
John Kellogg Walter Sequeira	Port of San Francisco, CA

The potential buildings, grouped by construction type, are listed below. The most promising building of each group is marked with an asterisk:

1. Modern Reinforced Bearing Wall Construction

   * Loma Prieta Community Center, load-bearing concrete block, very close to the epicenter

     Kelly-Moore Paint Store, Santa Clara

     Building under Construction (Soquel & Thurber), Santa Cruz

     Kinney Shoe Store, Santa Cruz

     Mormon Church, Loma Prieta

2. Modern Anchored or Adhered Veneer Construction

   * Peninsula Office Building, steel frame with brick veneer

     Lockheed Building, Palo Alto

3) Older Unreinforced Masonry Infills with Steel or Concrete Frames

* Atlas Building (Mission & New Montgomery), San Francisco

  Ferry Building, San Francisco

  Fairmont Hotel, San Francisco

  Touraine Hotel, Oakland

  Hotel Oakland, Oakland

  Unity Building, Oakland

  Hotel Harrison, Oakland

  Julia Morgan YWCA, Oakland

4. Wooden Apartments with Masonry Veneer

* Two Alhambra, Marina District

  1801 Mallorca, Marina District

In September 1990, all investigators made another trip to the San Francisco Bay area to select 3 to 4 of the above buildings for further study. During this trip, on-site inspections of several buildings were made and the following engineering and masonry industry representatives were visited:

Richard Dreyer	Peter Culley & Associates, San Francisco, CA
Bret Luzundia	Rutherford & Chekene, San Francisco, CA
Douglas Vossbrinck Larry Cofer	Vossbrinck & Associates, Palo Alto, CA
Warren Heid	Warren B. Heid AIA & Associates, Saratoga, CA
Kenneth Simpkins	Loma Prieta School, Loma Prieta, CA
Thomas Polizzi	Masonry Institute, San Mateo, CA
Randolph Langenbach	School of Architecture, University of California at Berkeley, CA

After the list of potential buildings had been developed, the Woodrow Hotel (masonry infill) was suggested for study by Prof. Randolph Langenbach of the School of Architecture at the University of California, Berkeley. During the September 1990 trip to the Bay Area, a site visit was made to the hotel and available plans were reviewed. It was determined that the Woodrow Hotel represented a good infilled-frame candidate for further study.

At the conclusion of the second trip, it was determined that the following four buildings represented the best candidates for further study:

1) Loma Prieta Community Center (modern reinforced bearing wall)

2) Peninsula Office Building (modern anchored and adhered veneer)

3) Two Alhambra Apartment Building (wooden apartments with masonry veneer)

4) Woodrow Hotel (older unreinforced masonry infill)

CHAPTER 3

BUILDING NO. 1 - LOMA PRIETA COMMUNITY CENTER

3.1 General Description of Building

The Loma Prieta Community Center, a one-story building covering approximately 20,000 square feet in plan, was constructed in 1987. The building was designed as a gymnasium/auditorium with classroom space in accordance with the 1979 Uniform Building Code. It was also intended to serve as a community center in times of natural disaster. It remained functional after the 1989 Loma Prieta Earthquake, and did in fact serve as an emergency relief center [28]. The building is located less than 1/2 mile from the primary fault trace on Summit Road.

The building construction consists of timber, steel and concrete masonry units (CMU). A large gymnasium is located at the center of the building with a classroom wing to the west and locker and storage rooms to the east. An overall floor plan of the building is presented in Figure 2. The roof of the gymnasium consists of plywood decking spanning to timber purlins supported by long-span steel trusses. The perimeter walls of the gymnasium are constructed of fully grouted load-bearing concrete masonry. The east and west wings are wood-framed structures. The roof structure over the two wings consists of plywood spanning to timber and glued laminated beams. The wings' roof structure is supported on interior wooden posts and load-bearing stud walls. The foundation system consists of a combination of grade-supported concrete beams and shallow spread footings. The soils beneath the building consist of a compacted silty clay fill approximately 10 feet deep over a claystone bedrock [38].

Lateral loads are intended to be resisted by the concrete masonry walls at the perimeter of the gymnasium, and by the plywood shear walls in each wing.

3.2 Description of Damage

The building was inspected visually on the afternoon of September 13, 1990. The interior and exterior walls of the building were visually inspected for signs of damage. The roof framing over the gymnasium and other structural elements exposed to view were also visually inspected.

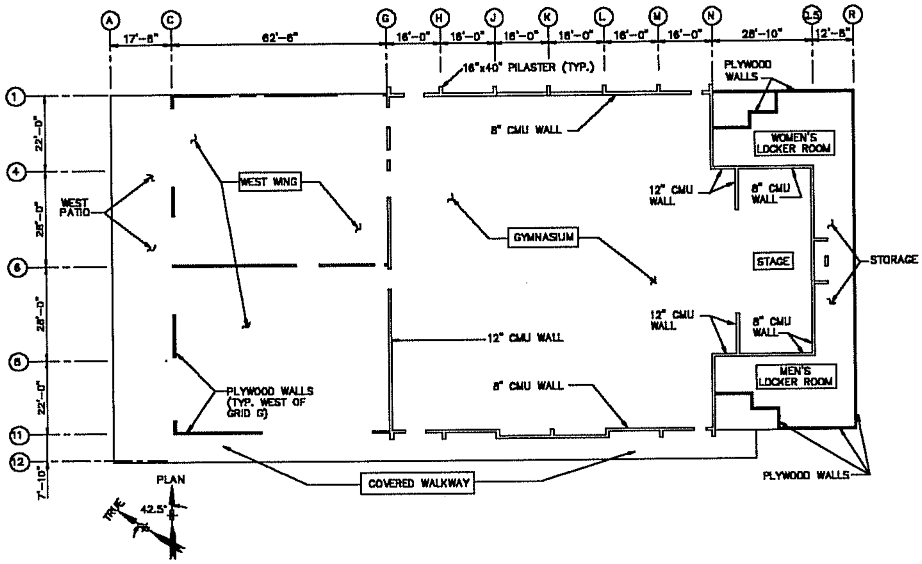

Figure 2a Loma Prieta Community Center - Building Floor Plan

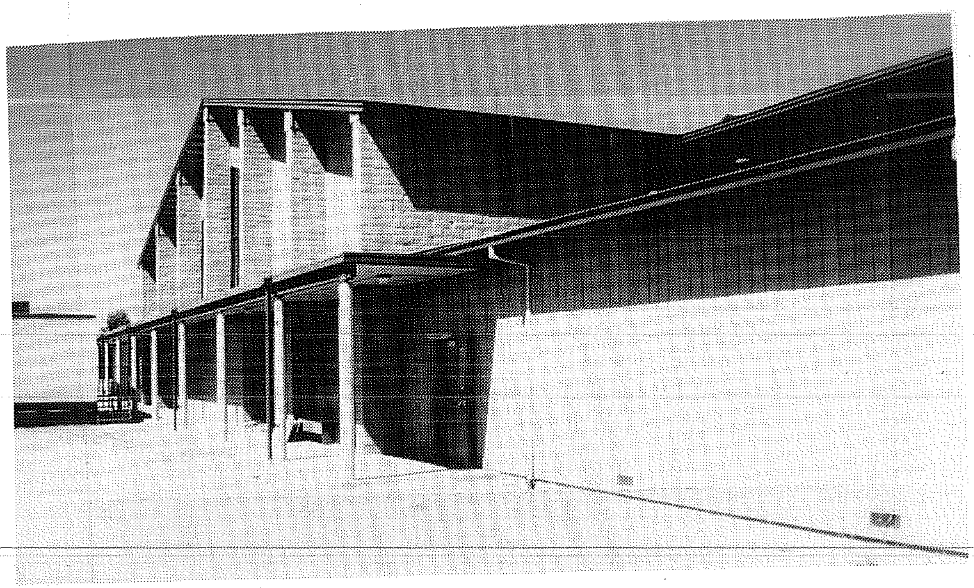

Figure 2b Loma Prieta Community Center - Overall View

The building appeared to have sustained only minor damage. A summary of the observed damage, referred to Plan North, is as follows:

o Small cracks were observed in the load-bearing masonry walls at the perimeter of the gymnasium. Refer to Figures 3 through 6.

o Two cracks, oriented in a north-south direction, were observed in the gymnasium concrete slab on grade. Lateral displacement of about 1/8 inch had occurred along these cracks, at construction joints in the slab.

o At the northeast corner of the gymnasium roof, one of the diagonal braces between the roof trusses buckled.

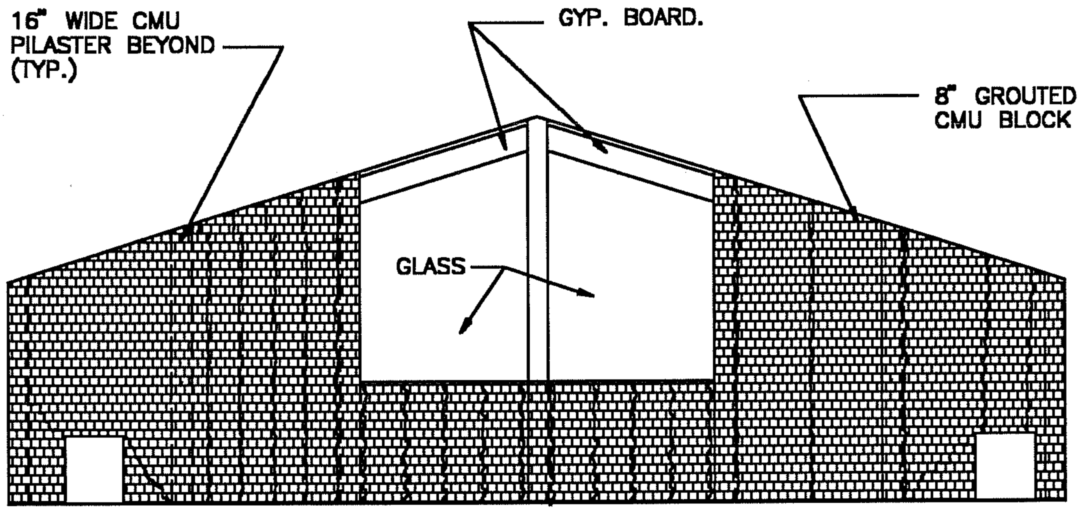

Figure 3 Loma Prieta Community Center - Observed cracks in north gymnasium wall

o Signs of water leakage were evident around the windows in the south wall of the gymnasium.

o There was evidence of slab settling at the west end of the building. In addition, exterior glass in this area broke and had to be replaced [38].

o The two wings of the building pulled away from the gymnasium walls, causing roof leaks along those joints [38].

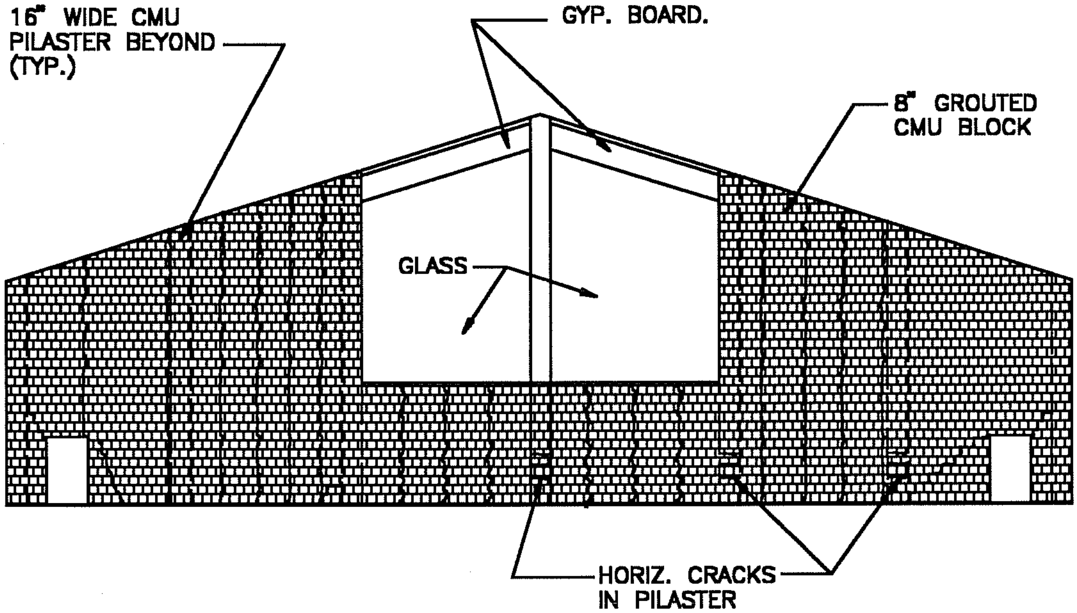

Figure 4 Loma Prieta Community Center - Observed cracks in south gymnasium wall

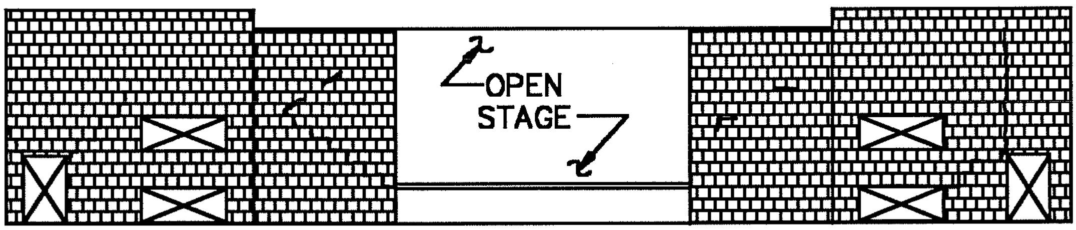

Figure 5 Loma Prieta Community Center - Observed cracks in east gymnasium wall

3.3 Selection of Ground Motion

Strong-motion records were obtained from the California Strong Motion Instrumentation Program (CSMIP) for nearby stations [8]. The Corralitos - Eureka Canyon Road Station (No. 57007), the closest station, is located approximately 10 miles southeast of the Loma Prieta Community Center. The ground motions recorded at the Corralitos station were considered to be representative of ground motions experienced at this site during the earthquake, and were therefore used in

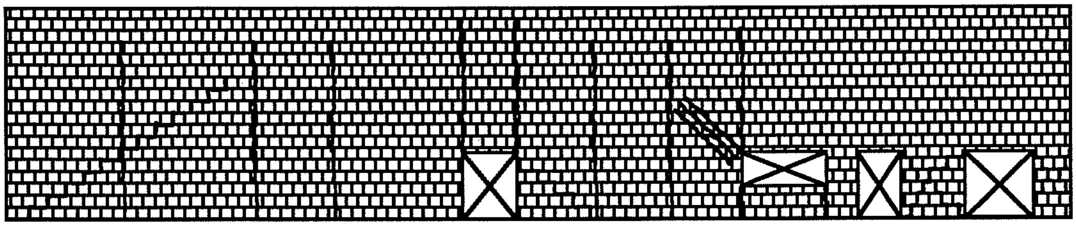

Figure 6 Loma Prieta Community Center - Observed cracks in west gymnasium wall

the analyses. Peak horizontal ground accelerations of 0.64g were recorded at the Corralitos station.

3.4 Analytical Modeling of Building

3.4.1 Selection of Computer Program. SAP90, a general purpose elastic static and dynamic finite element analysis program, was selected for analytical modeling of this building [15]. The primary reason for selecting this type of program was to capture the out-of-plane response of the masonry walls at the perimeter of the gymnasium. It was evident from the on-site inspection that this out-of-plane movement was significant during the earthquake. SAP90's three-dimensional shell elements reproduce both out-of-plane bending and in-plane membrane action. In addition, this building has a sloping roof diaphragm consisting of plywood decking; the overall behavior and in-plane shear flexibility of that diaphragm could be modeled using membrane elements.

3.4.2 Available Building Data. The original structural and architectural drawings and masonry specifications for the building were obtained. The drawings were used in determining horizontal and vertical building dimensions, sizes of framing members, and locations and thicknesses of masonry walls and plywood shear walls and decking. The masonry specifications provided information on the type of concrete masonry units, mortar, and grout used in construction. The original foundation investigation report, also available, provided information on subsurface soils at the site [38].

3.4.3 Description of Model. Using a combination of frame and shell finite elements, a three-dimensional model of the building was developed as shown in Figure 7. The plywood roof diaphragms and shear walls were modeled using membrane elements with in-plane stiffness only. The shear stiffness properties for

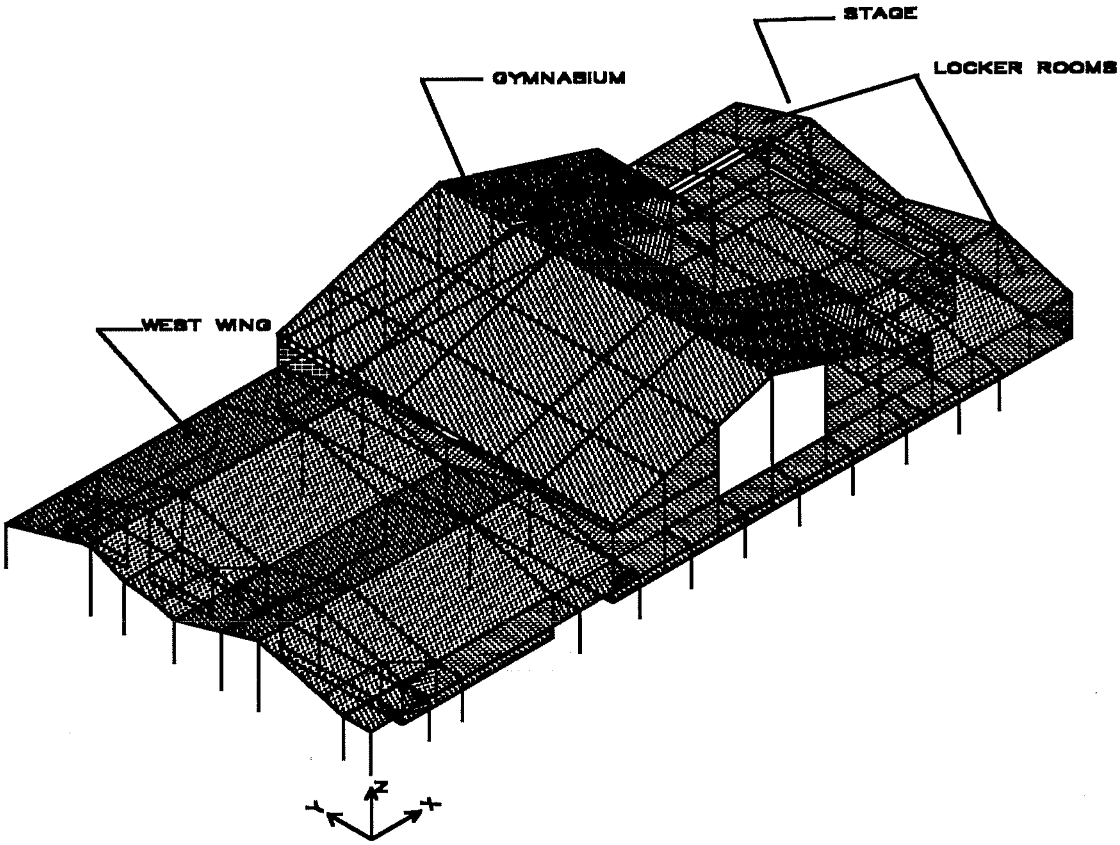

Figure 7 Loma Prieta Community Center - Three-dimensional view of the building model

these elements were based on published diaphragm deflection formulas [35]. Such formulas were not available for the plywood shear walls. However, load-deflection curves for 1/2-inch plywood shear walls with 10d nailing were obtained from the American Plywood Association [34].

The concrete masonry shear walls were modeled as shell elements to capture both their membrane and their bending behavior.

Frame elements were used to model the long-span steel roof trusses spanning north-south over the gymnasium, the 16-inch x 40-inch CMU pilasters supporting the steel roof trusses, the interior wood posts in the classroom wing, and the concrete columns supporting the exterior walkway and covered patio.

The roof and wall masses were input as nodal masses, distributed to multiple nodes to accurately model the building's response. The effects of soil-foundation flexibility were included in the model by adding springs beneath the interior and exterior walls of the building. Using the method proposed by Dobry and Gazetas [9], static stiffnesses of all degrees of freedom were estimated for each footing, and equivalent springs with appropriate horizontal, vertical and rotational stiffness values were inserted into the model.

A seismic analysis was performed using the response spectrum simultaneously for each horizontal and vertical component of the recorded ground motion. Figure 8 shows a plot of the response spectrum for the north-south and east-west directions. Figure 9 shows a plot of the response spectrum for the vertical ground motion. Modal damping coefficients of 5 percent of critical were used for each of these spectra.

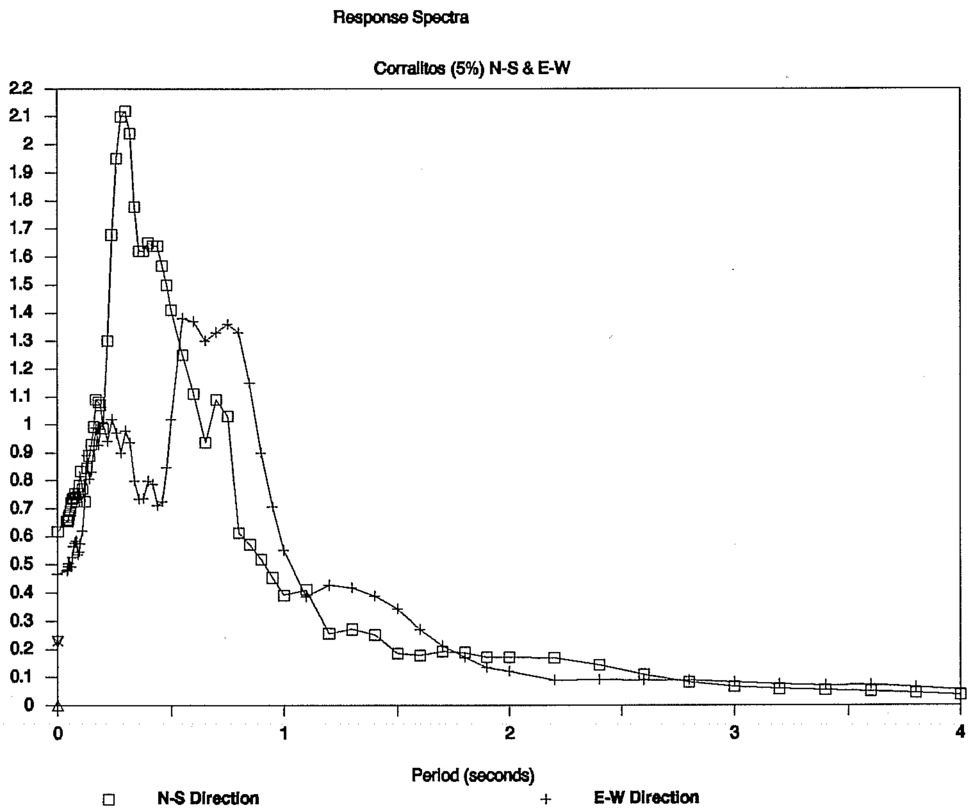

Figure 8 Loma Prieta Community Center - Corralitos Response Spectra (5% Damping) for N-S and E-W Ground Motion

Response Spectra

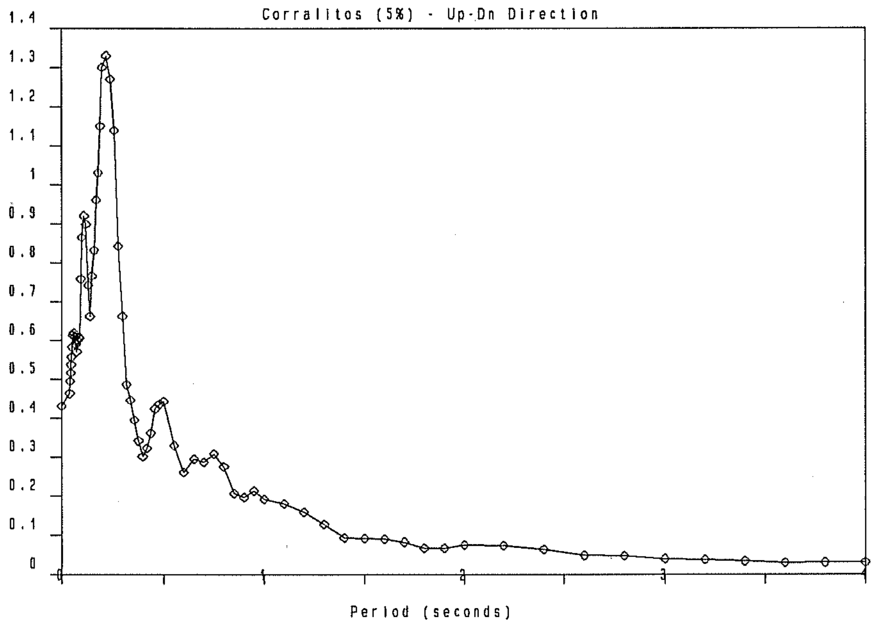

Figure 9 Loma Prieta Community Center - Corralitos Response Spectrum (5% Damping) for Vertical Ground Motion

3.5 Predicted Response

The analyses predict significant lateral displacement of the structure's west wing. At the west end of the classroom wing, the total lateral drift in the north-south direction is estimated to be 1.32 inches, equivalent to a large story drift ratio of 0.0076. The lateral displacement of the gymnasium roof in the north-south direction is estimated to be 2.43 inches. Considering the 96-foot lateral span of the roof diaphragm over the gymnasium, this equates to a lateral displacement of L/474, not significant. Figure 10 shows an envelope of the maximum lateral displacements in plan view.

The cracking shear stresses of the concrete masonry walls were predicted according to the following formula, for each masonry element:

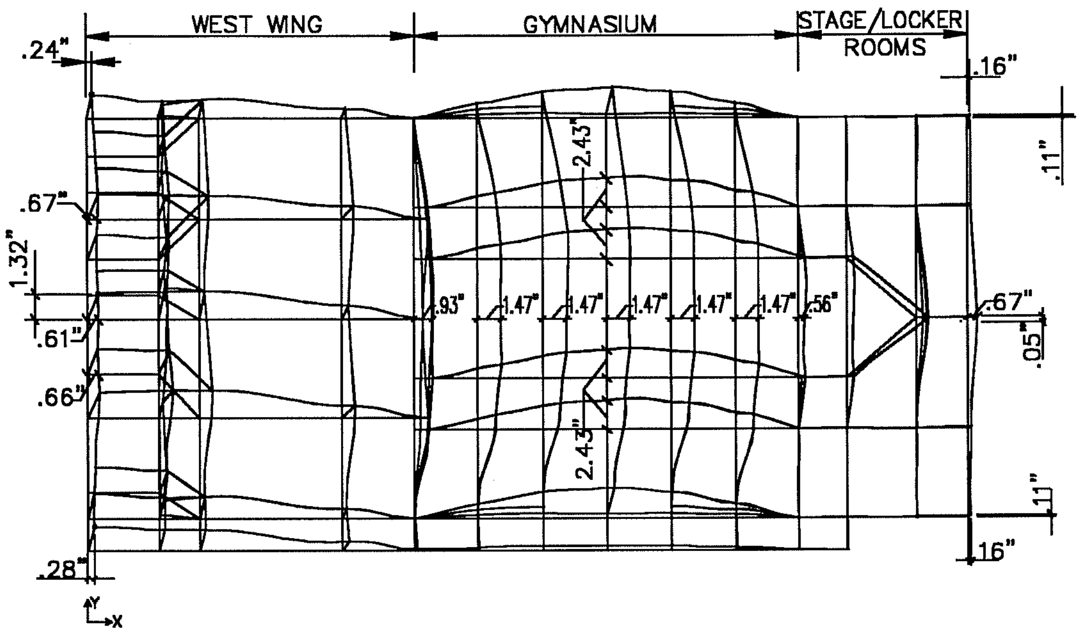

Figure 10 Loma Prieta Community Center - Envelope of Maximum Lateral Displacements in Plan View (Response Spectrum)

$$v_{cro} = \left[3.5 - 1.75 \times \left(\frac{M}{Vd} \right) \right] \times \sqrt{f'_m}$$

where $\frac{M}{Vd}$ = Aspect Ratio of the Wall

f'_m = Masonry Compressive Strength

The above expression, used in predicting cracking shear stresses, was based on testing performed at the University of California at Berkeley, and was found to be in reasonable agreement with results from subsequent research at the University of Colorado at Boulder [3]. Average axial stresses for this one-story building were found to have little effect on the cracking shear stresses of the masonry walls, and were consequently neglected in the analysis.

In Table 1, these predicted cracking shear stresses are compared to the shear stresses obtained from the SAP90 analysis. In all cases, the computed shear stresses in the walls are far less than the predicted cracking shear stresses.

Table 1 Comparison of computed shear stresses to predicted cracking shear stresses in gymnasium walls.

Wall Location	SAP 90 Computed Shear Stress psi	Predicted Cracking Shear Stress psi	Ratio of Computed Shear Stress to Predicted Cracking Shear Stress
West Gymnasium Wall	28	178	0.16
East Gymnasium Wall	27	160	0.17
East Stage Wall	7	170	0.04
North Gymnasium Wall	21	182	0.12
South Gymnasium Wall	23	182	0.13
North Stage Wall	37	154	0.24
South Stage Wall	37	154	0.24

Out-of-plane flexural stresses (tension parallel to head joints) were computed for the north and south walls of the gymnasium. The maximum stresses were found to be greater than 500 psi for most of the panels, and as high as 900 psi for several panels. The flexural cracking stress was estimated by increasing the 1988 UBC allowable flexural cracking stress by a factor of three. The observed values exceed the estimated flexural cracking stress of 240 psi by a factor of 2 to 4, indicating that those walls would crack vertically.

The moments and shear forces in the concrete masonry pilasters along the north and south walls of the gymnasium were also obtained from SAP90. In all cases, computed moments at the bases of the pilasters exceeded their cracking moment. The computed shear stresses at the bases of the pilasters were below the predicted cracking shear stress in all cases. The results for the pilasters along the south wall of the gymnasium are presented in Tables 2 and 3. The results for the pilasters along the north wall are similar.

The effect of the earthquake on the long-span steel roof trusses was of interest due to the high vertical acceleration component of the Loma Prieta

Table 2 Comparison of computed moments to predicted cracking moments for the pilasters along the south gymnasium wall.

Pilaster Location	SAP 90 Computed Moment in-k	Predicted Cracking Moment in-k	Ratio of Computed Moment to Predicted Cracking Moment
Grid H-1	5,708	1,429	3.99
Grid J-1	8,036	1,429	5.62
Grid K-1	6,690	1,429	4.68
Grid L-1	8,098	1,429	5.67
Grid M-1	5,857	1,429	4.10

Table 3 Comparison of computed shear stresses to predicted cracking shear stresses for the pilasters along the south gymnasium wall.

Pilaster Location	SAP 90 Computed Shear Stress psi	Predicted Cracking Shear Stress psi	Ratio of Computed Shear Stress to Predicted Cracking Shear Stress
Grid H-1	35.7	96	0.37
Grid J-1	50.5	96	0.53
Grid K-1	37.4	96	0.39
Grid L-1	50.8	96	0.53
Grid M-1	36.1	96	0.38

Earthquake, and also due to coupling between the horizontal and vertical responses of the sloping roof. At the Corralitos Station, a maximum vertical acceleration of 0.43g was recorded. In order to determine the response of the roof trusses during the earthquake, SAP90 computer analyses were made to determine the response due to gravity loads, and to the simultaneous application of one vertical and two horizontal components of ground motion. The results are summarized in Table 4. It can be seen that the ground motion had a significant effect on the response of the roof trusses at column lines H, L and M.

Table 4 Comparison of midspan roof truss deflections under gravity load versus response spectrum.

Roof Truss Location	(1) Midspan Deflection Under Gravity Load Only inches	(2) Midspan Deflection Under Response Spectrum Only inches	Ratio of (2) to (1)
Column Line H	0.89	0.76	0.85
Column Line J	1.03	0.44	0.43
Column Line K	1.04	0.23	0.22
Column Line L	1.03	0.98	0.95
Column Line M	0.86	1.86	2.16

Soil-foundation flexibility had only a minor effect on the overall building response. The fundamental period of the building increased from 0.29 to 0.30 seconds, and damping increased from 3% to 5% of critical. The increase in period results from the flexibility of the soil-foundation system; the change in damping results from increased energy dissipation in the soil due to radiation and material damping [23].

3.6 Comparison of Predicted Response with Observed Damage

In general, predicted responses agreed well with observed damage.

1) The reported damage to the west wall of the classroom wing, which included broken windows, can be attributed to the very large lateral displacement of the west wing structure in the north-south direction. In addition, it was reported that the east and west wings of the building pulled away from the east and west walls of the gymnasium resulting in roof leaks along these interfaces [28]. Again, the significant north-south movement of the structure, particularly evident in the west wing, caused large shear and tensile forces in the connections between the gymnasium and these two wings.

2) The observed cracking in the masonry walls at the perimeter of the gymnasium is generally consistent with the predicted response. The predicted cracking shear stresses for the masonry walls were much greater than the actual computed stresses. Except at locations of door and window openings, where stress concentrations can be expected, very few diagonal cracks were observed in the walls.

3) The observed vertical cracks in the north and south walls of the gymnasium are the result of large out-of-plane bending moments. The 8-inch wall spans horizontally in these locations to the pilasters, and the pilasters themselves show significant out-of-plane movement in the north-south direction. The computed stresses in these walls ranged from 2 to 4 times their predicted flexural cracking stress.

4) The flexural cracking observed near the bottom of the pilasters at the south wall is consistent with the predicted response. The computed moments in the pilasters exceed the predicted cracking moments by a factor of 4 to 6.

3.7 Evaluation of 1988 UBC Design Provisions as Applied to this Building

Minimum seismic design requirements are outlined in Chapter 23, Section 2312 of the 1988 Uniform Building Code (UBC) [37]. The UBC allows design by either a dynamic lateral force procedure or an equivalent static lateral force procedure. It is common among practicing structural engineers in California to use the static lateral force procedure for the analysis of buildings not having significant vertical or horizontal irregularities. Douglas Vossbrinck, the structural engineer of record, confirmed that this building was originally designed using the equivalent static lateral force method in accordance with the 1979 UBC [38]. The objective of this report was not to evaluate the original design, but to make observations regarding provisions of the 1988 UBC as applied to this building.

To perform an equivalent static analysis in accordance with the 1988 UBC, the first step is to compute the seismic base shear. The total base shear is then distributed to each level of the structure as prescribed by the UBC. In order to perform the UBC static analysis, this building was divided into three separate sections: Section #1 - West Classroom Wing; Section #2 - Gymnasium; and Section #3 - East Wing (Stage/Locker Rooms). Seismic forces were then computed for each section as follows:

$$\text{Seismic Base Shear, } V = \frac{ZIC_W}{R_w} \qquad \text{(UBC formula 12-1)}$$

The values used to compute the seismic base shear for each building section are summarized in Table 5. The seismic base shear was distributed to each section of the building in both directions according to the structure's actual weight distribution. Figure 11 shows the distribution of the base shear to the building in the north-south direction. The corresponding lateral displacements of the structure are shown in Figure 12. The force distribution and corresponding lateral displacements in the east-west direction are presented in Figures 13 and 14.

Table 5 Computation of Seismic Base Shear

1988 UBC Criteria	Section #1	Section #2	Section #3
Fundamental Period, T	0.15 sec.	0.24 sec.	0.19 sec.
Seismic Zone Factor, Z	0.40	0.40	0.40
Site Coefficient, S	1.5	1.5	1.5
Importance Factor, I	1.25	1.25	1.25
Numerical Coefficient, C	2.75	2.75	2.75
Numerical Coefficient, Rw	8	6	6
Total Seismic Dead Load, W	202 kips	745 kips	191 kips
Seismic Base Shear, V	34.7 kips	171 kips	43.8 kips

Several observations can be made regarding the distribution of lateral forces to the shear walls in the building under the UBC equivalent static analysis. First, the UBC static force applied to the west wing is 34.7 kips. Based on SAP90 results, only 10.6 kips (31%) is transferred to the shear wall along Column Line C. The balance of the force (24.1 kips, or 69%) is transferred to the masonry shear wall along Column Line G. It is apparent that the center gymnasium structure provides lateral support to the west wing of the building. The second observation has to do

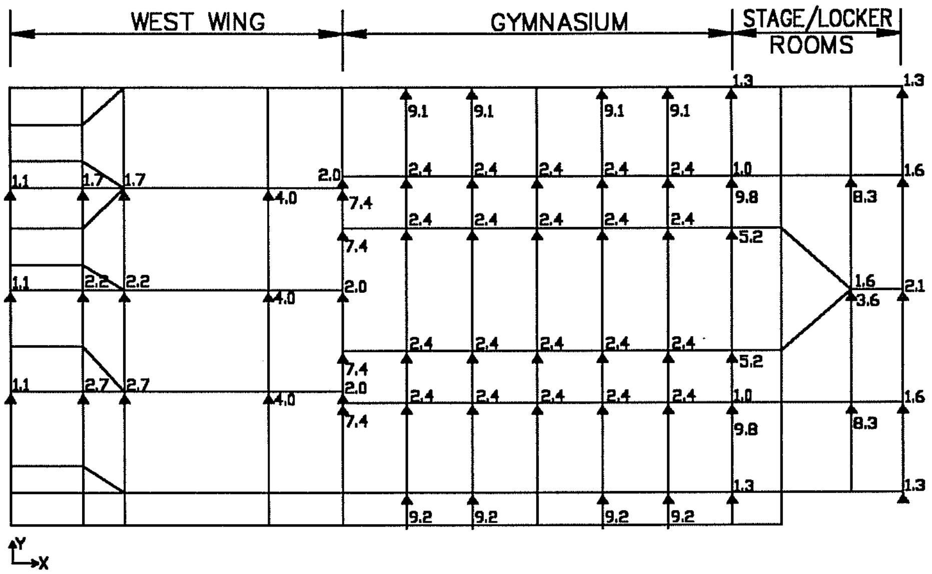

Figure 11 Loma Prieta Community Center - Distribution of UBC Seismic Base Shear to the Building in the N-S Direction (Force in Kips)

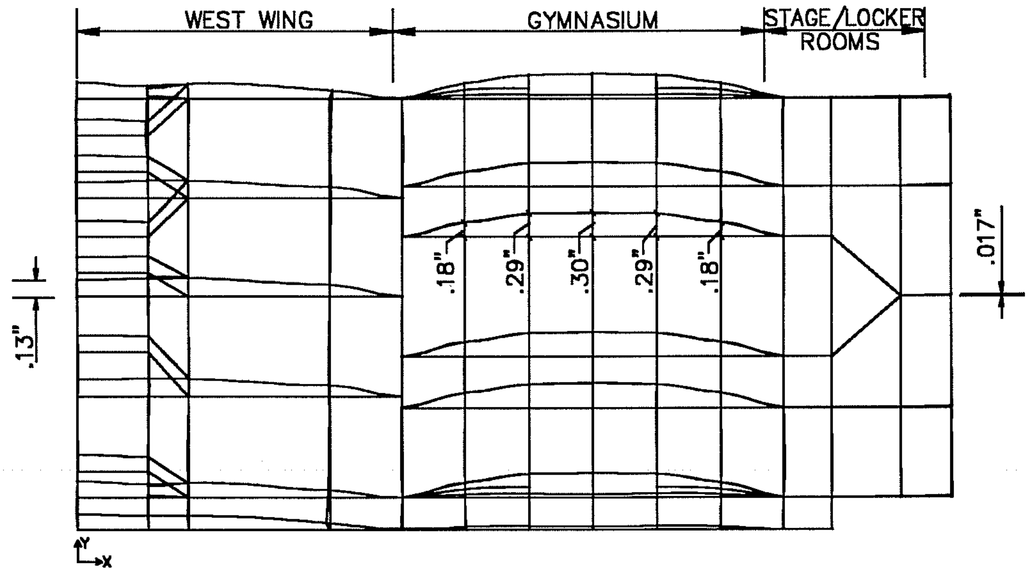

Figure 12 Loma Prieta Community Center - Lateral Displacement of the Building in the N-S Direction (UBC)

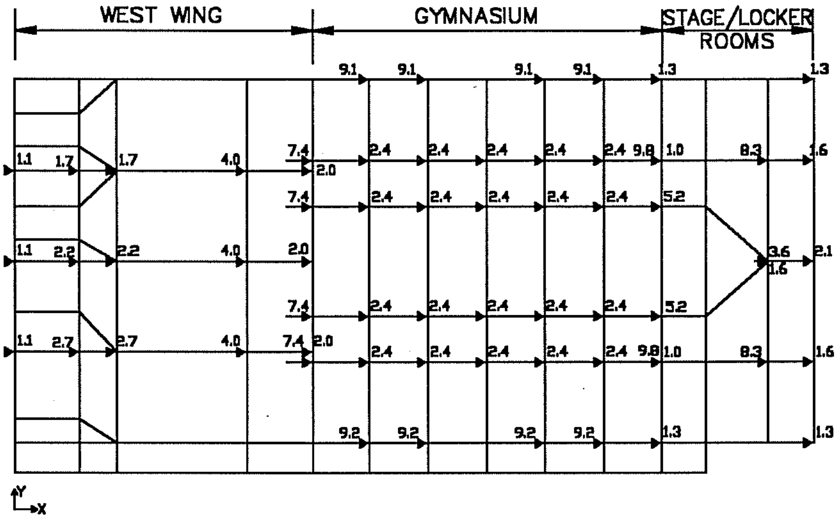

Figure 13 Loma Prieta Community Center - Distribution of UBC Seismic Base Shear to the Building in the E-W Direction (Force in Kips)

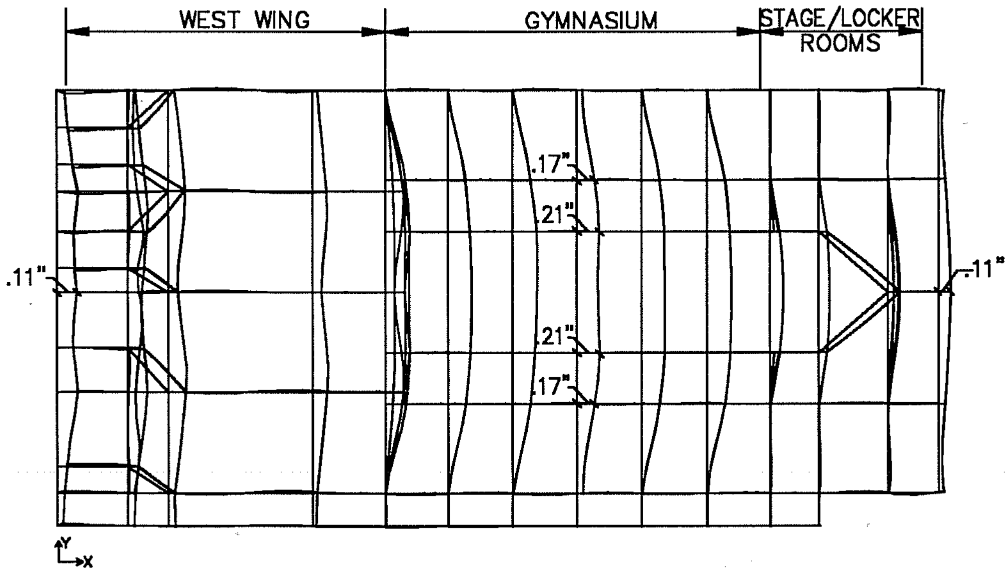

Figure 14 Loma Prieta Community Center - Lateral Displacement of the Building in the E-W Direction (UBC)

with the transfer of N-S lateral forces through the roof diaphragm to the pilasters along the east and west gymnasium walls. Of the applied force of 120.6 kips, one-third is resisted by the pilasters and two-thirds is resisted by the east and west walls. A totally rigid diaphragm would transfer almost 100% of the applied force to the east and west gymnasium walls. A totally flexible diaphragm would transfer approximately 20 kips each, or one-sixth, to the east and west walls. The plywood decking evidently provides a level of in-plane stiffness somewhere between those two extremes.

3.8 Comparison of Response Spectrum and 1988 UBC Responses

The results of the elastic dynamic analysis using response spectra for the actual ground motions recorded in the vicinity of the site differ greatly from those results obtained based on an equivalent lateral static analysis in accordance with the 1988 UBC. A comparison of the actual response spectrum for the N-S direction with the implied UBC spectrum, as shown in Figure 15, shows the large difference in accelerations between the two spectra, particularly in the short period range which is representative of the lower modes for this building. The lower two fundamental modes for this building in the north-south direction are 0.30 and 0.28 seconds for the gymnasium and west wing respectively.

Table 6 presents a summary of the forces in the shear walls and pilasters for the UBC equivalent static forces versus the results from the dynamic analysis. It is obvious that significant dynamic response of the structure occurred in the west wing of the building in the N-S direction. The shear transmitted to the wall along Column Line C is 9.9 times greater for the dynamic analysis as compared to the UBC static forces. The ratio of dynamic to UBC applied forces is also significant for the forces transferred to the pilasters.

3.9 Probable Response of Building in Stronger Earthquake

The Loma Prieta Earthquake represented a significant event for this building, due to its proximity to the epicenter. Peak horizontal ground accelerations of 0.64g were recorded 10 miles southeast, at the Corralitos Station. Due to the magnitude of the Loma Prieta event, a significantly stronger earthquake is not anticipated at this site. However, several observations can be made pertaining to the probable response of the building if subjected to a stronger earthquake.

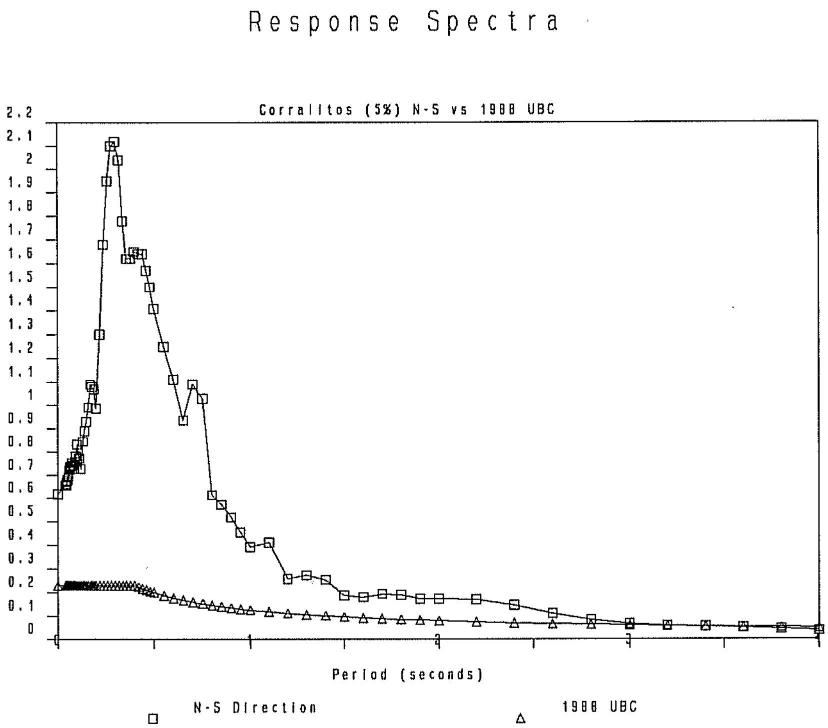

Figure 15 Loma Prieta Community Center - Response Spectrum for N-S Ground Motion versus Implied 1988 UBC Spectrum

Shear cracking of the concrete masonry walls at the perimeter of the gymnasium would not likely be a problem. The predicted cracking stresses for these walls exceeded the computed shear stresses by a factor of five for the Loma Prieta Earthquake. However, some isolated diagonal cracking would be expected around wall openings.

The west classroom wing of the building would most likely be damaged, due to significant response in the north-south direction. Glass would again break along the west wall of the building, accompanied by damage to the interior walls in this wing. Also, significant forces would develop in the connections which tie the east and west classroom wings to the main gymnasium structure. Damage to these connections would likely occur, and the roof over the east and west wings would pull away from the gymnasium walls, as occurred during the Loma Prieta event.

Table 6 Distribution of lateral shear forces to shear walls and pilasters

Wall/Pilaster Location	Orientation	1988 UBC	Dynamic Analysis	Ratio of Dynamic Analysis to 1988 UBC
Column Line C	N-S	10.6 kips	105 kips	9.9
Column Line G	N-S	92	392	4.3
Column Line H	N-S	7.4	54	7.3
Column Line J	N-S	8.9	76	8.5
Column Line K	N-S	6.2	53	8.5
Column Line L	N-S	8.9	77	8.6
Column Line M	N-S	7.6	55	7.2
Column Line N	N-S	75	257	3.4
Column Line Q.5	N-S	26	36	1.4
Column Line R	N-S	3.8	12	3.2
Column Line 1	E-W	65	219	3.4
Column Line 4	E-W	40	107	2.7
Column Line 6	E-W	37	274	7.4
Column Line 8	E-W	39	107	2.7
Column Line 11	E-W	67	258	3.9

3.10 Summary of Findings

The Loma Prieta Community Center received a significant test from this event, whose epicenter was located approximately four miles from the building site. Peak horizontal ground accelerations recorded at the Corralitos Station, 10 miles southeast of the building, were measured at 0.64g. The load-bearing concrete masonry walls performed extremely well and were only slightly damaged. The most noticeable damage occurred in the west classroom wing of the building, which is wood-framed with plywood shear walls. Significant north-south lateral displacements occurred in this wing of the building, accompanied by some racking damage and glass breakage in that wing.

Observations regarding the model of the building used in the analyses and the building's overall response and behavior during the earthquake are presented below.

1) Analytical Model of the Building

   a) The dynamic analysis reasonably modeled the response of the building, as predicted damage was consistent with observed damage.

   b) Soil-foundation flexibility did not have a significant effect on the results. The building's fundamental period increased approximately 5%. The effective damping factor for the structure-foundation system increased from 3%, when not accounting for soil-foundation effects, to 5% including those effects.

2) Aspects of Behavior

   a) The design intent of the <u>Uniform Building Code</u> was satisfied. The building did not collapse under a strong earthquake, and received only minor non-structural damage.

   b) The computed dynamic shear stresses were significantly less than predicted cracking shear stresses for all of the masonry walls.

   c) The computed dynamic out-of-plane flexural stresses for the north and south walls of the gymnasium were found to be 2 to

4 times greater than the predicted flexural cracking stress resulting in the observed vertical cracks.

d) The computed dynamic moments in the CMU pilasters would be expected to cause flexural cracking, as observed in several locations.

e) The long-span trusses supporting the sloping roof of the gymnasium responded strongly to the simultaneous application of one vertical and two horizontal components of ground motion.

f) The west classroom wing responded strongly in the north--south direction, as can be seen by comparing the magnitude of the response spectrum shear in the west wall to the UBC static lateral shear in the same wall. The dynamic response is roughly 10 times greater than that for the UBC loads. The roof of the west wing pulled slightly away from the gymnasium wall, resulting in water leakage along this joint.

3.11 Recommendations

The following recommendations are offered regarding the design of similar masonry structures:

1) The structural drawings clearly detailed the structural connections between the gymnasium and the east and west wings and the connections of the long-span roof trusses to the tops of the masonry pilasters. The dynamic analyses show that significant forces developed at those connections, particularly in the west wing. The connections performed extremely well under such a strong earthquake. The importance of providing adequate ties among structural elements, and of adequately detailing connections on the structural drawings, cannot be over-emphasized.

2) A modern general-purpose computer program, with good dynamic analysis and graphic capabilities, would aid the designer in better predicting the seismic response of irregular buildings, such as the Loma Prieta Community Center. The response of structures with significant irregularities in mass and/or stiffness is difficult to predict

using an equivalent lateral static force analysis. The large out-of--plane bending of the north and south walls of the gymnasium, the large vertical movements of the roof trusses, and the large north--south displacement of the west classroom wing could not have been predicted using the UBC equivalent static method. In addition, a computer program with good graphics capabilities enables the designer to view the completed model to verify the accuracy of the input data, and to visualize the deflected shape of the structure in different modes.

CHAPTER 4

BUILDING NO. 2 - PENINSULA OFFICE BUILDING

4.1 General Description of Building

The Peninsula Office Building covers approximately 29,000 square feet in plan. It has four stories plus a below-grade basement level. The building was constructed in 1984 and designed in accordance with the 1979 Uniform Building Code. At the request of the current owner, the building's name, location, and current use have been withheld from this report.

The building is a steel-framed structure with an exterior facade of brick masonry. The exterior facade is constructed using a combination of "full brick" (thickness = 3.625 inches) and "thin brick" (thickness = 1 inch). The building superstructure above ground level consists of structural steel beams, girders, columns and bracing members. Figure 16 shows a typical framing plan. Light-weight insulating fill over metal decking is provided at the roof, and 4-inch concrete fill over 1-1/2 inch composite metal decking is provided at levels two through four. The ground level has a 12-inch thick concrete flat slab. The perimeter basement walls are cast-in-place concrete. The foundation system, shown in Figure 17, consists of continuous strip footings beneath the perimeter basement walls, and shallow spread footings beneath the interior columns. Lateral seismic forces are intended to be resisted by four V-braced frames in each direction, as shown in Figure 18. The soils beneath the building consist of stiff to very stiff clays and silts with layers of medium dense sands down to the boring depth of 20 feet.

4.2 Description of Damage

Visual inspection of the building was conducted on the afternoon of September 15, 1990. The exterior brick veneer and building interior were visually inspected for cracks and other signs of damage. Since most areas of the building were finished out at the time of the inspection, observation of structural elements was limited. However, the interior non-structural walls were checked for signs of distress which might indicate significant structural movements.

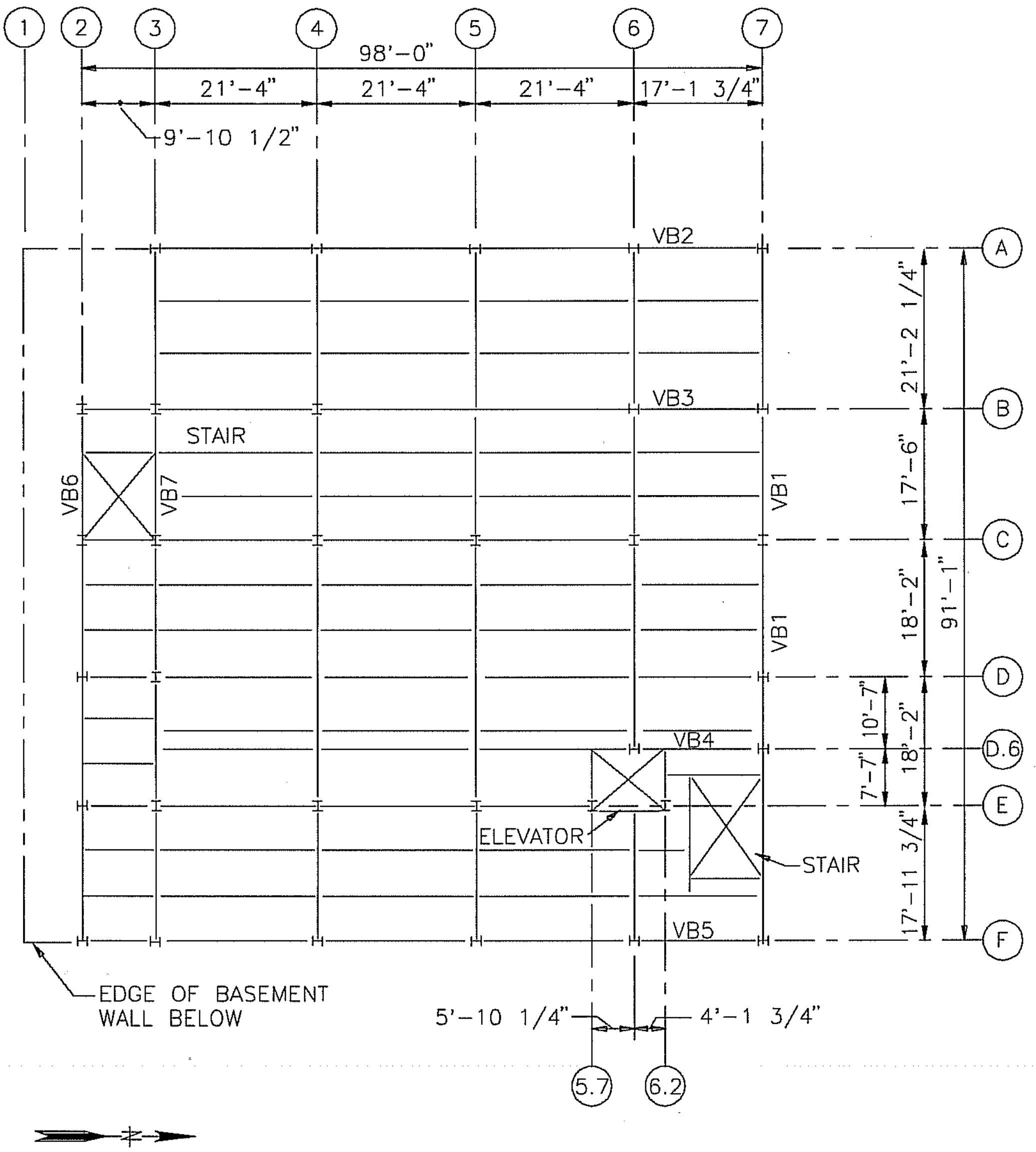

Figure 16 Peninsula Office Building - Typical Framing Plan

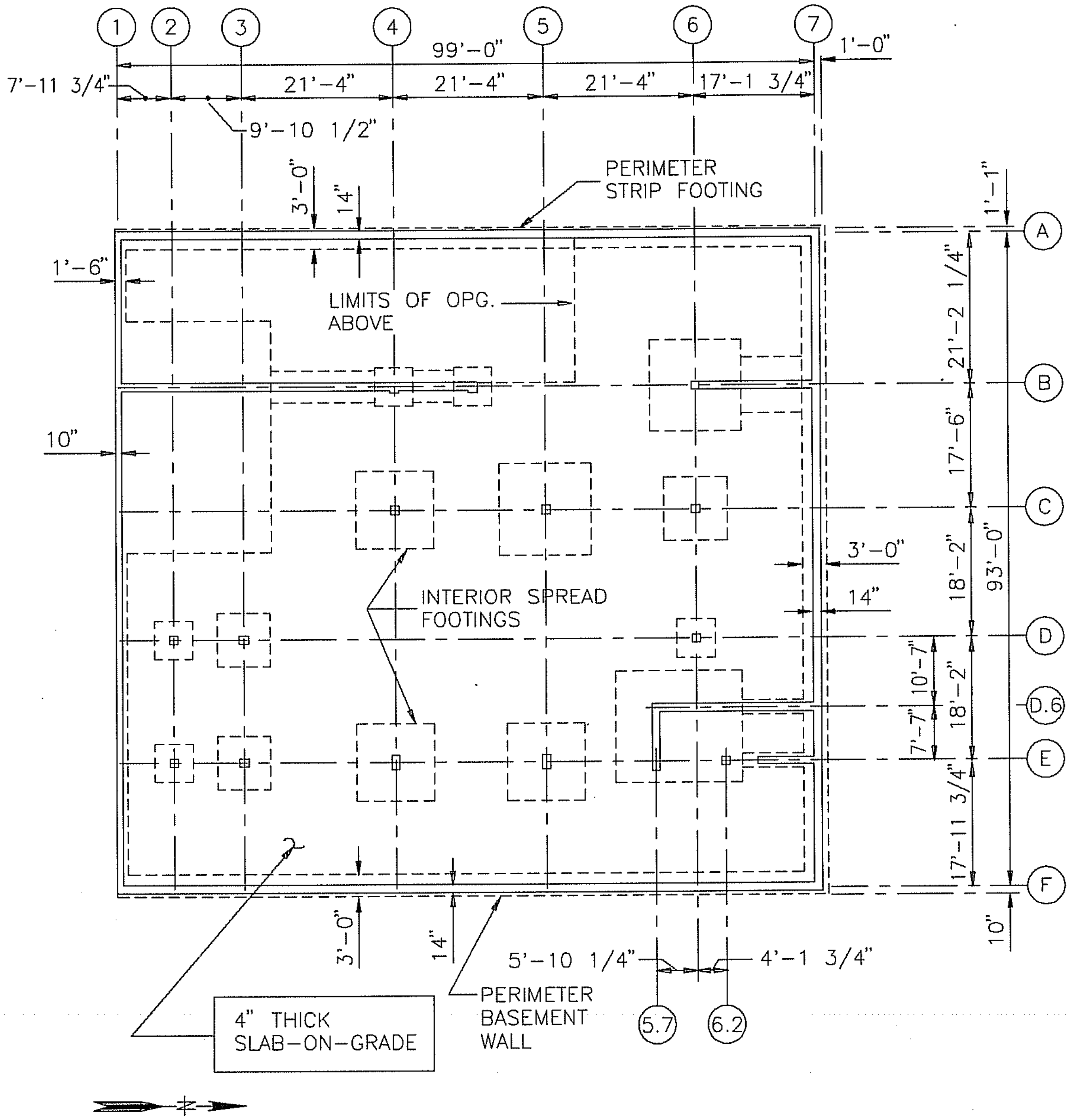

Figure 17 Peninsula Office Building - Foundation Plan

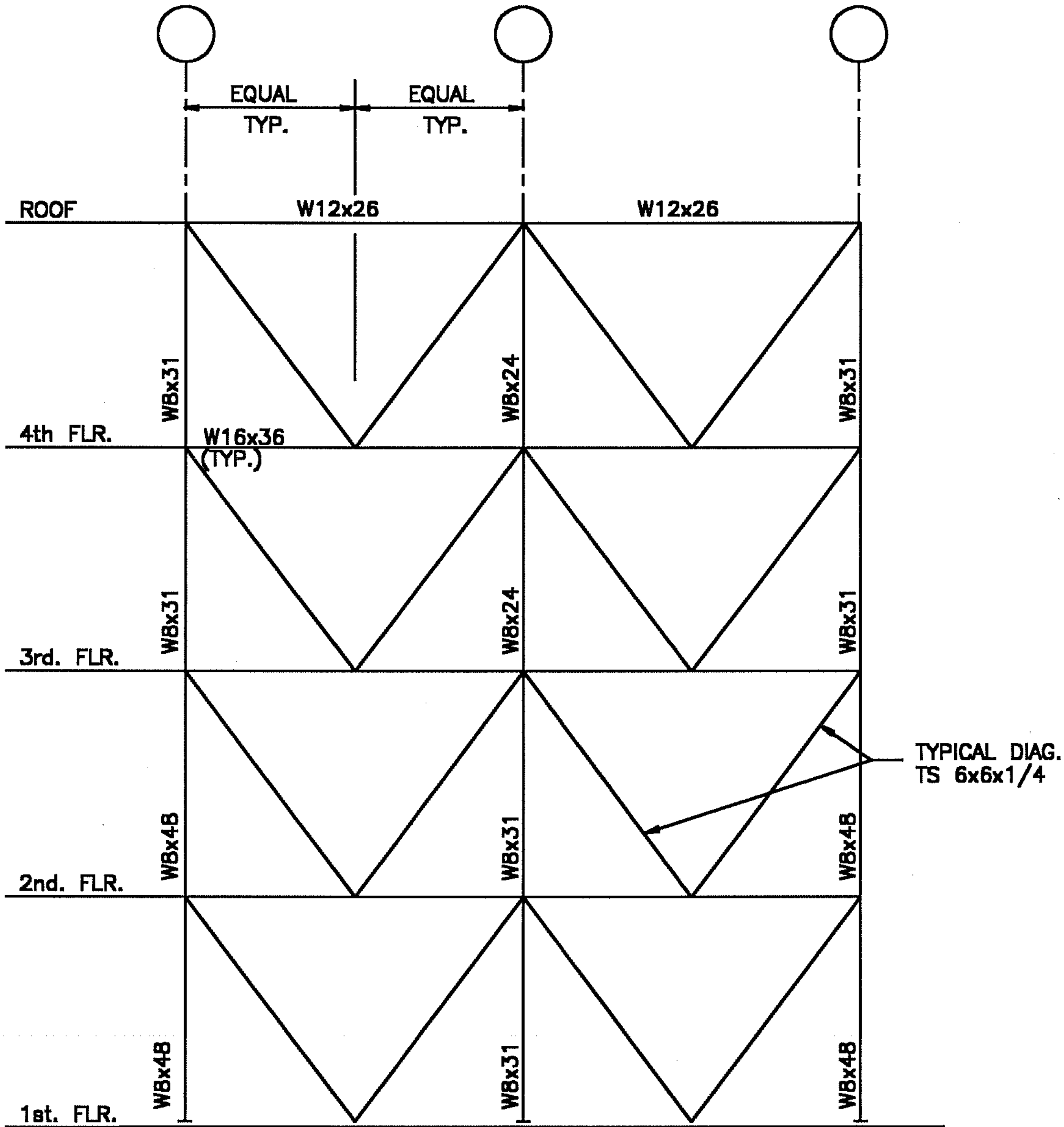

Figure 18 Peninsula Office Building - Typical V-Braced Frames

The building appeared to have sustained only minor non-structural damage from the earthquake. Diagonal cracks, 0.01 inches wide, were observed in the brick veneer on the east and west sides of the building at level one. This cracking was particularly evident in the east wall at the southeast corner. Some spalling of the brick veneer was observed on the west side of the building at level two in the vicinity of column A-5. Also, some isolated cracking of the interior sheetrock walls was observed, mainly at the corners of doors. Figures 19 and 20 show the observed cracking in the brick veneer at the east and west exterior walls, respectively.

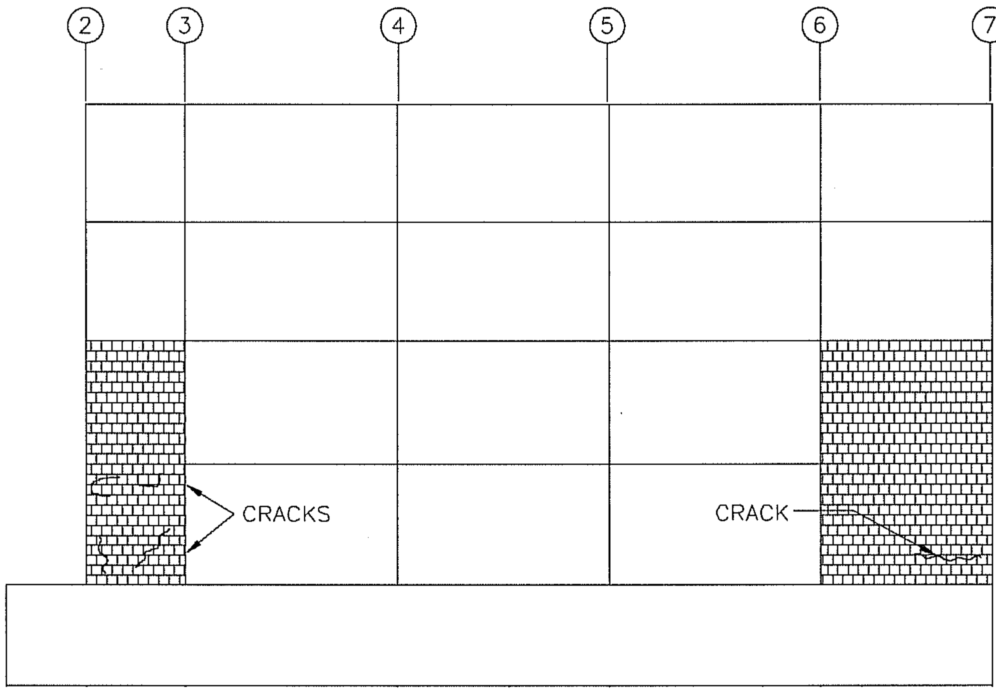

Figure 19 Peninsula Office Building - Observed Cracks in Brick Veneer on the East Wall

4.3 Selection of Ground Motion

Strong-motion records were obtained from the California Strong Motion Instrumentation Program (CSMIP) for several stations on the San Francisco Peninsula [8]. In terms of the building's general location and underlying soil, the Palo Alto 2-Story Office Building record (Station No. 58264) was the most

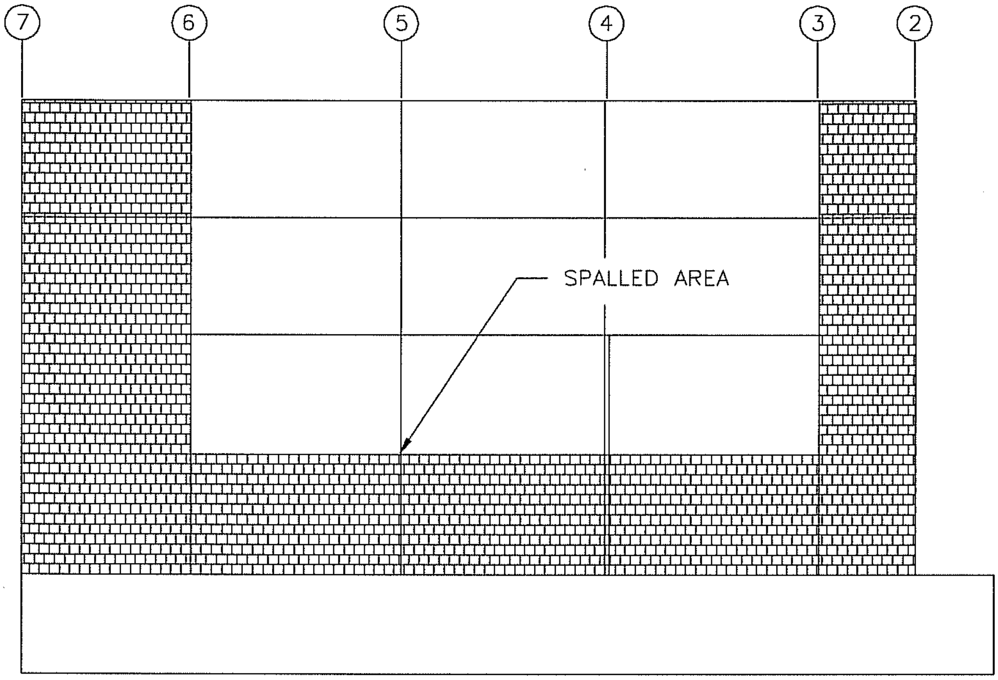

Figure 20 Peninsula Office Building - Observed Cracks in Brick Veneer on the West Wall

representative ground motion record and was therefore selected for use. Peak horizontal ground accelerations of 0.21g were recorded at the Palo Alto station.

4.4 Analytical Modeling of Building

4.4.1 Selection of Computer Program. ETABS, a special purpose elastic static and dynamic building analysis and design program, was selected for analytical modeling of this building [14]. The cast-in-place concrete floors at each level of the building provide rigid in-plane diaphragms, as assumed by ETABS. Also, since the building was only slightly damaged, a linear elastic analysis as performed by ETABS was appropriate here. Additional features of ETABS are its graphics and its ability to perform an equivalent static lateral analysis in accordance with the 1988 Uniform Building Code (UBC) [37].

4.4.2 Available Building Data. The original structural and architectural drawings and masonry specifications for the building were obtained. The drawings were used in defining horizontal and vertical building dimensions, sizes of beams, columns and bracing elements, thicknesses of concrete walls, sizes of footings and details of the exterior masonry veneer. The masonry specifications provided information on the type of brick and mortar used in construction, and on the components of the masonry veneer system. The original foundation investigation report was also available which provided information on subsurface soils at the site.

4.4.3 Description of Model: The building was modeled as a three-dimensional assemblage of beams, columns, braces and wall elements. The concrete floor slabs at each level were modeled as horizontal diaphragm slabs, rigid in their own planes. The bases of all steel columns at the ground floor and the ends of all diagonal braces were assumed to be pinned, consistent with the details presented in the structural drawings. Figure 21 shows a three-dimensional view of the building model.

The mass at each floor level was computed based on the dead loads plus a fraction of the floor live load to account for desks, file cabinets and other office equipment in place at the time of the earthquake. This mass was uniformly distributed over each floor level. Additional mass was added along the perimeter of the building at each level to account for the exterior brick veneer and glazing system above level one, and for the perimeter concrete basement wall below level one.

The effects of soil-foundation flexibility were considered by adding a "dummy" story to the structure below the foundation level. Using the method proposed by Dobry and Gazetas [9], static stiffnesses were estimated for all degrees of freedom for each independent foundation. Stiffnesses were first computed for the individual interior spread footings. Next, separate stiffness factors were computed for the perimeter basement walls, which were modeled as a "box." Based on these stiffness values, equivalent column properties were computed and included in the model.

The lateral seismic analysis was performed using a response spectrum, as shown in Figure 22, for the north-south direction of the Palo Alto ground motion. Modal damping of 5% was used for all modes including the effects of the structure-foundation system, as recommended by the NEHRP document [23]. The on-site inspection of the building revealed no cracking or other signs of distress in the north or south walls of the building. Because cracking of the east and west walls was observed, primarily as a result of north-south ground motion, the

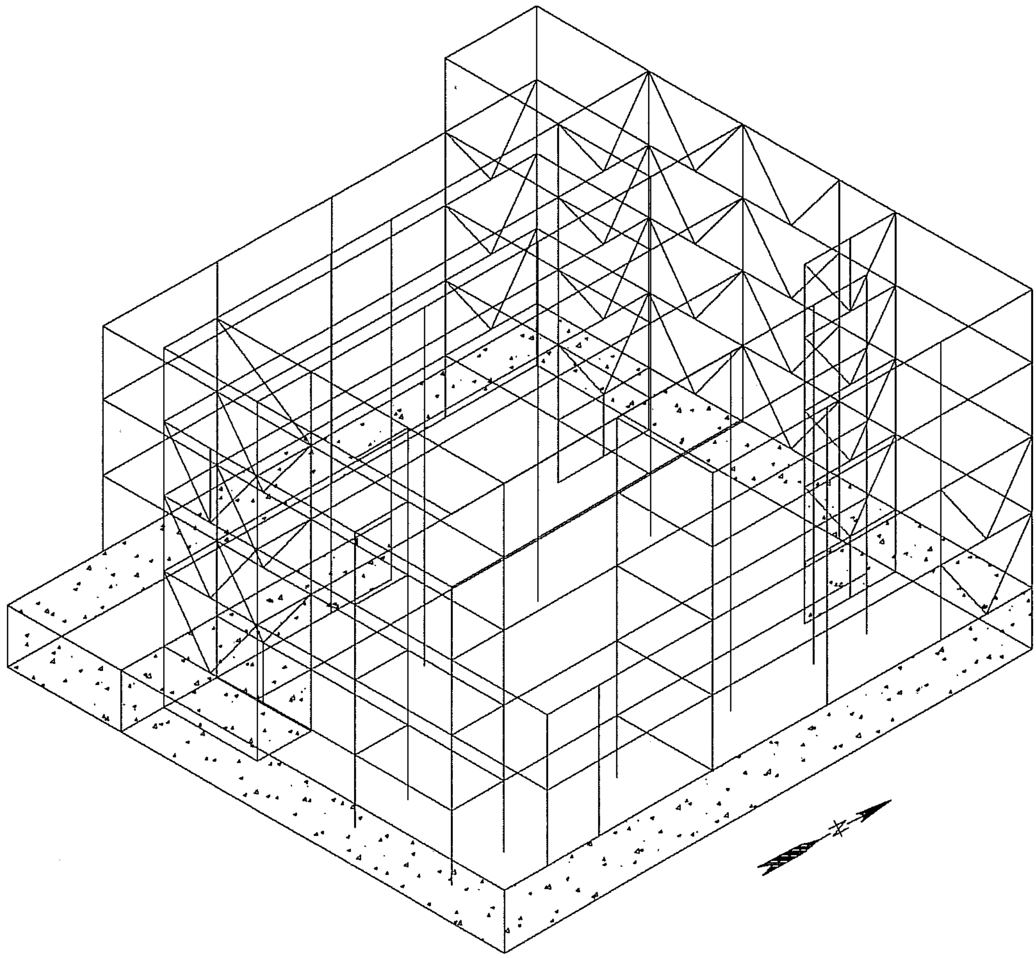

Figure 21 Peninsula Office Building - Three-Dimensional View of Building Model

north-south response was therefore of primary interest. Two separate lateral seismic analyses of the building were performed using north-south ground motion input in the north-south direction. First, the building was analyzed neglecting any stiffness contribution from the exterior brick veneer panels. Next, the building was analyzed with the exterior brick panels included in the model as shear panels.

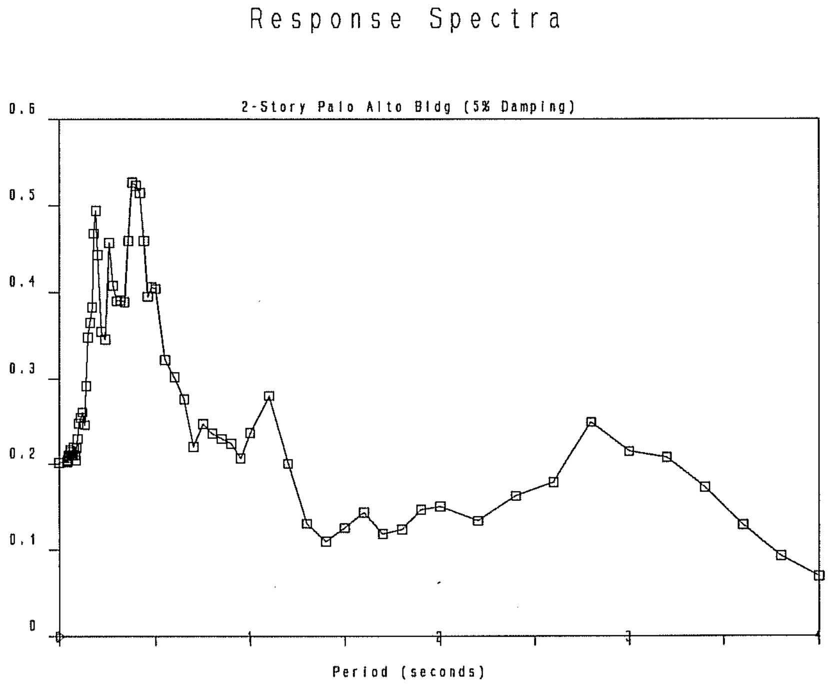

Figure 22 Peninsula Office Building - Response Spectrum for N-S Ground Motion with 5% Damping

4.5 Predicted Response

Neglecting any stiffness contribution from the exterior masonry veneer, the total building drift measured at the roof level was calculated as 1.9 inches from the Loma Prieta event. The maximum story drift ratio was 0.0047, between levels 3 and 4. The fundamental period of the building was 0.75 seconds, and the base shear was estimated at 846 kips.

Including the stiffness of the exterior masonry veneer panels as shown in Figure 23, the total building drift at the roof level was reduced to 1.3 inches, and the maximum story drift ratio, to 0.0035. The fundamental period of the building

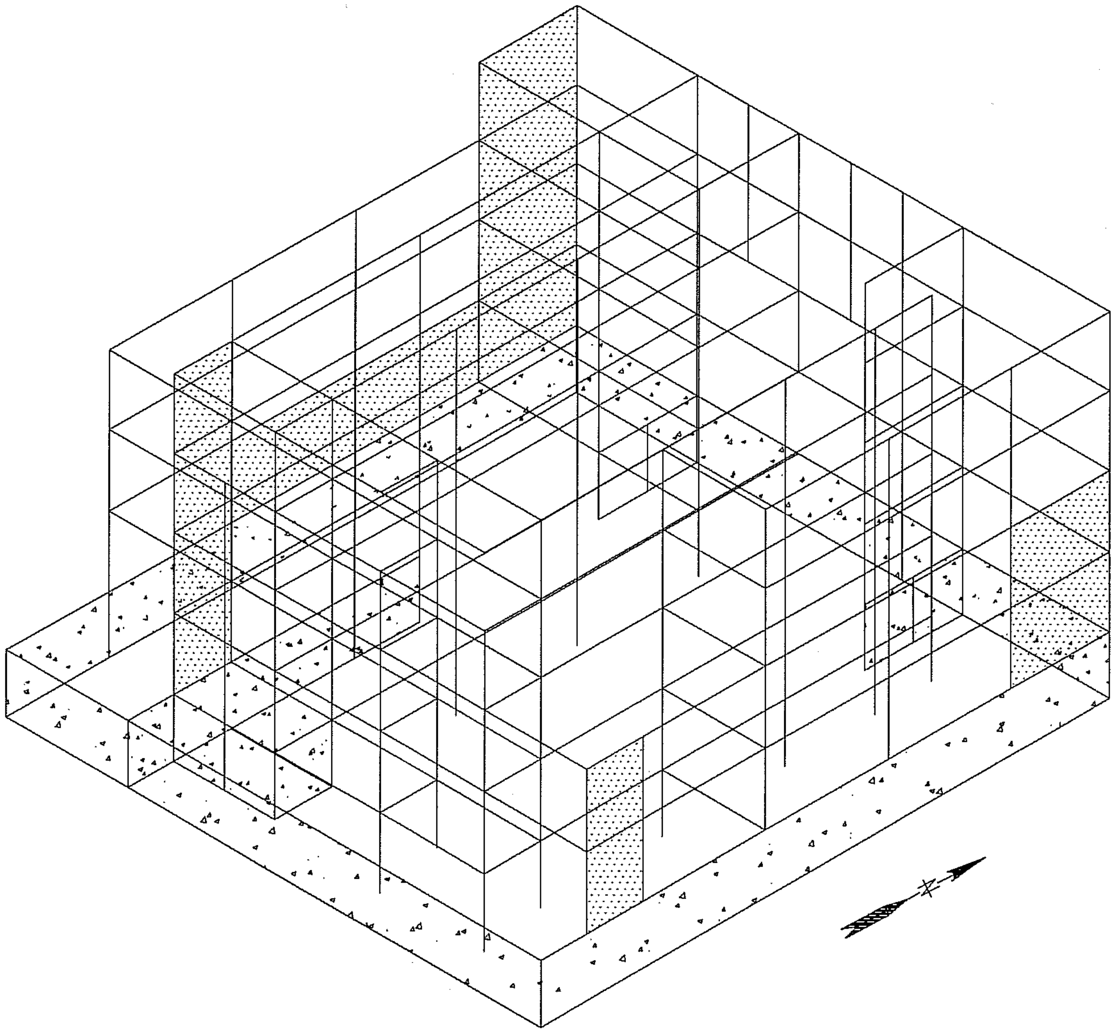

Figure 23 Peninsula Office Building - Three-Dimensional View of the Building Model Showing Locations of Brick Veneer Panels in the East and West Exterior Walls.

was 0.55 seconds and the base shear was 914 kips. Since cracking was observed in the masonry veneer at the east and west sides of the building, those elements evidently did act structurally during the earthquake, and resisted seismic forces.

The cracking shear stresses of the brick masonry were predicted according to the following formulas:

First, the cracking stress for zero axial load was computed:

$$v_{cro} = (4.2 - 1.75 \frac{M}{Vd}) \times \sqrt{f'm}$$

The effects of vertical compression were then included:

$$v_{cr} = \sqrt{v_{cro}^2 + (v_{cro} \, x \, \frac{f_a}{1.5})}$$

where

$\frac{M}{Vd}$ = Aspect Ratio of the Wall

f'_m = Masonry Compressive Strength

f_a = Average Compressive Stress

The above expression, used in predicting cracking shear stresses, was based on testing performed at the University of California at Berkeley, and found to be in reasonable agreement with results from subsequent research at the University of Colorado at Boulder [3].

The predicted cracking shear stresses were compared to the shear stresses obtained from the ETABS analysis. The results are summarized in Table 7.

Table 7 Predicted cracking shear stress versus computed shear stress (response spectrum).

Wall ID	Location	Predicted Cracking Shear Stress	Ratio of Computed to Predicted Cracking Shear Stress
W-39	West wall @ level one between grids 2 & 3	178 psi	0.98
W-51	West wall @ level one between grids 4 & 5	213	2.20
W-52	West all @ level one between grids 5 & 6	213	2.47
W-45/48	West wall @ level one between grids 6 & 7	139	1.86
W-53	East wall @ level one between grids 2 & 3	146	5.93
W-55/57	East wall @ level one between grids 6 & 7	139	6.21

The predicted buckling strength of the diagonal bracing members was compared to the axial forces obtained from the ETABS analysis. The results are summarized in Table 8.

4.6 Comparison of Predicted Response with Observed Damage

The observed cracking in the east and west masonry veneer walls is consistent with the predicted response when panels were included in the model, indicating that lateral seismic forces were transferred to the exterior masonry veneer panels during the earthquake.

To avoid cracking the masonry veneer, the structure must be detailed and constructed in a manner which permits the structural building frame to move independent of the masonry veneer. To determine the required clearance, an analysis was performed by inserting "dummy" diagonals at locations for which lateral movement of the frame was to be computed. A very small value for the elastic modulus and cross-sectional area was inserted for those diagonals so that overall building response would be unaffected. The resulting force in each diagonal was used to compute its elongation, from which the horizontal racking at each panel location was determined. The amount of horizontal racking varied from a minimum of 0.18 inches between Grids 6 & 7, to a maximum of 0.51 inches between Grids 2 & 3, as shown in Table 9.

Table 8 Predicted buckling strength of diagonal bracing members versus computed axial loads (response spectrum).

Frame ID	Computed Maximum Axial Brace Force	Predicted Buckling Strength of Brace	Ratio of Computed Brace Force to Predicted Capacity
VB2	100 kips	174 kips	0.57
VB3	99	174	0.57
VB4	113	174	0.65
VB5	148	174	0.85

4.7 Evaluation of 1988 UBC Design Provisions as Applied to this Building

Minimum seismic design requirements are outlined in Chapter 23, Section 2312 of the 1988 Uniform Building Code (UBC) [37]. The 1988 UBC allows design by either a dynamic lateral force procedure or an equivalent static lateral force procedure. It is common among practicing structural engineers in California to use the equivalent static force procedure for the analysis of buildings not having significant vertical or horizontal irregularities. The structural engineer of record confirmed that this building was originally designed using the equivalent static lateral force method.

Table 9 Predicted horizontal racking at panel locations neglecting stiffness contribution from exterior brick panels.

Panel Location	Dummy Diagonals	Horizontal Racking
West Elev - Grids 2-3		
- Level 4 to Roof	56 & 60	0.51 inches
- Level 3 to 4	57 & 61	0.50
- Level 2 to 3	58 & 62	0.41
- Level 1 to 2	59 & 63	0.35
West Elev - Grids 4-5		
- Level 2 to 3	64 & 66	0.42
- Level 1 to 2	65 & 67	0.36
West Elev - Grids 5-6		
- Level 2 to 3	68 & 70	0.48
- Level 1 to 2	69 & 71	0.37
West Elev - Grids 6-7		
- Level 4 to Roof	72 & 76	0.18
- Level 3 to 4	73 & 77	0.19
- Level 2 to 3	74 & 78	0.22
- Level 1 to 2	75 & 79	0.27
East Elev - Grids 2-3		
- Level 2 to 3	80 & 82	0.50
- Level 1 to 2	81 & 83	0.44
East Elev - Grids 6-7		
- Level 2 to 3	84 & 86	0.31
- Level 1 to 2	85 & 87	0.34

An equivalent static lateral analysis was performed using the ETABS model developed for the dynamic lateral analysis. The analysis was performed in accordance with the 1988 UBC. Although the building was originally designed in accordance with the 1979 UBC, the objective here was not to evaluate the original design, but to make observations regarding provisions of the 1988 UBC as applied to this building. The seismic forces were computed as follows:

$$\text{Seismic Base Shear, } V = \frac{ZIC}{R_w} W \qquad \text{(UBC formula 12-1)}$$

Fundamental Period, T	=	0.42 seconds
Seismic Zone Factor, Z	=	0.40
Site Coefficient, S	=	1.5
Importance Factor, I	=	1.0
Numerical Coefficient, C	=	2.75
Numerical Coefficient, R_w	=	8 (Concentrically Braced Frame)
Total Seismic Dead Load, W	=	5015 kips
Seismic Base Shear, V = 0.1375 x 5015	=	689 kips

Lateral forces were distributed to each level in accordance with UBC formula 12-6 as follows:

Roof Level	134 kips
Level Four	187 kips
Level Three	156 kips
Level Two	101 kips
Level One	111 kips

The results of the UBC analysis were as follows:

Maximum Building Drift at Roof Level	1.86 inches
Maximum Story Drift Ratio	0.0043 (4th Level)
Fundamental Building Period	0.42 seconds
Seismic Base Shear	689 kips

The 1988 UBC's maximum drift ratio limitation, 0.005, was met here. In addition to designing the main lateral force resisting system for the above lateral loads and drift limitations, the 1988 UBC requires that deformation compatibility of the structure be considered as discussed in UBC Section 2312 (h) 2. D [26, 37]:

> "exterior nonbearing, nonshear wall panels or ele ments which are attached to or enclose the exterior shall be designed to resist the forces per Formula (12-10) and shall accommodate movements of the structure resulting from lateral forces or temperature changes."

The UBC further states that

> "connections and panel joints shall allow for a relative movement between stories of not less than two times the story drift caused by wind, [3 x (R_w / 8)] times the calculated elastic story drift caused by design seismic forces, or 1/2 inch, whichever is greater."

Computed UBC story drifts and corresponding required clearances to avoid contact between the masonry veneer panels and the structural frame are presented in Table 10. The UBC required clearances were not provided for in this building. If these clearances had been provided, the veneer panels would probably not have cracked in the Loma Prieta event.

4.8 Comparison of Response Spectrum and 1988 UBC Responses

Results from the dynamic analysis and the 1988 UBC static analysis are compared in Table 11. The story shears, total base shear, story drift ratios and total building drift predicted using the dynamic analysis are greater than those based on the 1988 UBC. Generally, the results from an elastic dynamic analysis using representative strong ground motions produce significantly greater moments, shears and drifts than would be predicted using the UBC static lateral force method. The

Table 10 Required clearances between the structural frame and the masonry veneer panels, 1988 UBC.

Building Level	UBC Computed Elastic Story Drift	UBC Required Clearance [Story Drift x 3(Rw/8)]
Level Four	0.511 inches	1.532 inches
Level Three	0.519	1.557
Level Two	0.440	1.318
Level One	0.395	1.186
Basement Level	0.0022	0.0065

Table 11 Comparison of response spectrum versus 1988 UBC Results

Building Level	Response Spectrum Results		1988 UBC Results	
	Story Drift	Story Shear	Story Drift	Story Shear
Level Four	0.524 inches	149 kips	0.511 inches	134 kips
Level Three	0.526	330	0.519	321
Level Two	0.443	455	0.440	477
Level One	0.394	550	0.395	578
Basement Level	0.0094	723	0.0022	689

results are close here because the ground motions used for the dynamic analysis were relatively low, with peak ground accelerations in the order of 0.21g. Figure 24 compares the Palo Alto response spectrum and the implied 1988 UBC spectrum. For the building's fundamental period of 0.75 seconds, the two response spectra are fairly close. For moderate to strong ground motions, the actual elastic response spectrum would be much higher than that implied by the 1988 UBC.

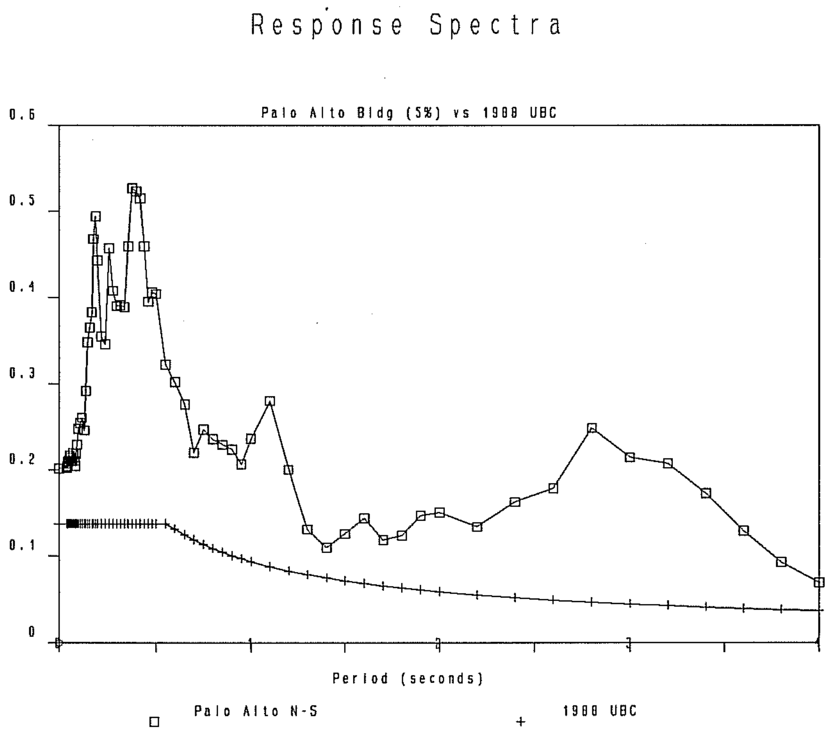

Figure 24 Peninsula Office Building - Response Spectrum for N-S Ground Motion versus Implied 1988 UBC Spectrum

4.9 Probable Response of Building in Stronger Earthquake

The Peninsula Office Building is a flexible structure. If it were subjected to a moderate to strong earthquake, it would undergo lateral drift greater than that experienced in the Loma Prieta event. During a strong earthquake, it is likely that lateral shear forces would be transferred to the non-structural exterior masonry veneer panels, causing significant cracking. The increased lateral forces would probably cause buckling of bracing members. In addition, high lateral drifts would most likely result in noticeable damage to the interior non-loadbearing walls.

4.10 Summary of Findings

The Peninsula Office Building, a steel framed structure with a masonry veneer exterior wall system, is representative of many masonry veneer buildings in California. The building performed very well in the earthquake with only minor cracking observed in the masonry veneer. With peak horizontal ground accelerations of 0.21g recorded in the general vicinity of this site, the Loma Prieta earthquake did not represent a significant test for this building.

Observations regarding the model of the building used in the analyses and the building's overall response and behavior during the earthquake are presented below:

1) Analytical Model of the Building

   a) The dynamic analysis reasonably modeled the response of the building, as predicted damage was consistent with observed damage.

   b) Soil-foundation flexibility did not have a significant effect on the results. The building's fundamental period increased from 0.72 to 0.75 seconds, and the effective damping factor for the structure-foundation system increased from 2% when soil-foundation flexibility was neglected, to 5% when this flexibility was included.

   c) The insertion of a dummy diagonal into the building model is a simple yet effective method of computing horizontal racking of the structure at any panel location.

2) Aspects of Behavior

a) The design intent of the UBC was satisfied. The building received only minor non-structural damage under a moderate earthquake.

b) The building is very flexible; its total drift was computed as 1.9 inches under response spectrum loading, neglecting the stiffness contribution from the exterior brick veneer panels. The corresponding maximum story drift ratio was 0.0047.

c) Including the exterior brick veneer panels in the building model reduces the calculated fundamental period of the structure from 0.75 to 0.55 seconds, and reduces the total building drift from 1.9 to 1.3 inches. The maximum story drift ratio is then 0.0035.

d) The computed cracking shear stress in the veneer panels under response spectrum loading exceeded the predicted cracking shear stress for all but one panel; this is consistent with the observed damage.

e) The 1988 UBC requires a clearance of [3 x (R_w / 8)] times the calculated elastic story drift or 1/2 inch, whichever is greater, between the structural frame and exterior nonbearing, nonshear wall panels. This results in a clearance requirement of approximately 1.50 inches in some locations. This is significant, and it may be difficult for the designer to properly detail for this amount of clearance.

f) In a stronger earthquake, bracing members would likely have buckled and veneer panels would have suffered more serious cracking.

g) The estimated horizontal racking of the structure in a stronger earthquake is approximately 1.50 inches, consistent with the clearance required by the UBC. However, detailing of a 1-1/2-inch gap may present problems for the designer.

h) Results from the response spectrum analysis are close to those obtained with the 1988 UBC equivalent static analysis. The peak horizontal ground accelerations at this site were on the order of 0.21g (fairly low). Review of Figure 24 shows that for the building's fundamental period of 0.75 seconds, the response spectrum for this earthquake and the implied 1988 spectrum are fairly close.

i) The use of thin bricks rather than standard units in the masonry veneer reduces the mass of the structure by approximately 4%, and consequently reduces story shears and overall building drift.

j) The use of masonry veneer to enclose steel framed structures is a viable type of construction if the veneer is isolated from the structure.

4.11 Recommendations

The following recommendations are offered regarding the design of similar masonry structures:

1) Generally, the stiffness of the veneer panels is neglected in lateral analysis. The results of this study point out the need to isolate the structural frame from the masonry veneer. Although the UBC allows a story drift ratio of 0.005, it also requires that a clearance of 1/2 inch [3 x (R_w / 8)] times the elastic drift or whichever is greater, be provided between the structural frame and exterior nonbearing, nonshear wall panels. As shown in Table 4-4, this can result in significant clearance requirements. It is important for the structural engineer and architect to detail the structure in a manner which provides this clearance.

2) The 1988 UBC limits the maximum story drift ratio to 0.005. The designer should give consideration to increasing the lateral stiffness of the structure beyond that required to meet this ratio if clearance requirements as discussed in Item No. 1 above become excessive and impractical to detail properly.

3) The use of a modern computer program which has good dynamic analysis and graphic capabilities, is recommended in performing dynamic analyses of multi-story buildings. Once the model is created,

the impact of differing ground motions or changes in the structure can be quickly evaluated. Also, the graphics capabilities allows the user to visually check the input data file and to visualize the modal response of the structure to the earthquake. It should be noted that programs like ETABS only permit evaluation of elastic response.

CHAPTER 5

BUILDING NO. 3 - 2 ALHAMBRA

5.1 General Description of the Building

The Two Alhambra building, located at 2 Alhambra street in the Marina District of San Francisco, is a four-story apartment complex (Figure 25). The ground floor serves as a garage and the upper three floors house the residential units. The total covered area is approximately (all floors) 21,500 square feet.

Figure 25 Facade of the building (2 Alhambra)

The building occupies the corner of Alhambra and Cervantes streets. It was erected in about 1905, wood being the major construction material. The vertical load carrying system in the upper three stories essentially consists of wooden beams and walls. On the ground floor, however, the system is an assemblage of a number of wooden columns, wooden beams and very few interior walls. The east, south and

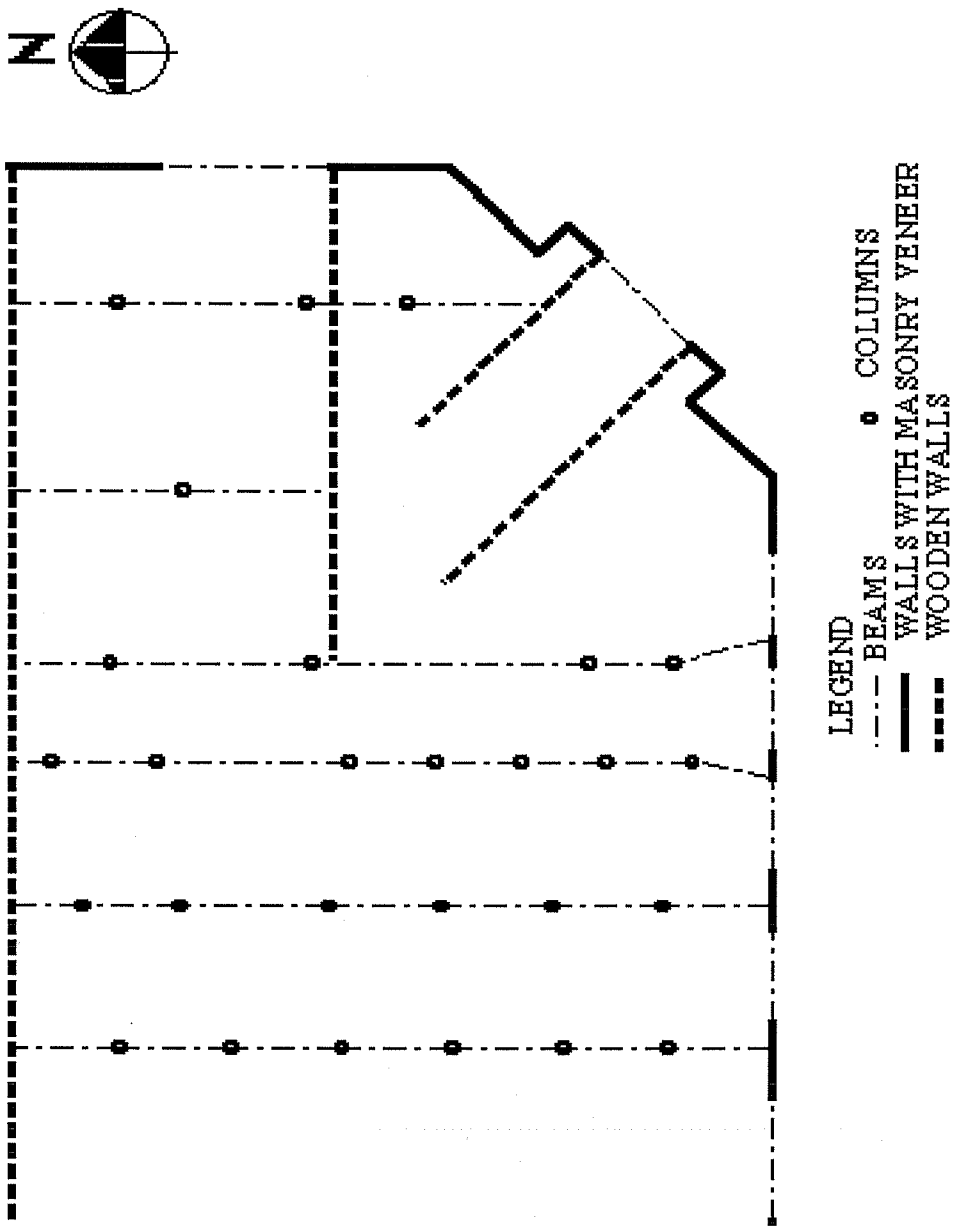

Figure 26 Original layout of Level 1 (2 Alhambra)

southeast facades (Figure 26) consist of stucco on wood lath on the upper three levels, and clay masonry veneer on the first level. On the other two sides of the building, all facades are of wooden siding.

The floors on all levels are wooden and are supported on wooden joists. The foundation system comprises spread footings under the columns and strip footings under the walls. Lateral loads are mainly resisted by plaster/stucco on wooden walls, and by the masonry veneer [10].

5.2 Description of Damage

On September 14, 1990 an on-site inspection of the building was carried out and details of damage due to the Loma Prieta earthquake were recorded.

The ground floor of the building had suffered extensive damage. On the ground floor, the east, south and southeast walls (Figure 27) had lost their masonry veneer, and the openings in these walls had become skewed.

The interior walls had also undergone damage; however, the details could not be recorded because of the retrofitting process in progress at that time. The north and west walls showed no significant sign of distress. The upper three levels of the building were essentially unscathed during the earthquake [10].

5.3 Selection of Ground Motion

The California Strong Motion Instrumentation Program (CSMIP) maintains a number of recording instruments in the vicinity of the building (Figure 29). The selection of the record was made on the basis of the following criteria:

a) The instrument station should be close to the building and

b) The instrument should either be located on a similar type of soil as the building or on a rock outcrop.

Using the above criteria, the SF Presidio record (CSMIP Station No. 58222) was finally selected for use [8]. As the instrument was located on a hard serpentine formation, the record was modified for soil effects as explained below.

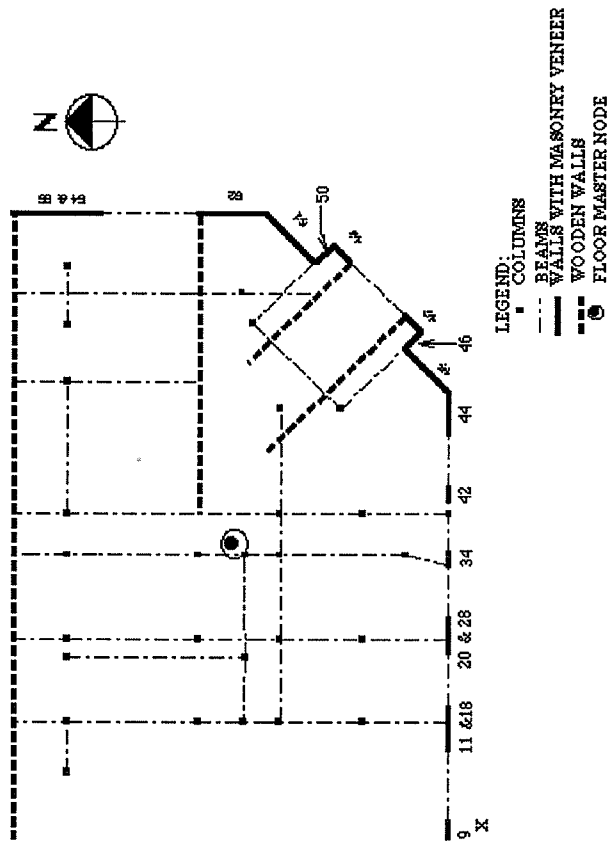

Figure 27 Modified layout of Level 1 (2 Alhambra)

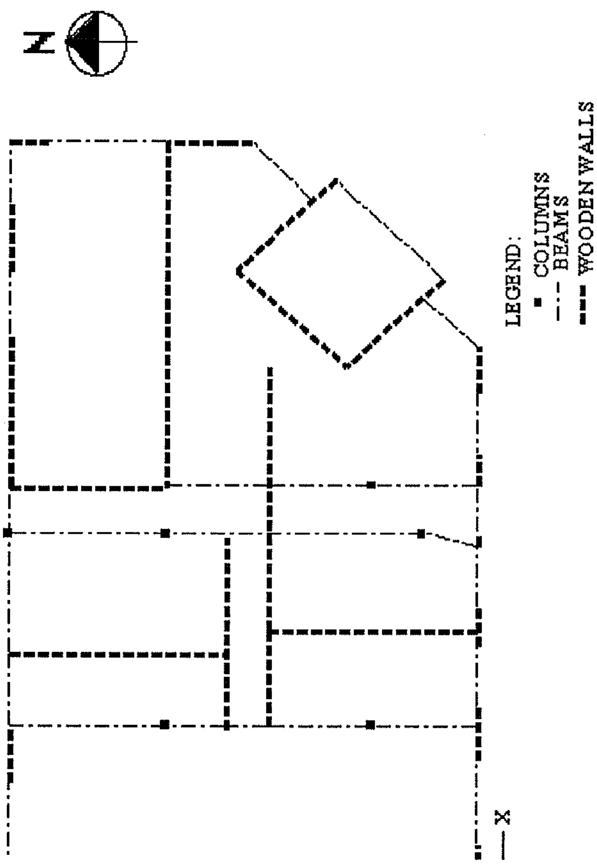

Figure 28 Layout of Levels 2, 3, & 4 (2 Alhambra)

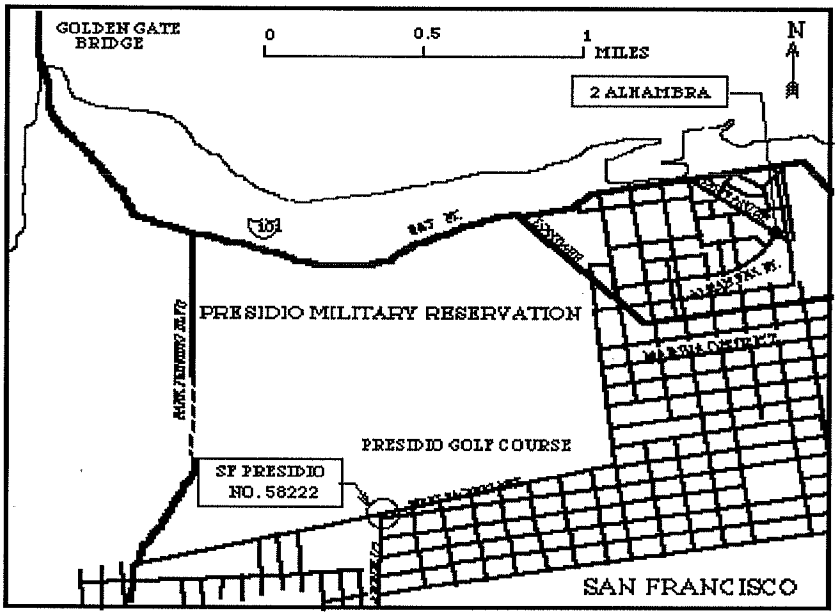

Figure 29 Location of the building and the instrument station

5.3.1 Modification of Ground Motion Record for Building Orientation: The two components of the record available from the above station were recorded along N00E and N90E, while the plan north of the building was at an angle of 45 degrees, measured counter-clockwise with respect to true north. Therefore, the two records were vectorially combined to get N45W and N45E records so that earthquake loads could be applied in directions parallel to the plan north-south and plan east-west of the building respectively.

5.3.2 Modification of Ground Motion Record for Soil Effects: The soils underlying the building are primarily various types of sands, overlying bedrock at an approximate depth of 225 ft. The type of soil made it imperative that the ground motion be modified to incorporate the effects of these soil deposits. The aforementioned objective was achieved by using program TIMOD.FOR, described in Appendix A.

During the inspection of the building no evidence of settlement or rigid body motion of the structure was found. This led to the conclusion that an assumption of linear elastic soil response would give a reasonable estimate of the ground motion experienced by the building.

Response spectra generated for both the modified and unmodified ground motion records are compared in Figures 30 and 31 for each component. In the dynamic analysis, the response spectrum for N45W was applied in the global Y direction and the response spectrum for N45E was applied in the global X direction (Figure 32).

5.4 Analytical Modelling of Building

5.4.1 Computer Program: Because of its simplicity, the structure allowed great latitude in selection of computer software for analysis. The microcomputer version SK-COMBAT [29] of the COMBAT [7] computer program was selected mainly because of its powerful foundation modelling capabilities, and because of its relative ease of use.

5.4.2 Available Building Data: The building was originally constructed without plans. However, as-built drawings prepared during the retrofitting of the structure were available [10]. These plans provided sufficient information about the layout of the building and the types of various structural members used in it. No information was available about foundations and material properties. A soils report was also prepared in the process of retrofitting, and provided information about the underlying soils [13].

5.4.3 Description of Analytical Model: The building was modelled as a three-dimensional wooden frame in such a way that the essential aspects of its lateral response were reproduced. Data not directly available were estimated as described here.

From visual inspection it was determined that all the beams and columns were essentially pin-ended, and hence did not contribute to the lateral stiffness of the structure. It was also concluded that the wooden walls, which consisted of horizontal wooden members nailed to wooden studs at each end, were in fact only reliable for carrying gravity loads; and whatever lateral stiffness they offered came from the plaster/stucco on the walls.

Using the above guidelines, the original layout of the first level was modified (Figure 33) in order to achieve the following objectives:

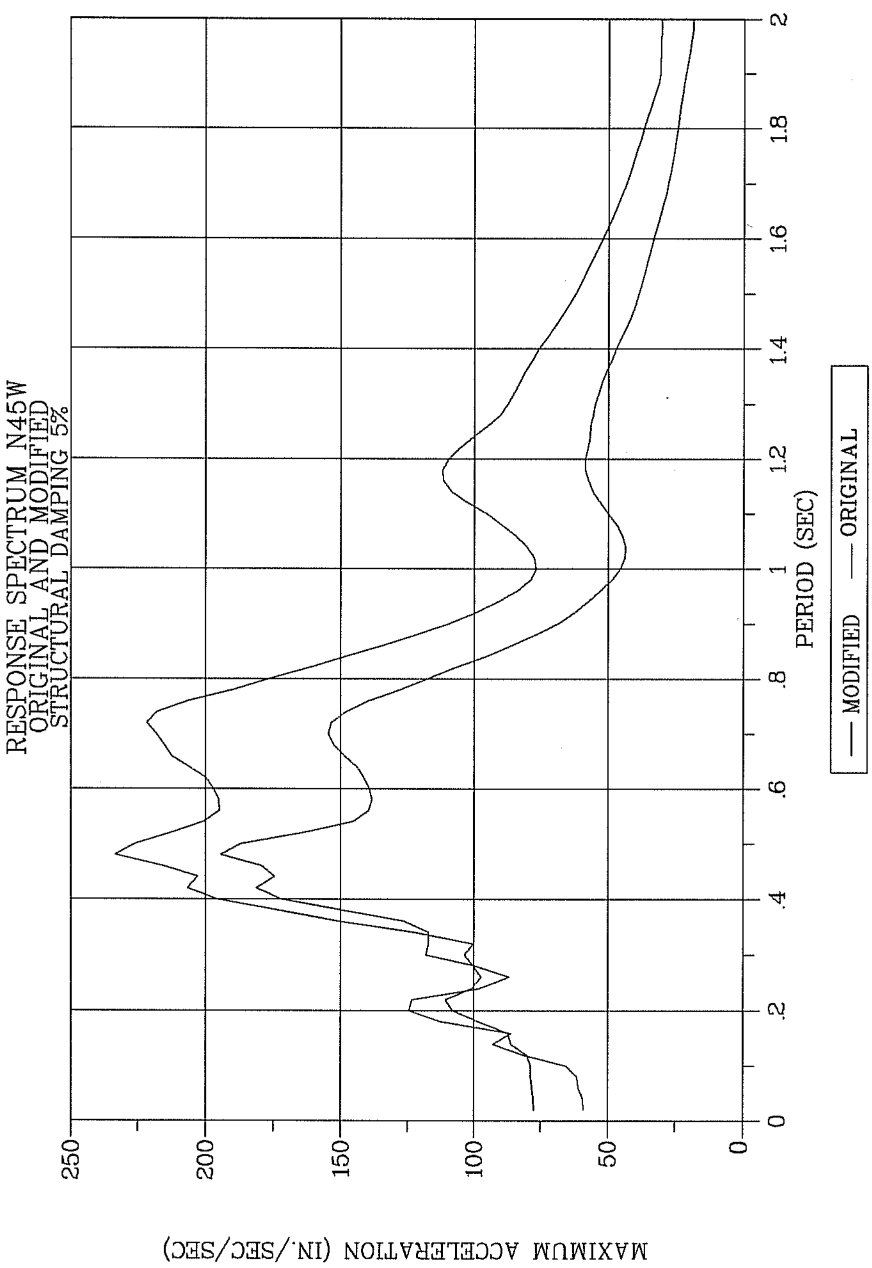

Figure 30 Effect of soils on response of spectrum N45W

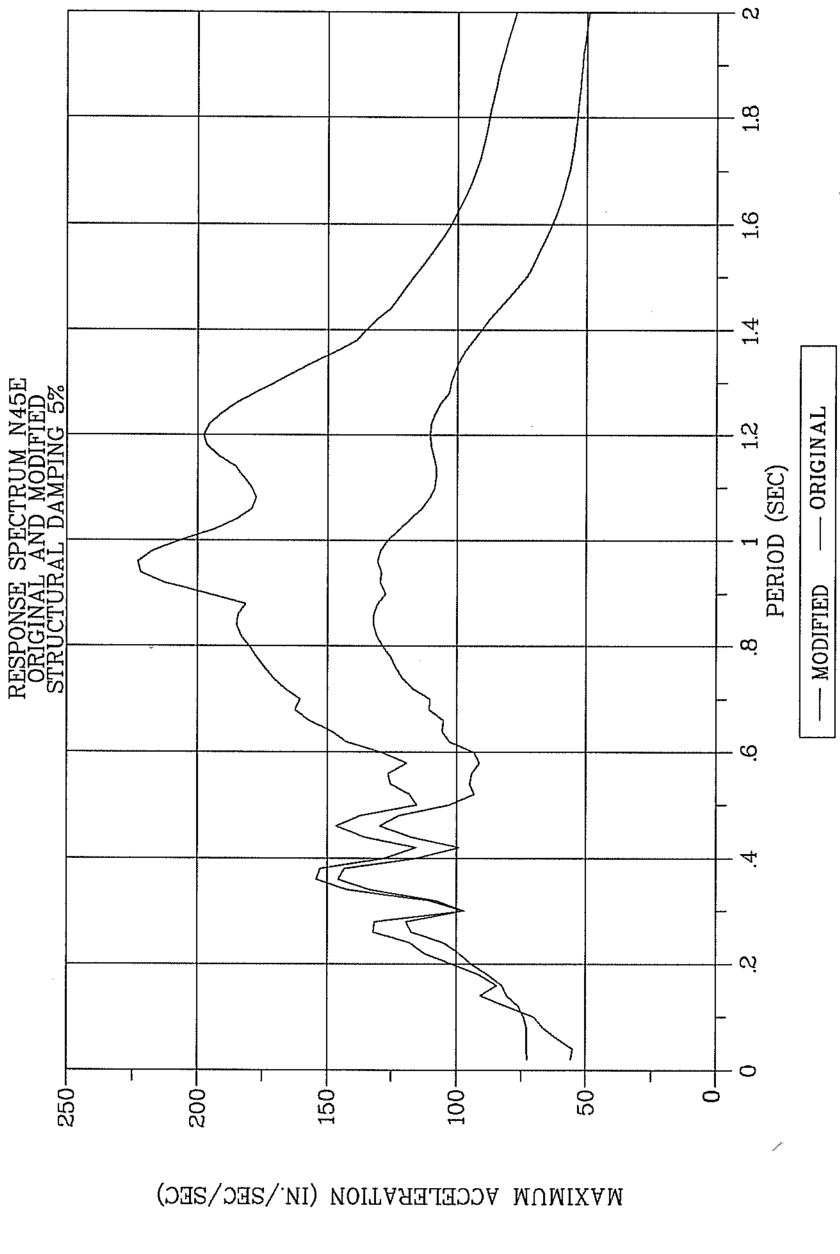

Figure 31 Effect of soils on response spectrum N45E

a) Reduction in the total number of nodes.

b) Alignment of nodes that fell approximately on the same column line, in order to reduce the number of bays and column lines in the model.

Most of these changes were carried out on the first level, because its layout was quite different from that of the upper three levels (Figures 26 and 28). Many columns on the first level were relocated while a small number of columns were eliminated altogether.

Because beams and columns did not contribute to the lateral stiffness of the structure, rough estimates of the material properties of the wood used in these members were sufficient [5]. All the plastered/stuccoed wooden walls were modelled as wall panels made of low strength concrete (f'_c = 2000 psi). Each such panel was assigned a thickness equal to that of the plaster/stucco on the wooden wall it replaced. Hence, the lateral stiffness contributed by the plastered/stuccoed wooden walls was preserved. On the first level, wooden walls with masonry veneer were ignored altogether; it was assumed that as a result of expansion of brick masonry and deterioration of wooden walls over the years, the masonry veneer was taking all the gravity loads. It was estimated that the masonry veneer had an equivalent strength of a 4-inch masonry wall made of 6000 psi bricks with Type N mortar; hence, the code-suggested stiffness values for that type of masonry were used [6].

The floor diaphragm consisted of a 0.25-inch finished floor over a 1-inch hardwood subfloor nailed to 2- x 12-inch wooden joists spaced at 16 inches. This construction made the floor diaphragm quite rigid in its own plane, and it was so modelled. As a result, the shearing and elastic moduli of the wood in the floor diaphragms became irrelevant.

Dimensions of spread and strip footings were assumed based on typical design practice. Since the columns were modelled as pin ended, only the vertical stiffness of the spread footing was important for the purpose of this analysis; it was calculated using the technique of Dobry and Gazetas [9]. This stiffness was incorporated into the model by attaching vertical springs to the bases of the columns. For the strip footings under the walls, horizontal stiffness perpendicular to the plane of the wall was ignored as the wall elements in the program have no out-of-plane stiffness. Horizontal stiffness parallel to the plane of the wall was incorporated by attaching springs of appropriate stiffness at each node at the base of the wall element, in the same direction. Vertical stiffness was treated in the same manner. Rotational stiffness of the strip footing could have been accounted for in the model by changing the stiffness of the vertical stiffness of the wall, which was deemed more important in the context of the response of the structure. Hence, the rotational stiffness of the footings was ignored.

Mass and gravity load calculations were carried out using the self-weight of the structure in addition to 70% of the code-suggested value for live loads [1, 16, 19, 37]. Gravity loads were assigned to bays on a tributary area basis, and bays were loaded with equivalent uniformly distributed loads.

5.5 Calculated Response

The structure was first analyzed with all the masonry veneer intact. Maximum flexural tensile stress and average shear stress were calculated for each panel from the result of the analysis. All those panels in which the stresses would exceed the assumed failure stress (see below) were removed from the model and replaced by pin-ended columns. The new model was analyzed again and it was determined that the rest of the panels had failed, too. At this point the model was again revised. All the remaining masonry panels were removed from the model, and a final run was made to investigate the behavior of the structure without the masonry veneer.

Any panel was considered as failed in flexural tension if its tensile stresses exceeded 100 psi. This limit was far greater than the code allowable value of 27 psi. The allowable shear stress for brick masonry was calculated by using the following formula [3].

$$v_{cro} = \left[4.2 - 1.75\left(\frac{M}{Vd}\right)\right]\sqrt{f_m'}$$

$$v_{cr} = \sqrt{v_{cro}^2 + \left(v_{cro}\frac{f_a}{1.5}\right)}$$

Where

v_{cro} = Cracking shear capacity neglecting axial load

v_2 = Cracking shear capacity including axial load

M = In-plane moment on the panel

V = Shear force acting on the panel

d = Effective depth of the panel

f_m' = Specified compressive strength of brick masonry

f_a = Compressive stress in the panel

Assuming all masonry to be intact, the calculated flexural and shear stresses are compared with the assumed failure values in Figures 32 and 33. The bay numbering scheme used in these figures is that of Figure 27. Shear stress exceeded the estimated capacity in only one bay. In most of the masonry panels, flexural tension exceeded the estimated capacity. This is very important because a failure in tension leads to reduced effective areas, which in turn can cause complete failure of the panel. These results strongly suggest that all or most of the ground-floor masonry veneer would have failed in the Loma Prieta event.

Lateral displacements in both directions under various combinations of gravity (static) loads and spectral loads are shown in Figures 34-37. When the masonry panels were intact, the structure behaved essentially as a stiff cantilever on a flexible base; the maximum roof drift was 0.23 inches in the X-direction, and 0.46 inches in the Y-direction. The drift at the first level was 24% of the roof drift in the X-direction and 34% of the roof drift in Y-direction. With all the masonry panels failed, the roof drift in the X-direction increased to 1.2 inches, and in the Y-direction to 1.4 inches. At this stage the drift at the first level was 65% of the roof drift in the X-direction, and 70% of the roof drift in the Y direction. This clearly indicated that the ground floor acted as a soft story after the masonry panels there failed.

5.6 Comparison of Calculated Response with Observed Damage

From the discussion in the previous section it is quite evident that the structure, as observed, would have lost all its masonry veneer, and that the ground floor acted as a soft story during the earthquake.

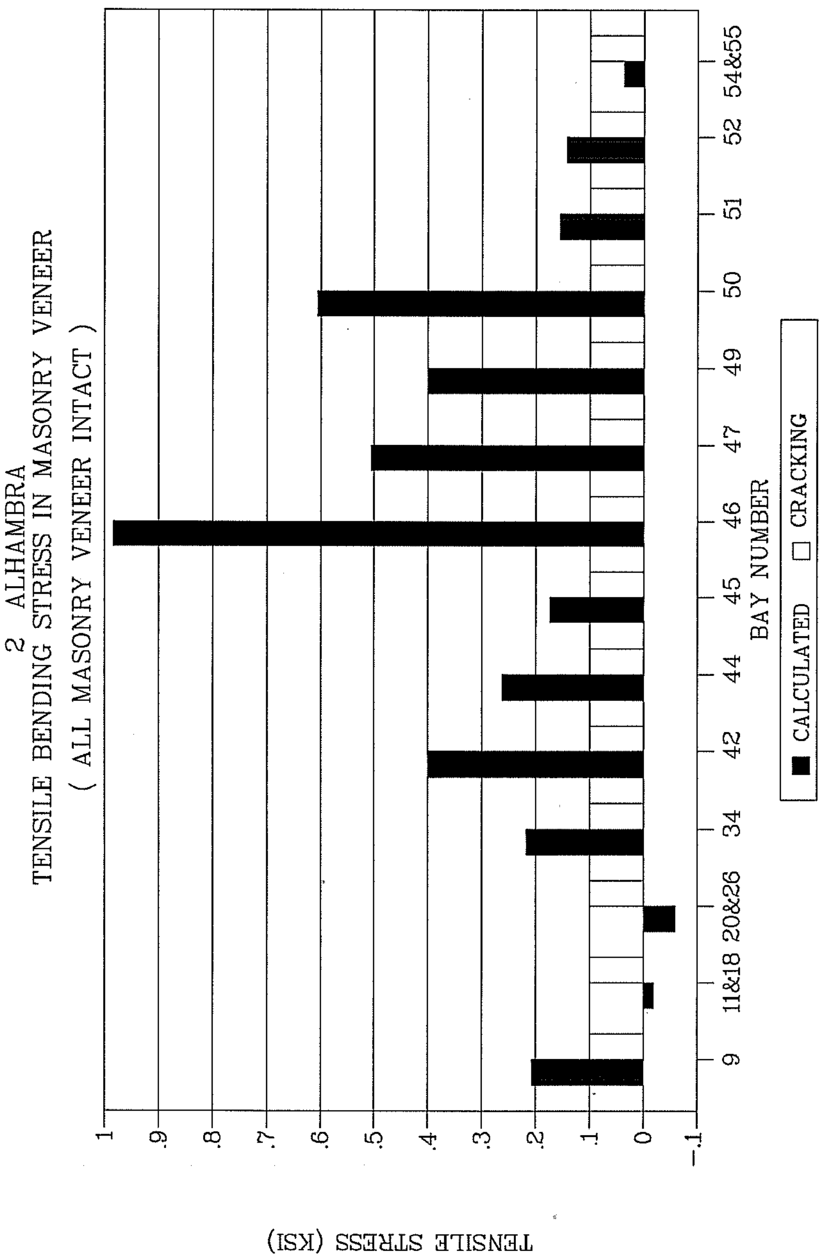

Figure 32 Calculated tensile bending stress versus cracking stress

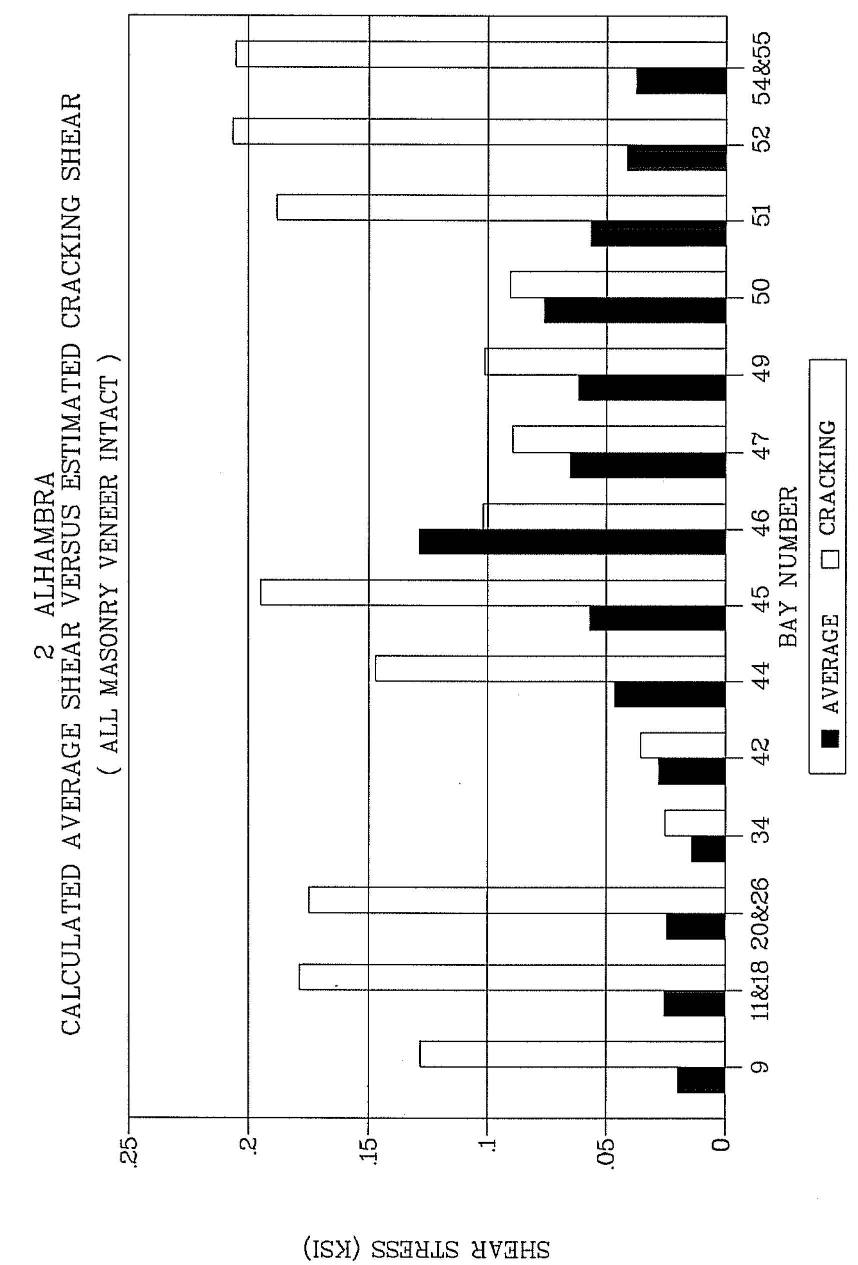

Figure 33 Calculated average shear versus estimated cracking shear

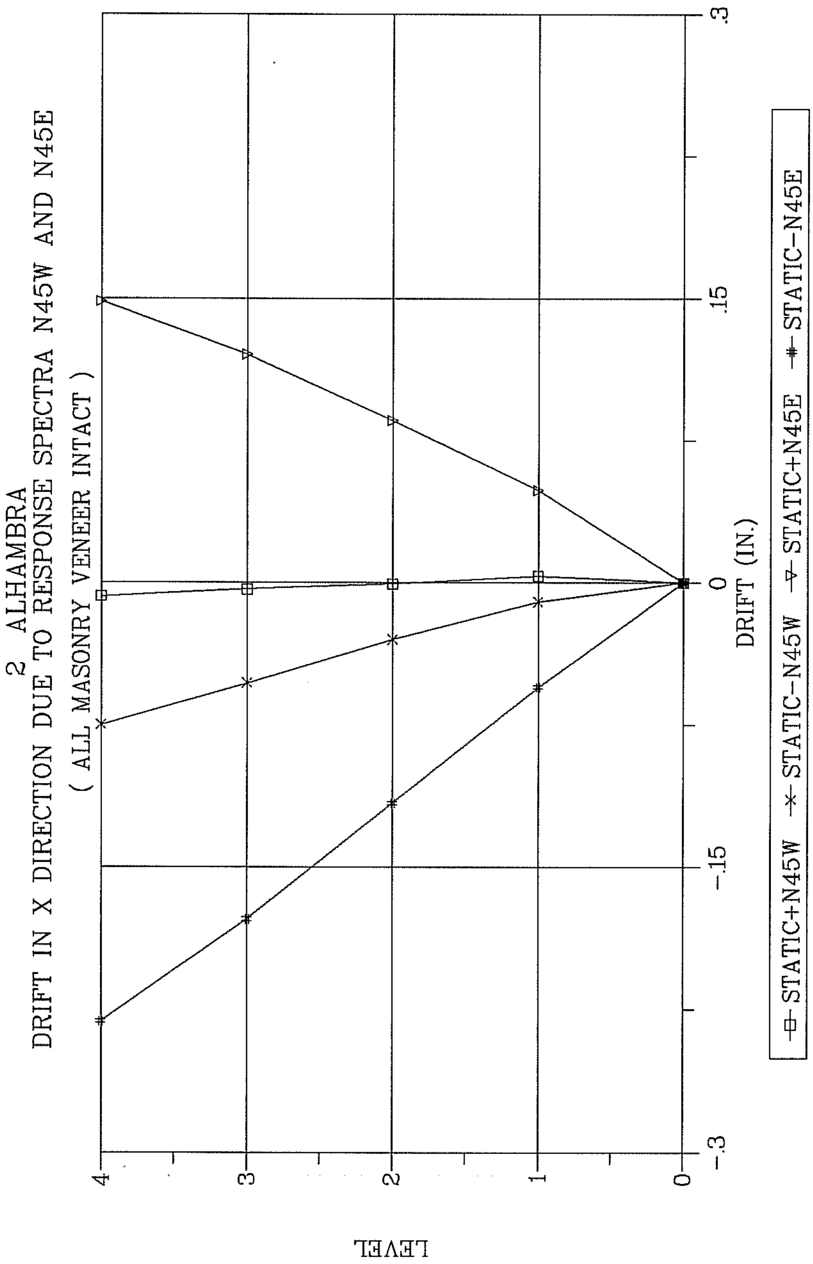

Figure 34 Drift in X direction (masonry veneer intact)

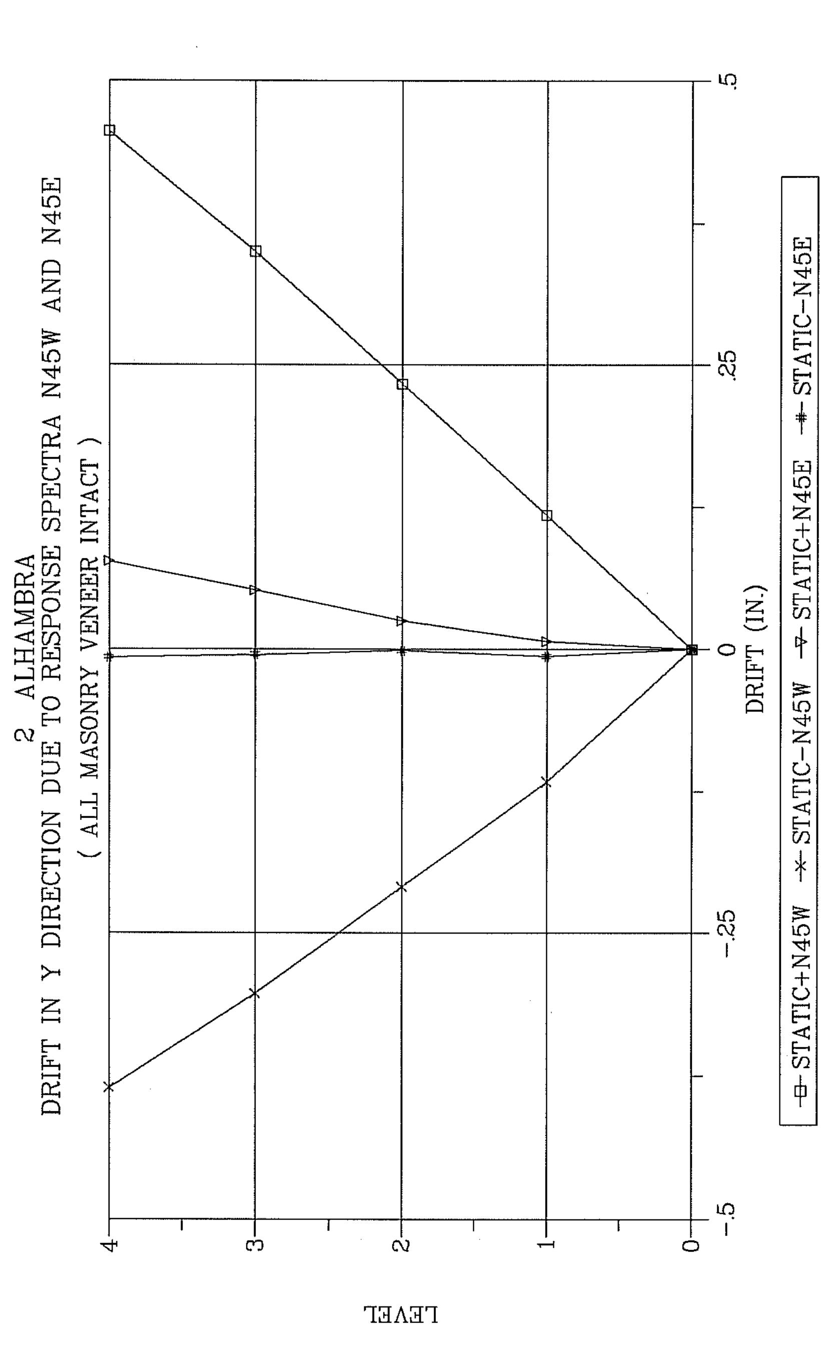

Figure 35 Drift in Y direction (masonry veneer intact)

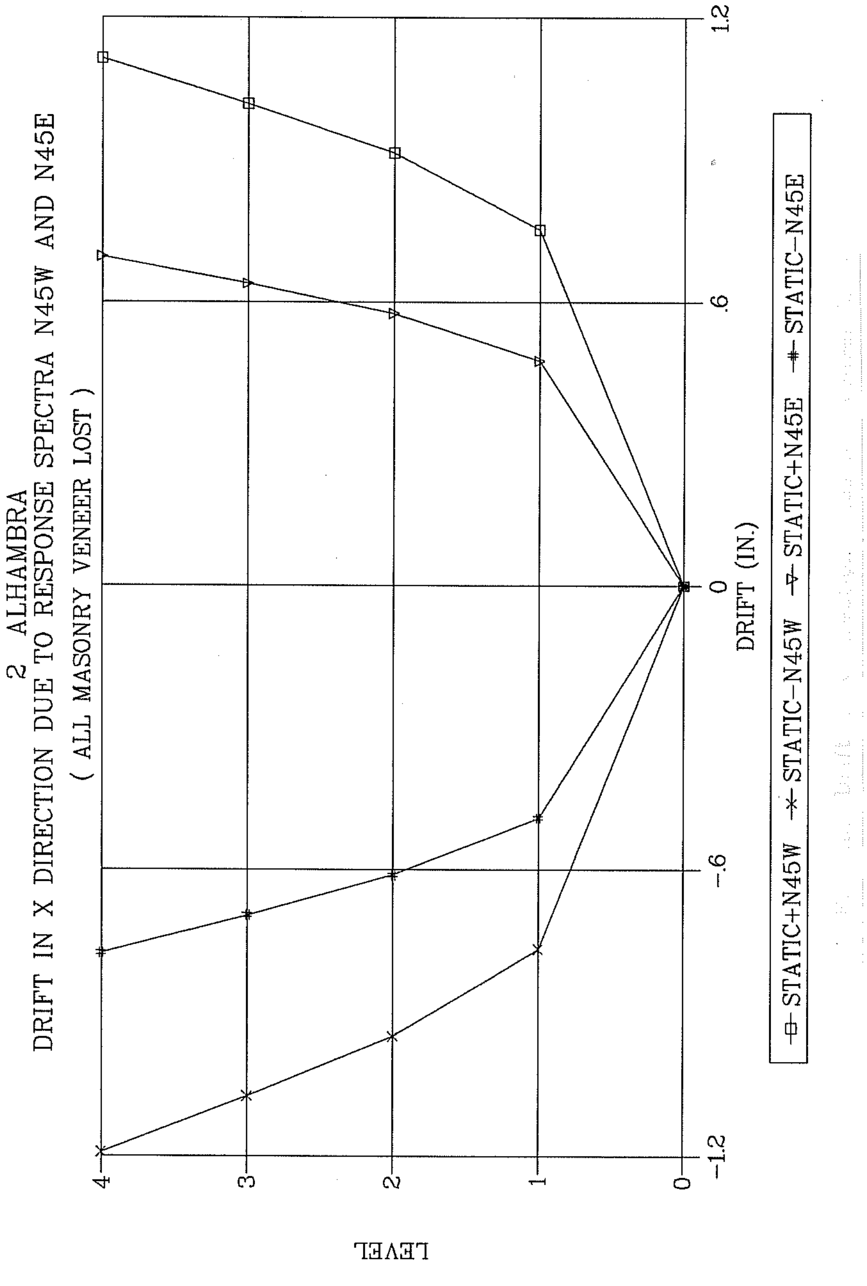

Figure 36 Drift in X direction (masonry veneer lost)

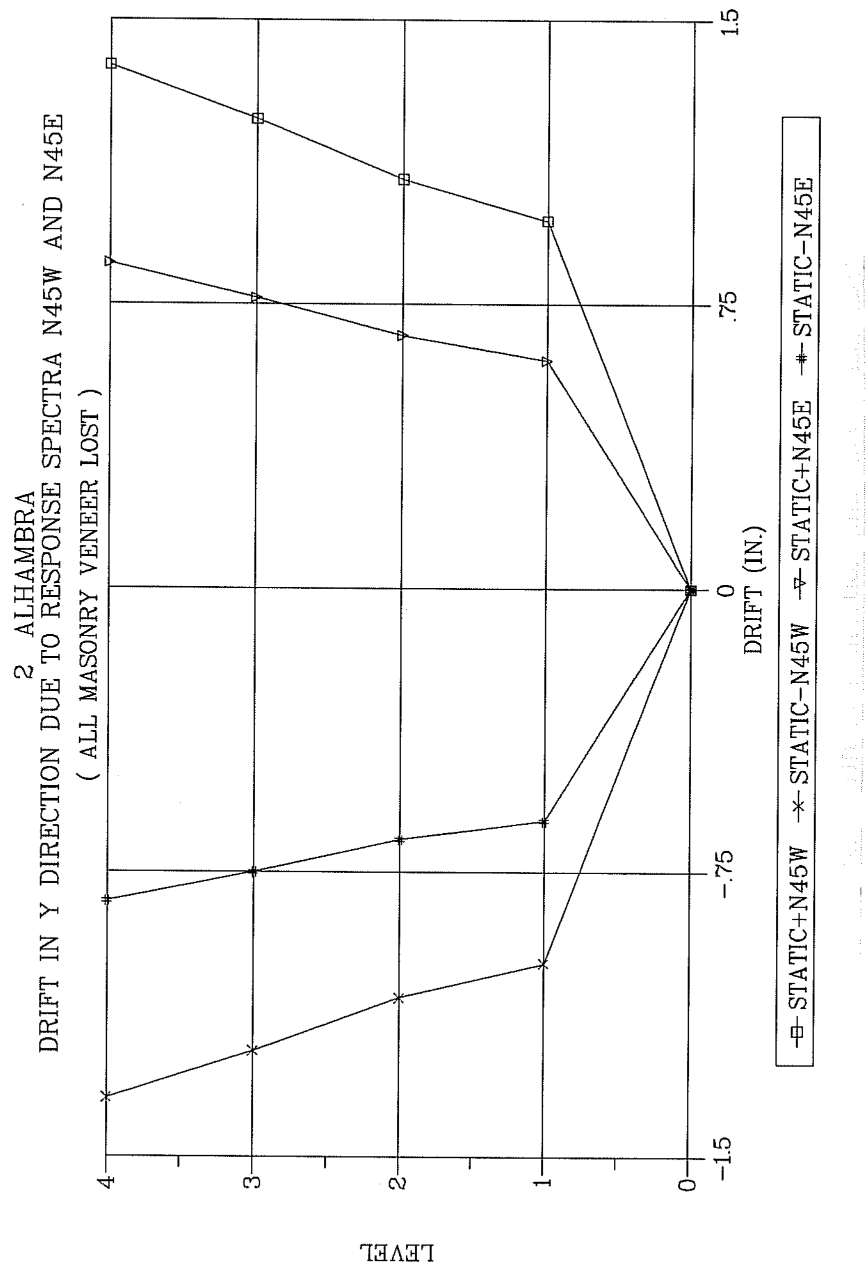

Figure 37 Drift in Y direction (masonry veneer lost)

Details of damage discussed earlier corroborate these inferences. The structure did lose all of its veneer during the Loma Prieta earthquake; racked openings in the exterior walls in the ground floor and virtually undamaged upper stories suggest that the ground floor did act as a soft story.

5.7 Comments Regarding the Behavior of the 2 Alhambra building

The observed damage to the 2 Alhambra building agreed well with that predicted analytically. The analytical approach used therefore seems reasonable.

Masonry veneer at the masonry street level was essentially destroyed by the earthquake. The remaining stiffness at the ground floor was due only to the wooden siding.

After the loss of its masonry veneer, the 2 Alhambra building probably experienced first-story lateral drifts of more than one inch, corresponding to a first-story lateral drift ratio of about 1%. Given the lack of lateral stiffness without the veneer, had the building been subjected to an event as strong as Loma Prieta but of longer duration, it might well have collapsed. The same comment would apply in the case of a stronger earthquake. Although the masonry veneer of the 2 Alhambra building was not originally conceived as acting structurally, it did reduce the first story drifts.

Because the 2 Alhambra building was not formally designed, it did not seem necessary to discuss the extent of its compliance with current building codes, nor to discuss ways in which its seismic performance might be improved. However, it is worthwhile to note that the seismic retrofit of the building, involving the installation of shear diaphragms at all levels, and X-braces at the ground floor, seems appropriate in view of the analysis results obtained here.

CHAPTER 6

BUILDING NO. 4 - HOTEL WOODROW

6.1 General Description of the Building

The Hotel Woodrow, located at the corner of MLK Way (Grove Street) and 14th Street in downtown Oakland, California, is a seven-story building with a basement, a mezzanine and a roof attic (Figures 38 and 43).

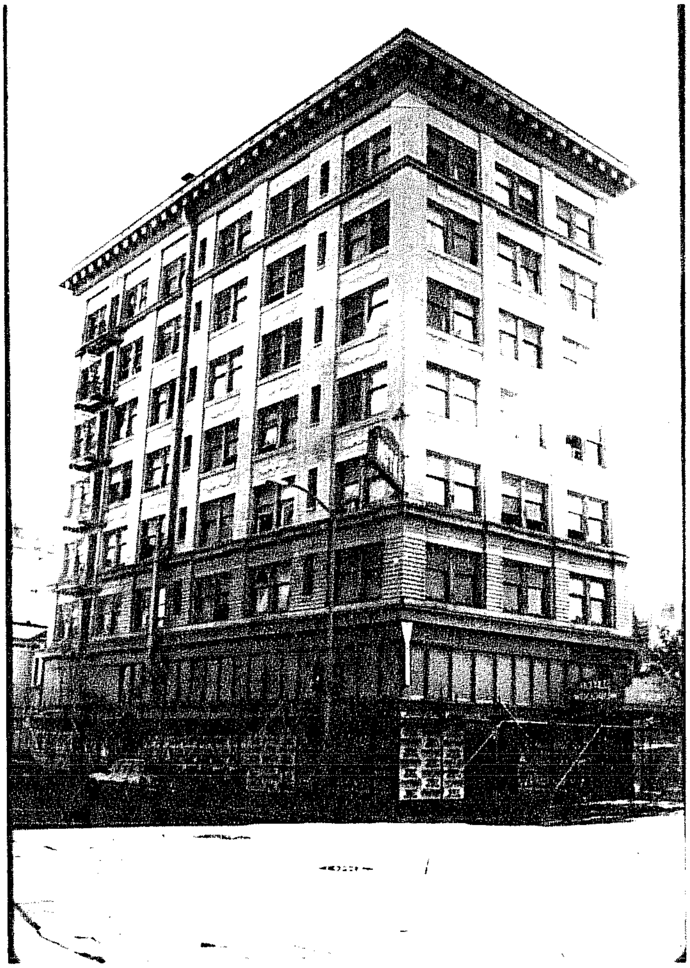

Figure 38 Street facade of the building (Hotel Woodrow).

The structure was built in 1912 of steel, wood, and brick masonry laid in lime mortar. The floor plans for various levels are shown in Figures 39-43. The vertical load carrying system consists of an assemblage of steel beams and columns in addition to walls. The structure depends entirely upon walls for lateral load resistance.

All exterior wall panels above ground level consist of 4-in. masonry veneer and 4-in. brick masonry infill, joined by bonded headers and a collar joint. The only exception is the north wall on the second level, which consists of an 8-in. masonry infill plus a 4-in. masonry veneer (Figure 40). The basement walls are 8-in. concrete panels which act as foundation walls (Figure 39). Except for some walls on Levels 2 and 3 which are made of 8-in. hollow terra cotta (Figures 40 and 41), all interior walls in the structure are 4-in. wooden stud walls.

The columns along the east side of the building and all the interior columns start at the basement. The other columns start at the second level. Most of the beam-column connections are riveted at the beam webs only, and were assumed to be simple connections. Those on the east and south of the building on the 3rd level have additional gussets attached to the beam flanges; these were assumed to be moment-resisting connections.

The story height for Levels 1, 2 and 4 through 9 is 123 inches. The story heights for Levels 3 and 10 are 93 and 84 in. respectively.

The floor diaphragm is made of a 0.75-in. finished floor nailed to 1-in. sub-floor, which is in turn nailed to 2-x 12-in. timber joists, which simply rest on steel beams. The floor diaphragm on the 2nd level covers only a part of the internal space, and is in two segments as shown in Figure 40.

The foundation system consists of spread footings under the columns, and strip footings under the walls [20, 24].

6.2 Description of Damage

On September 15, 1990 an on-site inspection of the building was carried out and details of damage due to the Loma Prieta earthquake were recorded. The street facade of the building had suffered no visible structural damage (Figure 38). However, most of the wider masonry panels in the back wall had undergone significant shear cracking. Most of the damage was concentrated on Levels 4 and 5. Some of the masonry veneer on the west wall had failed in compression and spalled off. Details of damage on the two back walls are shown in Figures 44 and 45.

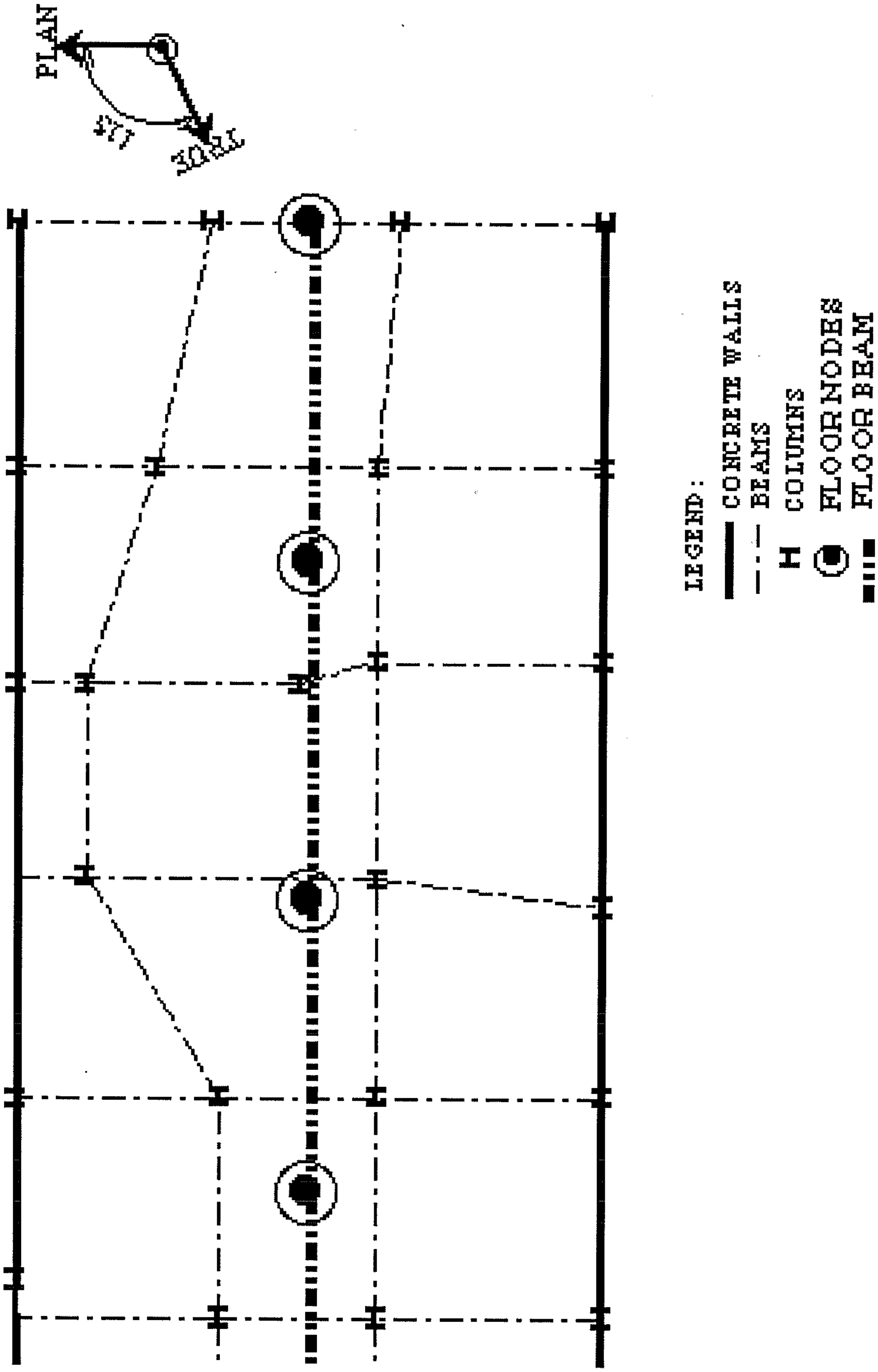

Figure 39 Layout of Level 1 (Hotel Woodrow)

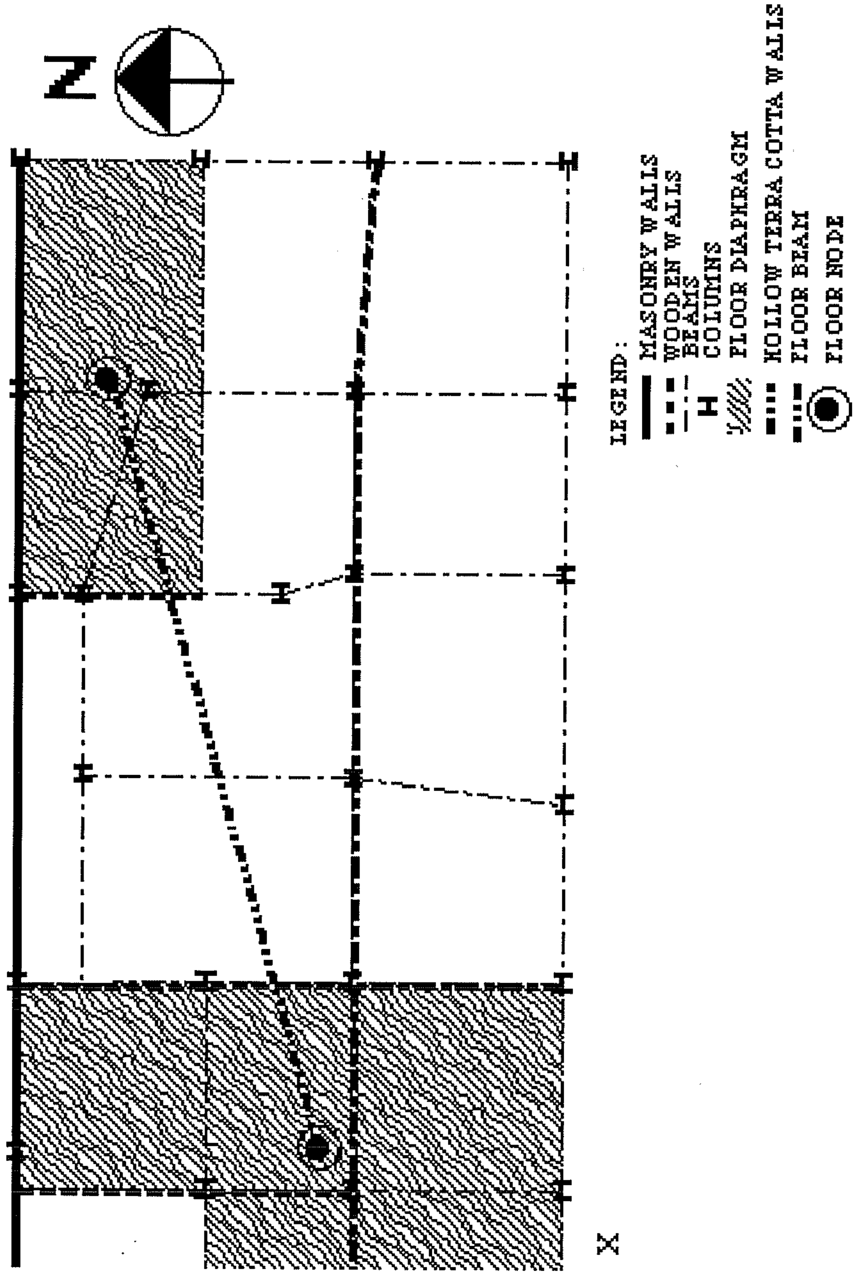

Figure 40 Layout of Level 2 (Hotel Woodrow)

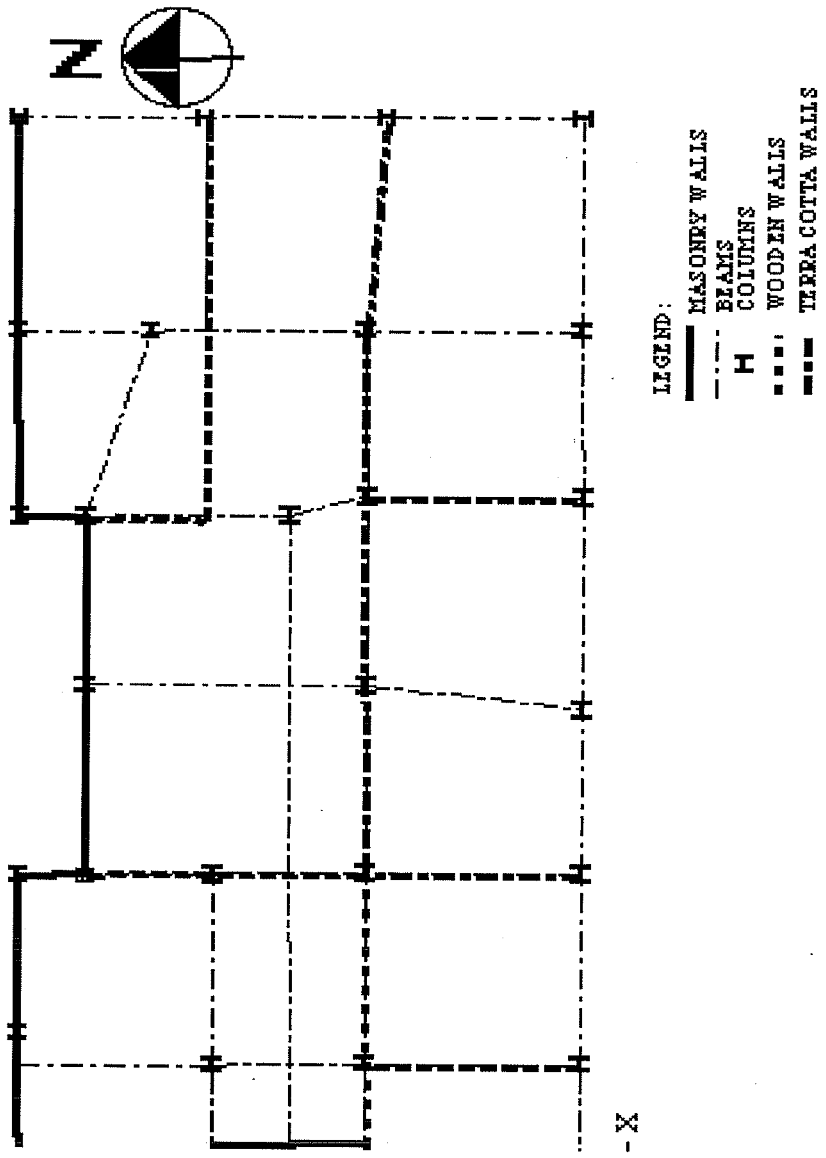

Figure 41 Layout of Level 3 (Hotel Woodrow)

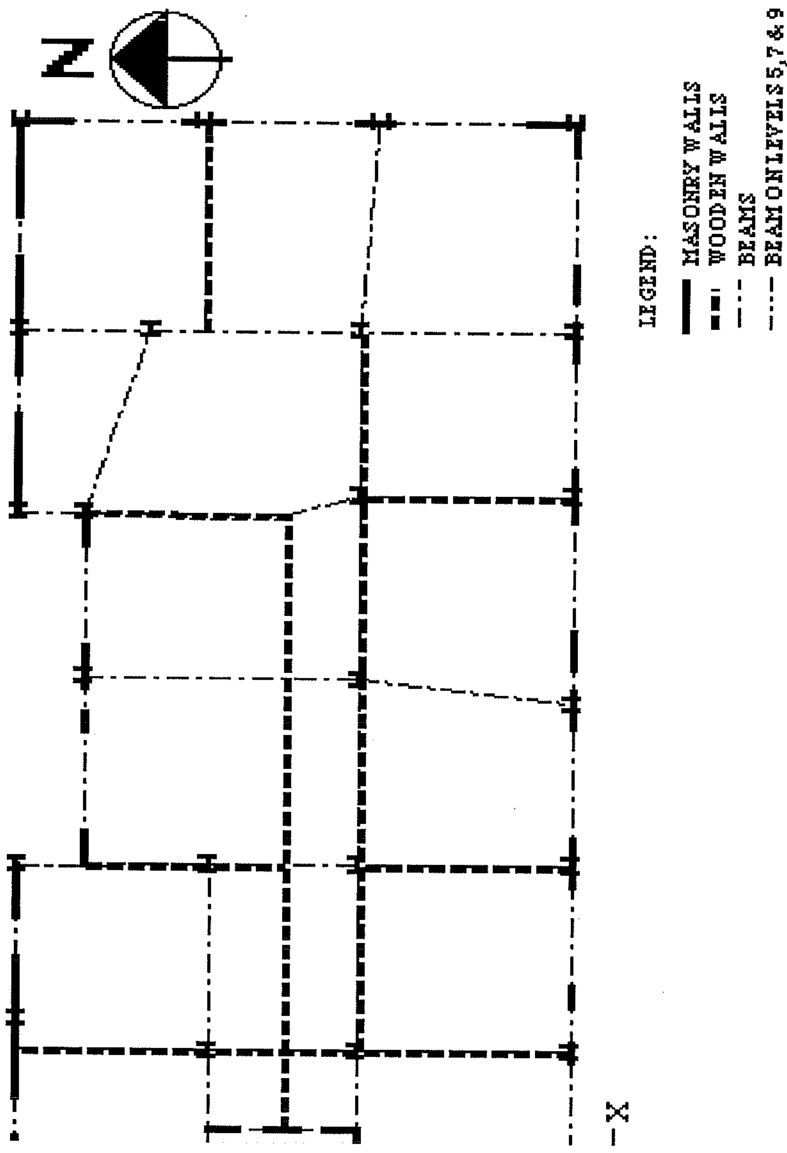

Figure 42 Layout of Levels 4 to 9 (Hotel Woodrow)

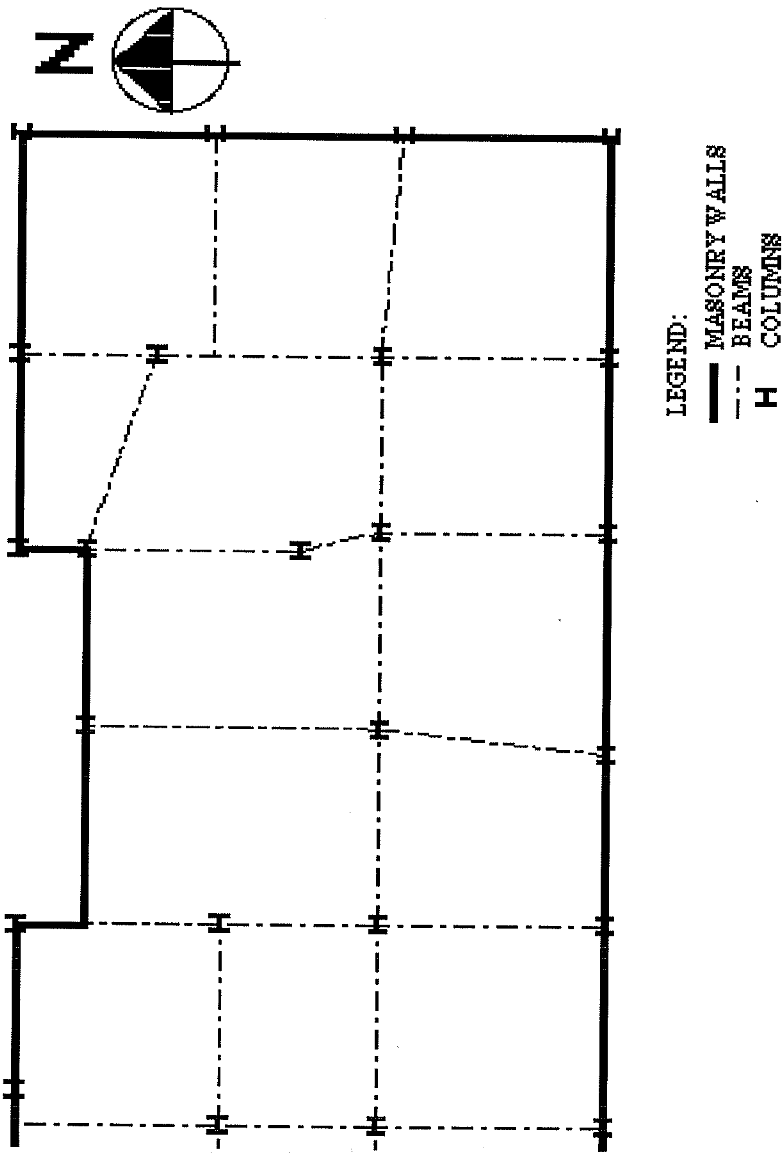

Figure 43 Layout of Level 10 (Hotel Woodrow)

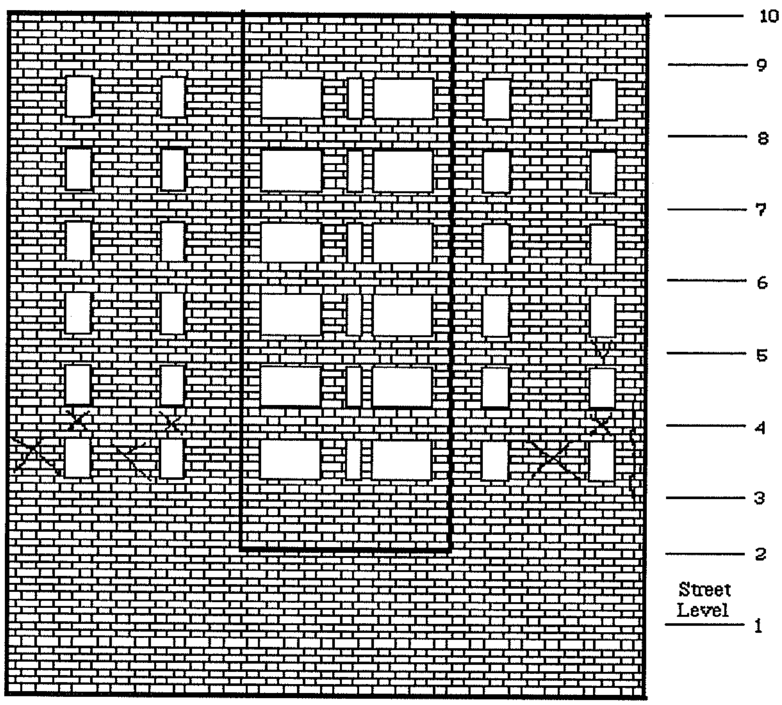

Figure 44 Damage on the north wall (Hotel Woodrow)

6.3 Selection of Ground Motion

The California Strong Motion Instrumentation Program (CSMIP) maintains some recording instruments in the general vicinity of the building. The selection of an appropriate record was made on the basis of following criteria:

a) The instrument station should be close to the building; and

b) The instrument should either be located on a similar type of soil as the building, or on a rock outcrop.

Using the above criteria, the Oakland record (CSMIP Station No. 58224) was selected for use (Figure 46) [8]. The instrument for this record was located on the ground floor of a 2-story office building, founded on soil similar to that underlying the Hotel Woodrow. It was decided that on account of the small size of that office building, effects of soil-structure interaction on this earthquake record could be ignored.

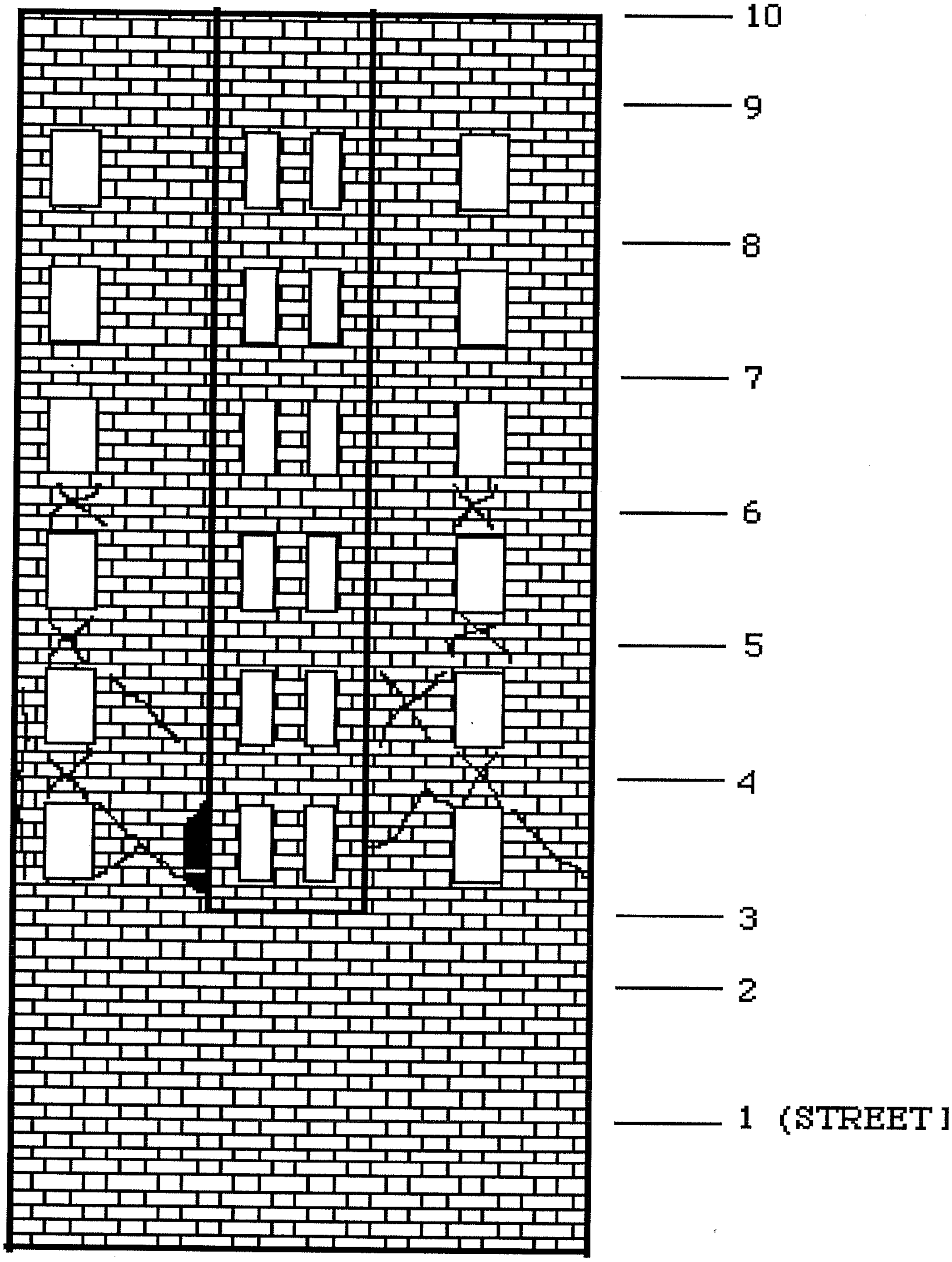

Figure 45 Damage on the west wall (Hotel Woodrow)

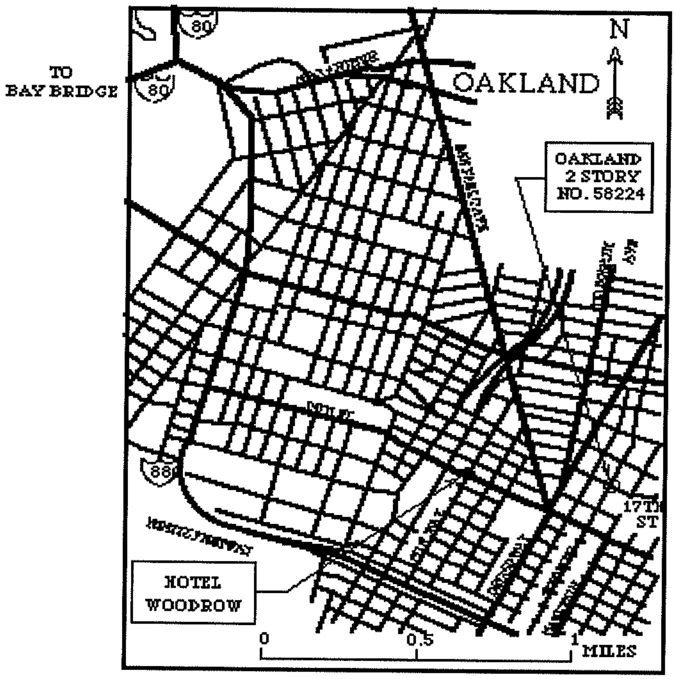

Figure 46 Location of the building and the instrument station

6.3.1 Modification of Ground Motion Record for Building

Orientation: The plan north of the building was rotated 115° clockwise from true north (Figure 39). The two components of the record available were along N70W and N20E. These two components were vectorially added to get N115W and N25W records, which were applied along the plan north-south and plan east-west of the building respectively. The acceleration spectra generated for these two components are shown in Figure 47.

6.4 Analytical Modelling of Building

6.4.1 Computer Program: As is evident from the general description of the building, its floor diaphragms were quite flexible in their own planes. Also, the wall panels had multiple perforations. To address these aspects of the building, the microcomputer version SK-COMBAT [29] of the COMBAT [7] computer program was selected. The program uses a finite element approach for the modelling of the

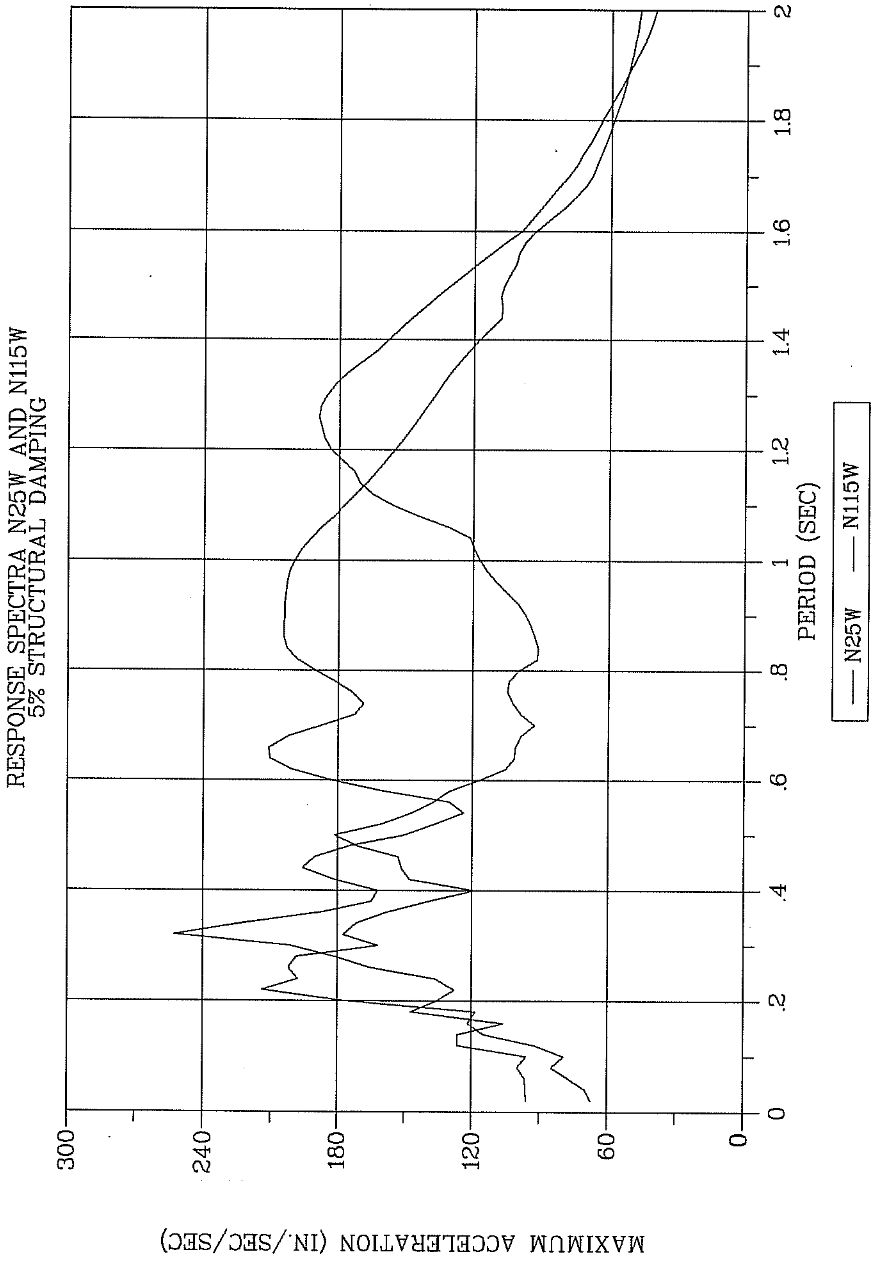

Figure 47 Response spectra N25W and N115W

wall panels. Each panel can further be subdivided into discrete sub-elements. An opening is represented by assigning a zero thickness to the corresponding sub-element. A floor diaphragm can be modelled as a series of floor nodes connected by flexible floor beams. The program also has a very powerful foundation modelling capability.

6.4.2 Available Building Data: The original structural drawings were available, and provided most of the information about the layout of the structure and about beams, columns and foundations. Additional data about walls and floor diaphragms were obtained from those involved in seismic evaluation and possible retrofit of the building [20, 24].

In the absence of a geotechnical report for the building site, information about soils in the general vicinity was obtained and used in the model [30]. No information was available about the structural properties of the principal construction materials used in the building; these were estimated as described here.

6.4.3 Description of Model: The building was modelled as a three-dimensional steel frame with masonry panels, in such a way that the essential aspects of its lateral response were reproduced.

From structural drawings it was determined that all beam-column connections, except for those at the third level on the east and south sides of the building, were not moment-resisting; hence, those beams were modelled as pin-ended. It was also determined that all the columns were pinned at their bases and continuous above that point; they were so modelled. Properties for most of the beam and column sections were available, with the exception of column section 8 H 32. Section properties of a W 8x31 were used in this case.

The wooden stud walls, which consisted of horizontal top and bottom plates nailed to wooden studs, were only effective for resisting gravity loads; whatever lateral stiffness they offered came only from the plaster on the walls. Each such wall was therefore modelled as a low-strength concrete panel (f'_c = 2000 psi) having a thickness equal to that of the plaster on the original wall it represented. The 8-in. hollow terra cotta walls were modelled as 1-in. brick masonry panels.

The exterior masonry panels, which consisted of masonry infill and veneer, were modelled as masonry walls with a thickness equal to the combined thickness of the veneer and the infill. This decision was based on the fact that the infill and the veneer were bonded together by headers and collar joint, and hence acted together under gravity and seismic loads. Openings were represented in the model by sub-elements of zero thickness. The foundation walls in the basement were modelled as 8-in. concrete panels.

In developing the analytical models, masonry infill elements were idealized both as equivalent diagonal struts [33] and as substructured finite elements. No significant differences were noted in overall response calculated using those modelling approaches. Because it permitted the application of multiple cracking criteria, the substructured finite element approach was preferred.

Soil-foundation flexibility was considered by the inclusion of discrete soil springs. Under the columns, only the vertical and horizontal translational stiffnesses of the spread footings were important for this analysis. These stiffnesses were calculated using the techniques of Dobry and Gazetas [9]. For the strip footings under the walls, horizontal stiffness perpendicular to the plane of the wall was ignored, as the wall elements in the program have no out-of-plane stiffness. Horizontal stiffness parallel to the plane of each wall was incorporated by attaching horizontal springs of appropriate stiffness at each node at the base of the wall element, parallel to the wall. Vertical stiffness was treated in the same manner. Rotational stiffness of the strip footing could have been accounted for in the model by changing the stiffnesses of the vertical springs at each end. However, this would have altered the vertical stiffness of the wall, which was deemed more important in the context of the response of the structure. Hence, the rotational stiffness of the footings was ignored.

Since the floor diaphragms had negligible in-plane flexural stiffness, the floor beams in the model were assigned negligible moments of inertia. Their shear areas corresponded to sections having a thickness equal to the combined thickness of the finished floor and sub-floor, and a width equal to the width of the diaphragm, at each location. This was believed to represent an upper bound to the shear stiffness of the floor diaphragm. At each level, the floor diaphragm was represented by four floor beams, spanning between concentrated masses (floor nodes) as shown in Figure 39. This was done to best represent the mass distribution at each floor level. As the floor diaphragm on the second level covered only a part of the structure, and was in two relatively small portions, it was essentially represented by two floor nodes, one in each segment, connected by a floor beam of negligible cross-sectional area, shear area and moment of inertia (Figure 40). Thus, the diaphragm was represented as two rigid segments connected by a very flexible floor beam.

The shearing and elastic moduli for steel were taken as 11200 ksi and 29000 ksi respectively. However, the material properties of the wood used in floor diaphragms and of the brick masonry, laid in lime mortar, could only be crudely estimated. The shear and elastic moduli for wood were assumed to be 1500 ksi and 625 ksi respectively; and for masonry, these quantities were assumed to be 1600 ksi and 640 ksi respectively. The concrete used in foundation walls in the basement was assumed to have a compressive strength of 3000 psi.

Mass and gravity load calculations were carried out using the self-weight of the structure plus 70% of the UBC value for live loads [37]. This value of live load represented an average value of live load present in the occupied building at the time of the earthquake [16]. Gravity loads were assigned to bays on a tributary area basis, and the bays were loaded with equivalent uniformly distributed loads.

6.5 Calculated Response

6.5.1 General Approach: From the observed damage to the structure, it is quite obvious that the masonry cracked and acted nonlinearly. However, the program available assumes linear isotropic material behavior. This implies that a highly stressed panel, which would actually have cracked (resulting in stress redistribution), remains effective in the analysis; hence, stresses in other panels are misrepresented to some extent. This limitation was addressed by performing a series of sequential linear analyses, removing panels as they cracked or crushed

Under earthquake loads, a panel may develop cracks as a result of following actions:

a) Shear

b) Axial tension or compression

c) Flexural tension or compression

If the average shear stresses in a panel exceed the cracking shear, diagonal cracks will develop; an unreinforced panel will lose its ability to resist shear forces, and will not contribute further to the shear stiffness of the structure. Also, it will not be able to withstand significant tension, whether direct or flexural.

If a panel cracks in direct tension; it cannot resist shear combined with net tension. However, it is still effective in compression; in the presence of net compression, it can still resist some shear.

If a panel crushes under axial compression, it cannot resist any further force.

Once a panel cracks in its extreme fibers due to flexural tension, its ability to resist shear forces depends on its aspect ratio and on the magnitude of the flexural stresses that caused the cracking. If the element is narrow, the reduced effective area still available for shear resistance is quite small. Hence, the panel loses most of its shear stiffness. Similarly, if the flexural stresses are very high, then the extent of cracking is much greater and the panel has a significantly reduced ability to resist shear forces. The same train of reasoning can be followed for cracks due to flexural compression.

To address the issue of cracking and stress redistribution, an event-by-event analysis approach was devised which would give conservative (high) estimates of drifts. The general steps of this procedure are as follows:

i) The structure is first analyzed with all of its masonry panels intact. Stresses are calculated in a number of panels.

ii) If many panels are found to be under high axial tension, this would imply that those panels in reality had cracked, and could not resist the shear associated with the axial tension. Those panels would still be effective in compression. It could therefore be concluded that shear stresses in other panels differ in reality from those calculated by the analysis. However, to get a better estimate of shear stresses in those panels, the cracked panels are removed from the model by assigning a zero thickness to the corresponding sub-element in the model. The model is reanalyzed.

iii) This process is continued until few of the remaining elements crack in axial tension. This implies that the stress states in the remaining panels are much closer to reality, and hence more reliable.

iv) At this stage, all the panels cracking in shear, along with those failing in axial tension, are removed from the model using the same technique as explained above. Flexural tensile stresses are also calculated at the extreme fibers. However, panels developing cracks due to flexural stresses are not removed from the model unless axial tension is also very high. The modified model is again analyzed.

v) This process is continued until no panel elements have significant calculated tensions.

As has already been mentioned, material strengths were a major uncertainty in this analysis. To overcome this problem, computed stress levels for different panels were compared to the actual damage, giving estimates of stresses at which a panel could be assumed to have cracked.

6.5.2 Procedure for Calculation of Stresses: The technique of response spectrum analysis was used to analyze the building. This method has disadvantages. First, the earthquake loads are applied in the form of two response spectra, applied orthogonally, one at a time; second, modal response combinations lose sign information; and third, no information is available about the time when maximum forces occur. Therefore, the forces resulting from the two spectra cannot be simply added.

To overcome this handicap, an approach was devised in the light of NEHRP recommendations [23]. The maximum spectral shear resulting from either of the two spectra was combined with the shear resulting from gravity loads; to this quantity was added 30% of the spectral shear resulting from the other response spectrum, to get an estimate of the maximum shear that the panel probably faced.

Axial spectral force associated with the maximum spectral shear was treated as a tensile load, and was combined with the compressive force resulting from the gravity loads. The resulting quantity was combined with 30% of the spectral axial tension resulting from the other spectrum. A similar approach was used to calculate moments in the panels.

Once these forces had been determined, average shear stress and axial tensile stress were calculated for each panel.

The cracking shear stress for masonry was calculated by using the following formulation [3, 19]:

$$V_{cro} = \left[4.2 - 1.75 \left(\frac{M}{Vd} \right) \right] \sqrt{f'_m}$$

$$V_{cr} = \sqrt{V_{cro}^2 + \left(V_{cro} \frac{f_a}{1.5} \right)}$$

Where

v_{cro} = Cracking shear capacity neglecting axial load effects

v_{cr} = Cracking shear capacity including axial load effects

M = In-plane moment acting on the panel

V = Shear force on the panel

d = Effective depth of panel

f_m' = Specified compressive strength of brick masonry

f_a = Compressive stress in the panel

For each panel, the ratio of the average shear stress to cracking shear stress was calculated. A ratio of unity or more would in principle imply shear cracking of the panel.

6.5.3 Event-by-Event Analysis of the Structure: This procedure was intended to thoroughly investigate the stress states in the panels on Level 4, while checking damage on other levels as well. The process was started with the analysis of the structure with all of its masonry panels intact (Case A); the model was then modified as discussed in Section 6.5.1. The model was modified two more times before all the damage was confirmed. Thus, four cases in all were considered. These cases, denoted successively as Cases A through D, are explained in the rest of this section.

For the purpose of stress calculation, wall panels were divided into two categories: vertical wall elements, and spandrel wall elements. In some cases, the vertical wall elements occupied two adjacent bays. However, all the spandrel wall elements consisted of the wall above the opening in the lower story and the wall below the opening in the upper story, as shown in Figure 48. In the case of vertical wall elements, forces on a horizontal plane were considered for the calculation of stresses. In the case of the spandrel wall elements, forces acting on a vertical plane were considered (Figure 48).

The numbering schemes used for both types of panels are shown in Figures 49 and 50. In Figures 51-59, an asterisk next to a panel number shows that the panel was actually damaged in the earthquake.

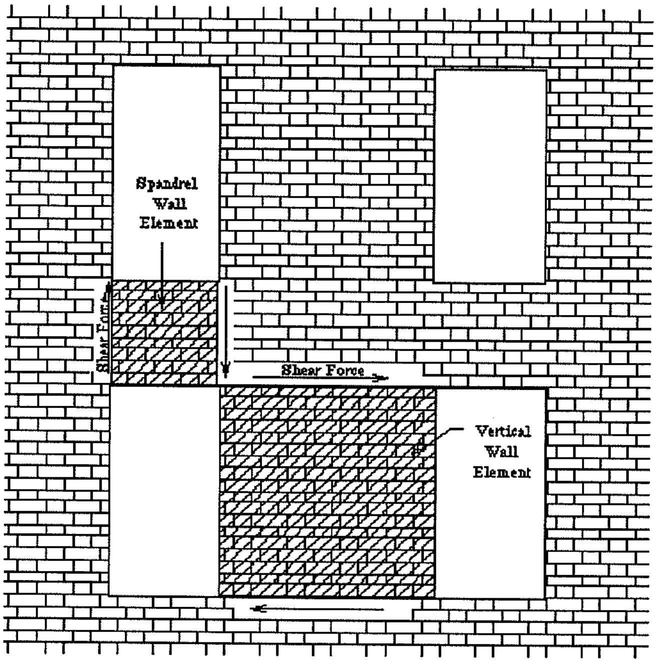

Figure 48 Types of wall elements

CASE A: As described earlier in this section, the model was first analyzed with all the masonry panels intact; the results are shown in Figures 51 and 52. At this stage, it was determined that each vertical wall element on the facade was under very significant axial tension at some time during the response. This led to the conclusion that each panel had developed bed joint cracks due to the direct tension and thus could not resist shear forces in combination with those tensile forces. Hence the calculated shear stresses in other panels were not valid.

Because of high axial tension in Panels 5, 11 and 12, the radical in Eq. 4.5.1 became imaginary, and ratios of average shear stress to cracking shear stress could not be calculated for those cases.

CASE B: All the vertical wall elements on the facade on Levels 4-9 were removed from the model. The spandrel wall elements were modelled as continuous beams. Also, Panel 13 was removed because of its narrow width and the presence of high axial tension (Figure 52). A zero thickness was assigned to the corresponding sub-element in the model. The model was again analyzed and the results are shown in Figures 53-55.

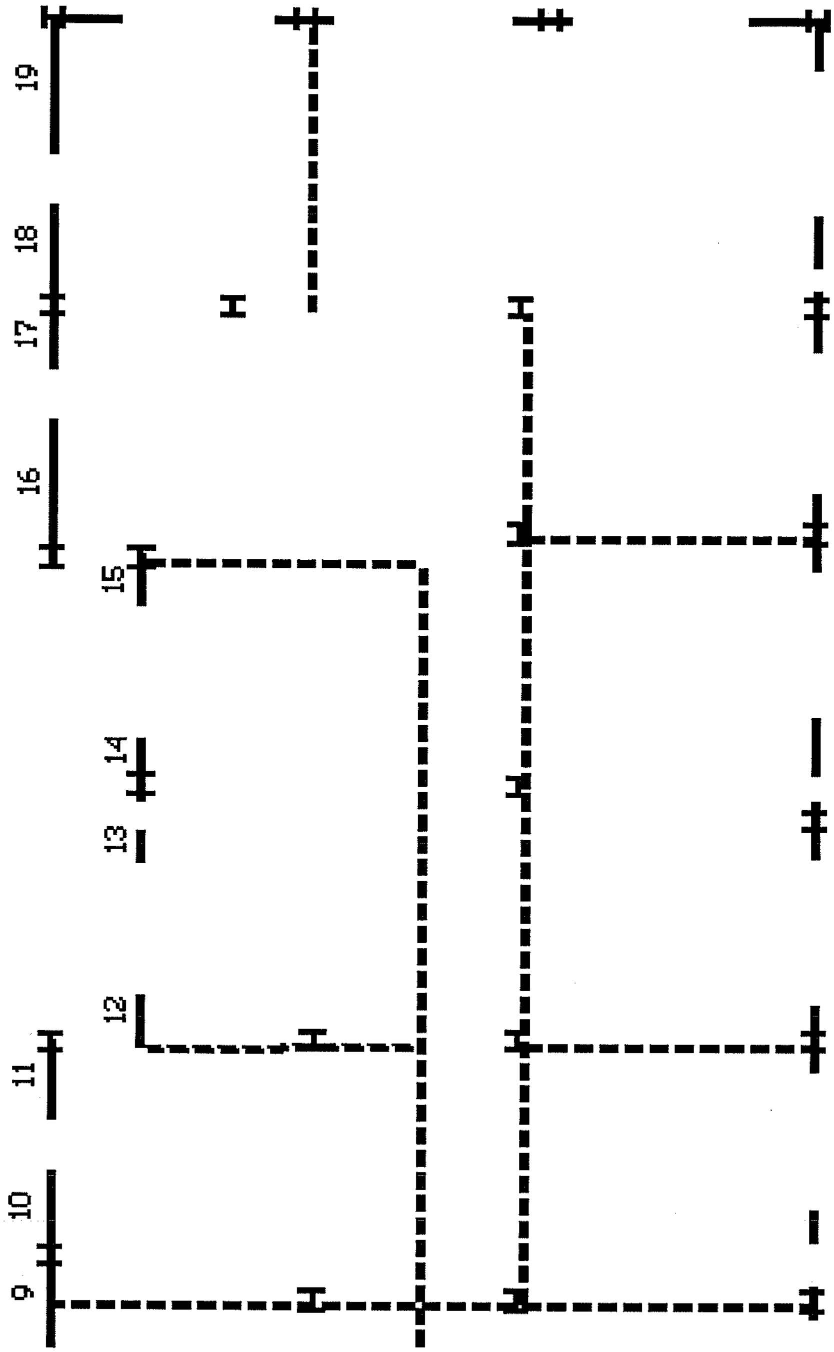

Figure 49 Numbering scheme for vertical wall elements (Hotel Woodrow)

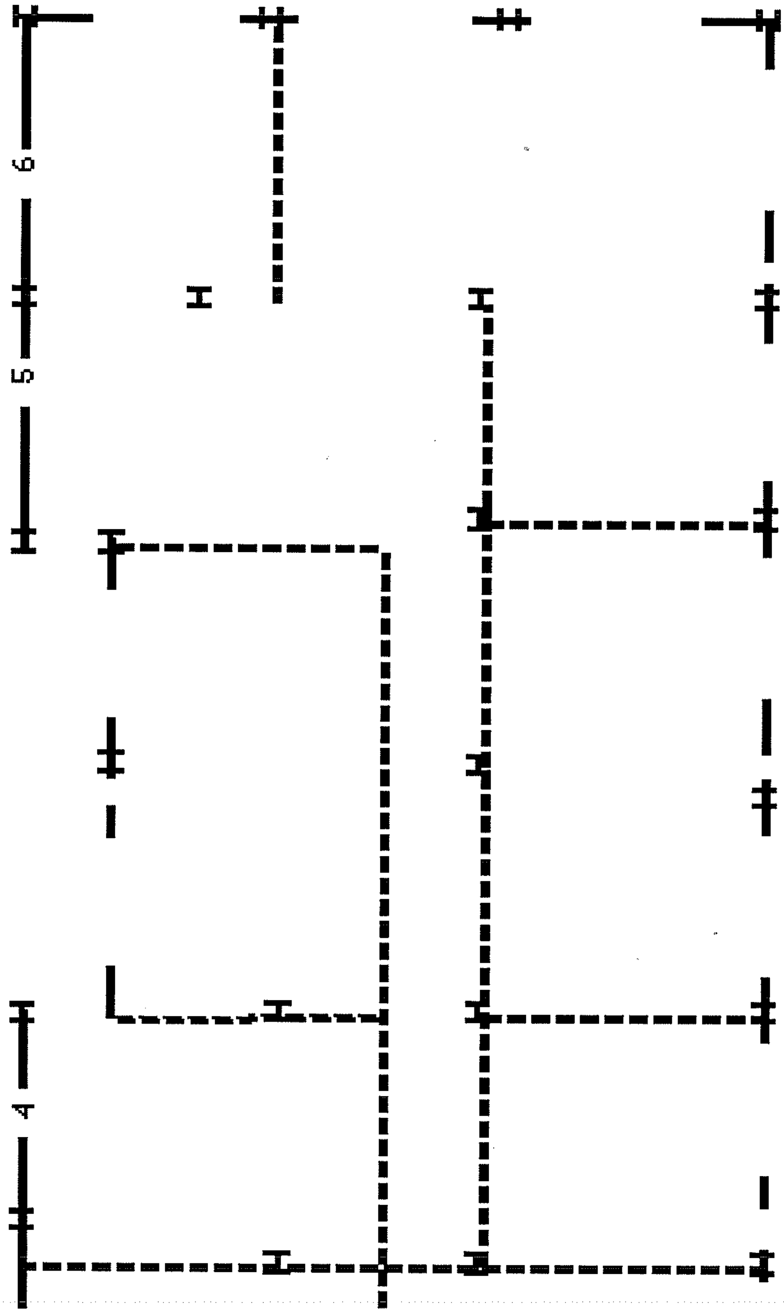

Figure 50 Numbering scheme for spandrel wall elements (Hotel Woodrow)

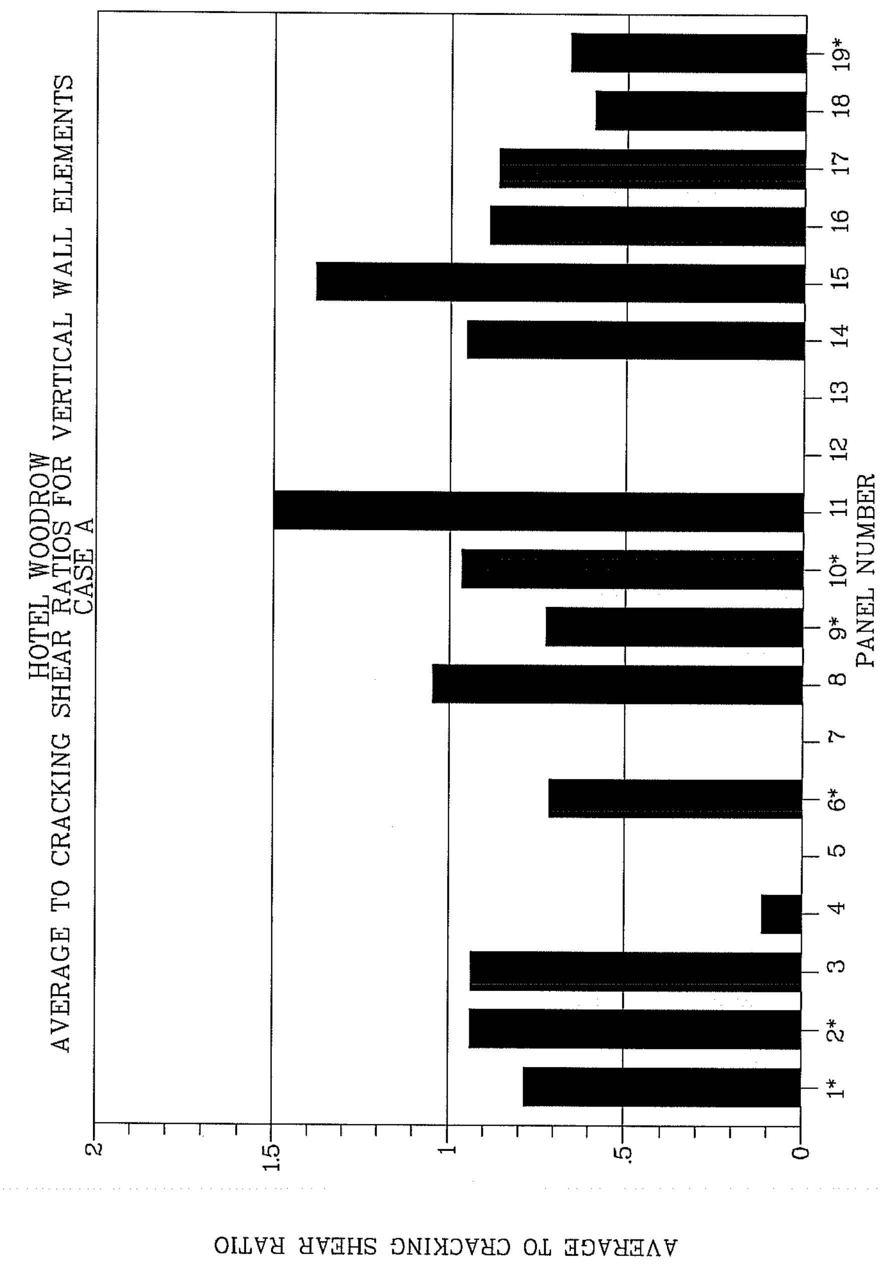

Figure 51 Average to cracking shear ratios for vertical wall elements (Case A)

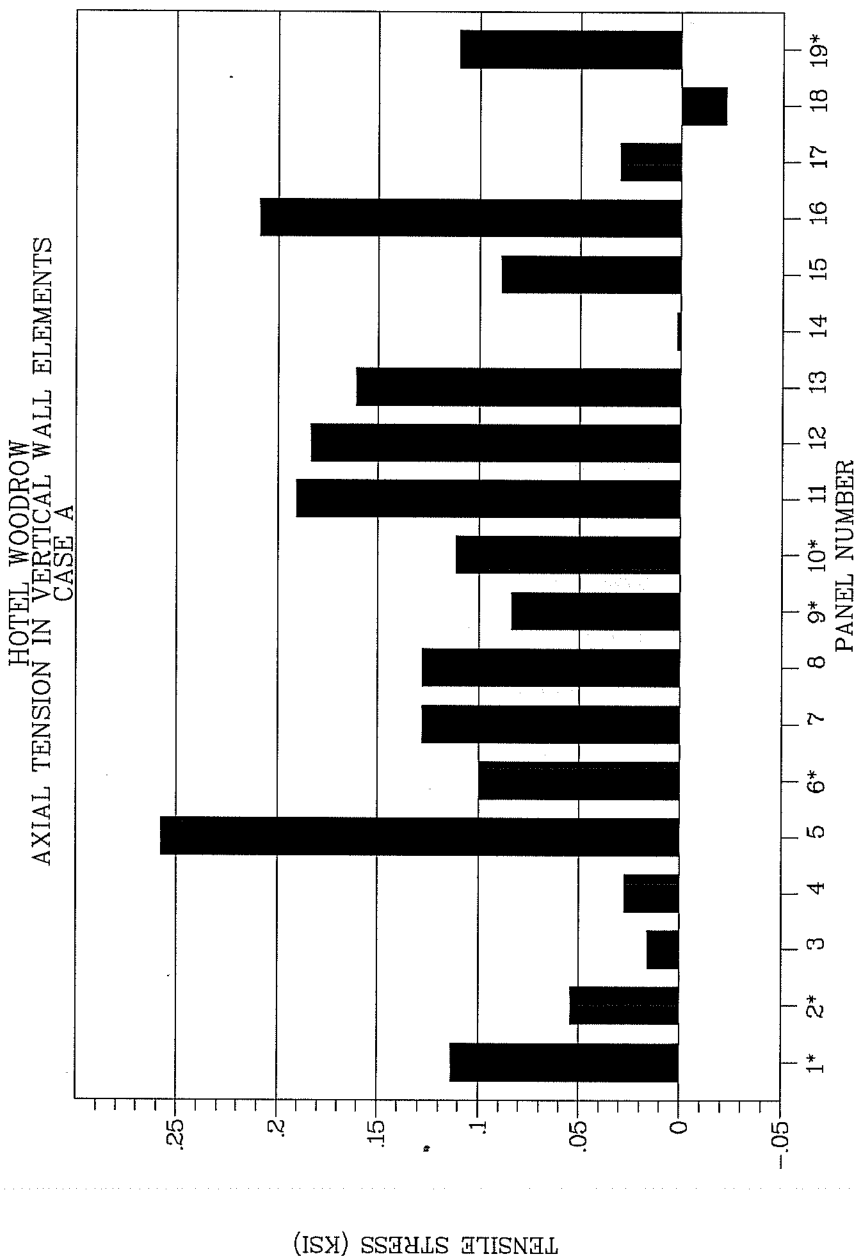

Figure 52 Axial tension in vertical wall elements (Case A)

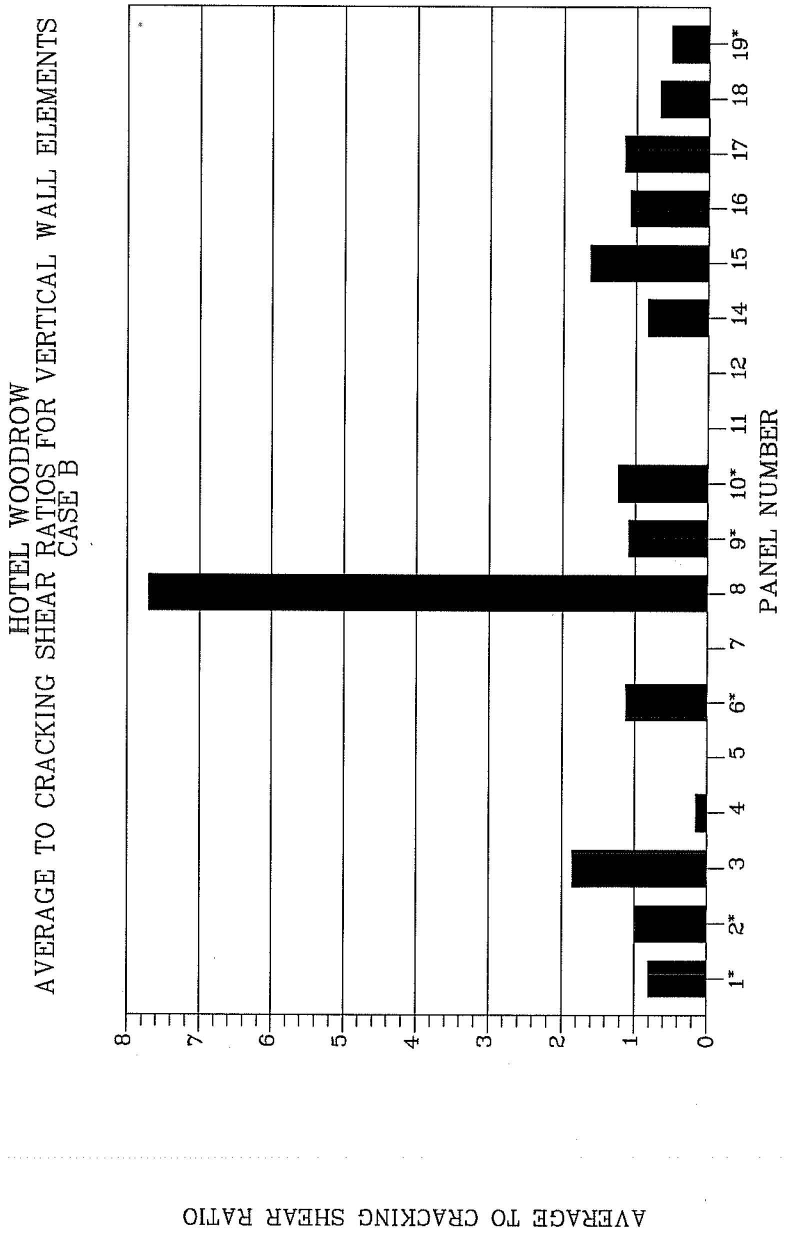

Figure 53 Average to cracking shear ratios for vertical wall elements (Case B).

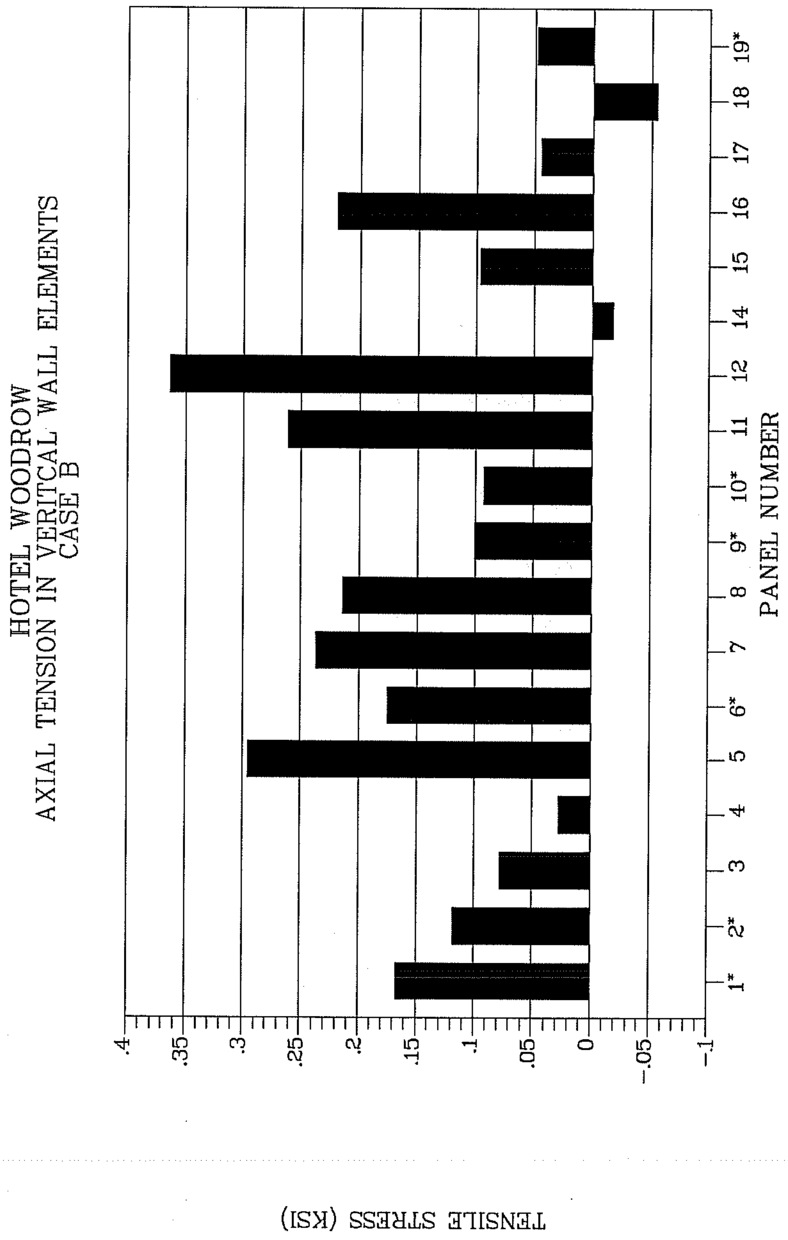

Figure 54 Axial tension in vertical wall elements (Case B)

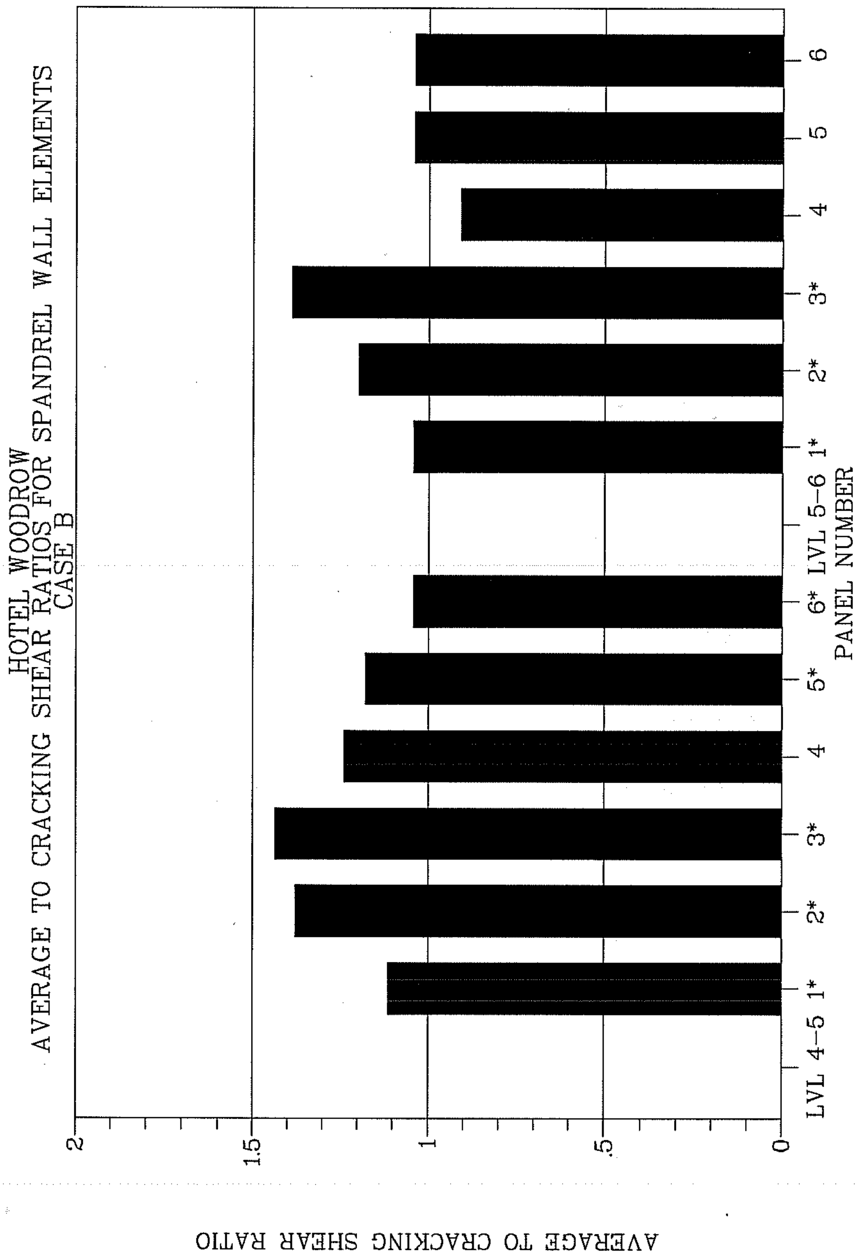

Figure 55 Average to cracking shear ratios for spandrel wall elements (Case B)

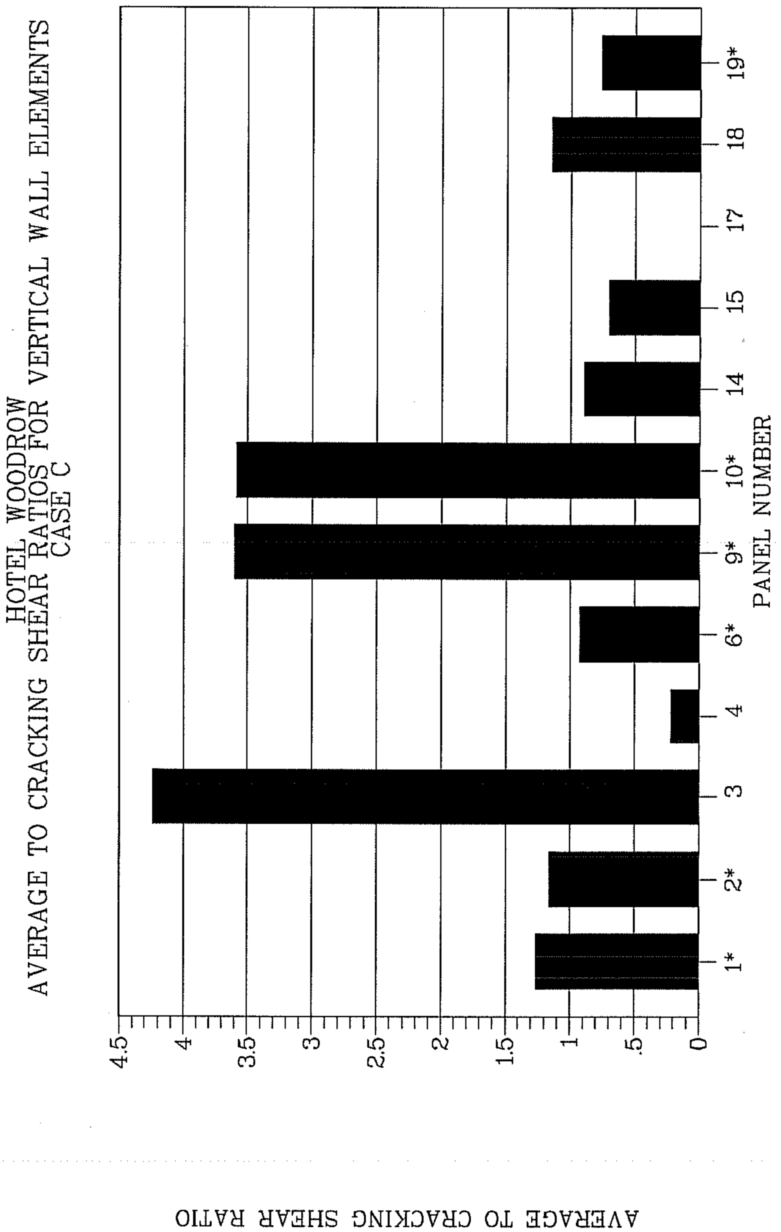

Figure 56 Average to cracking shear ratios for vertical wall elements (Case C)

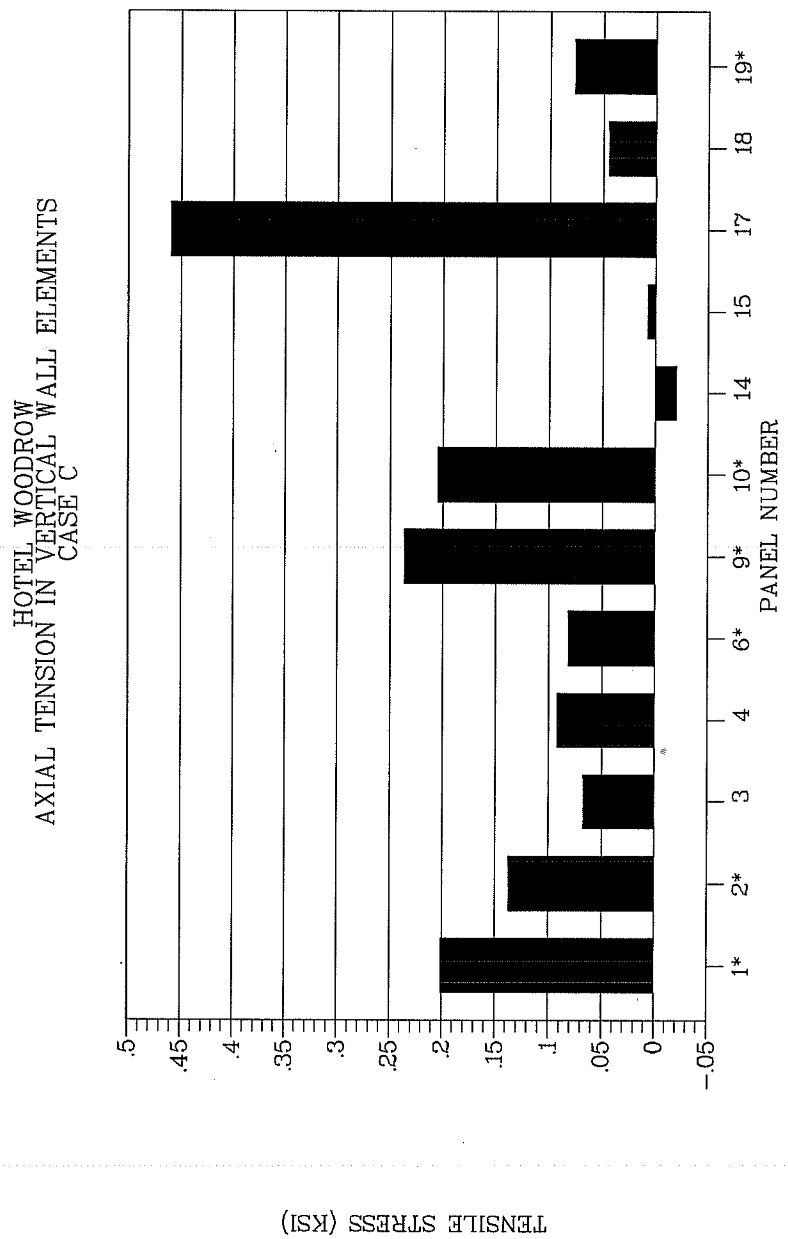

Figure 57 Axial tension in vertical wall elements (Case C)

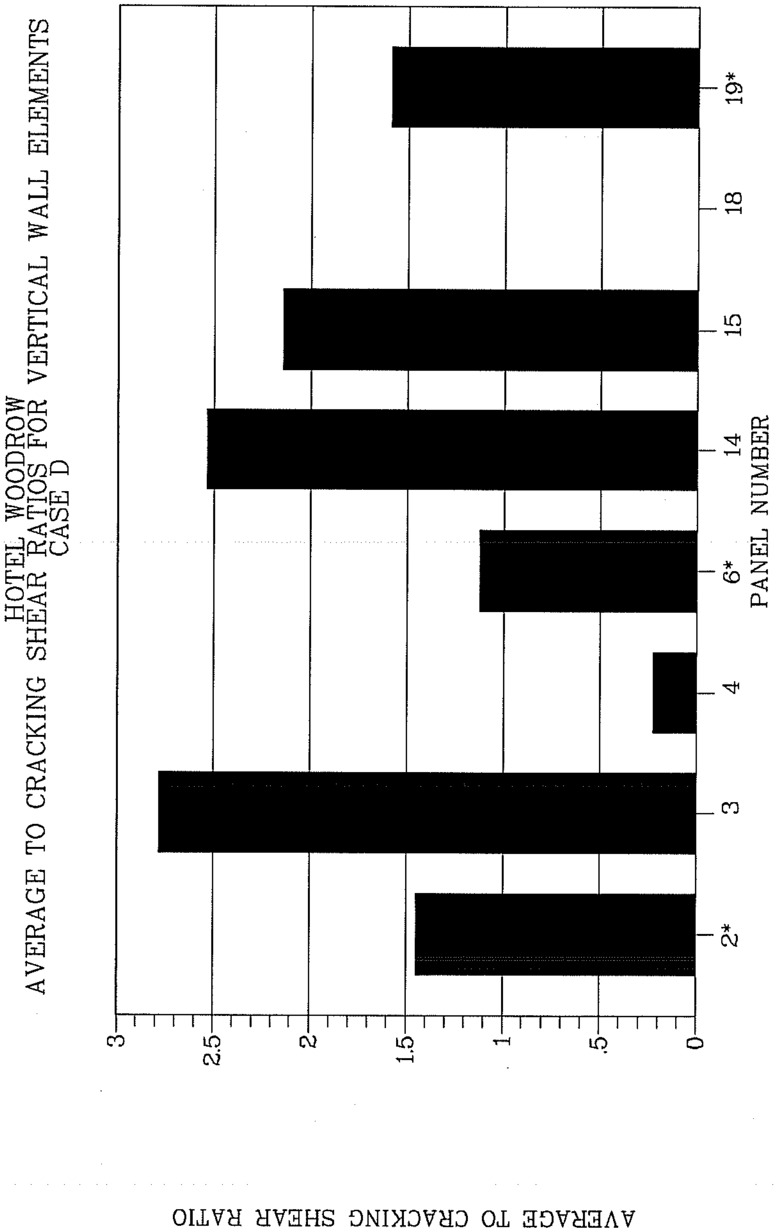

Figure 58 Average to cracking shear ratios for vertical wall elements (Case D)

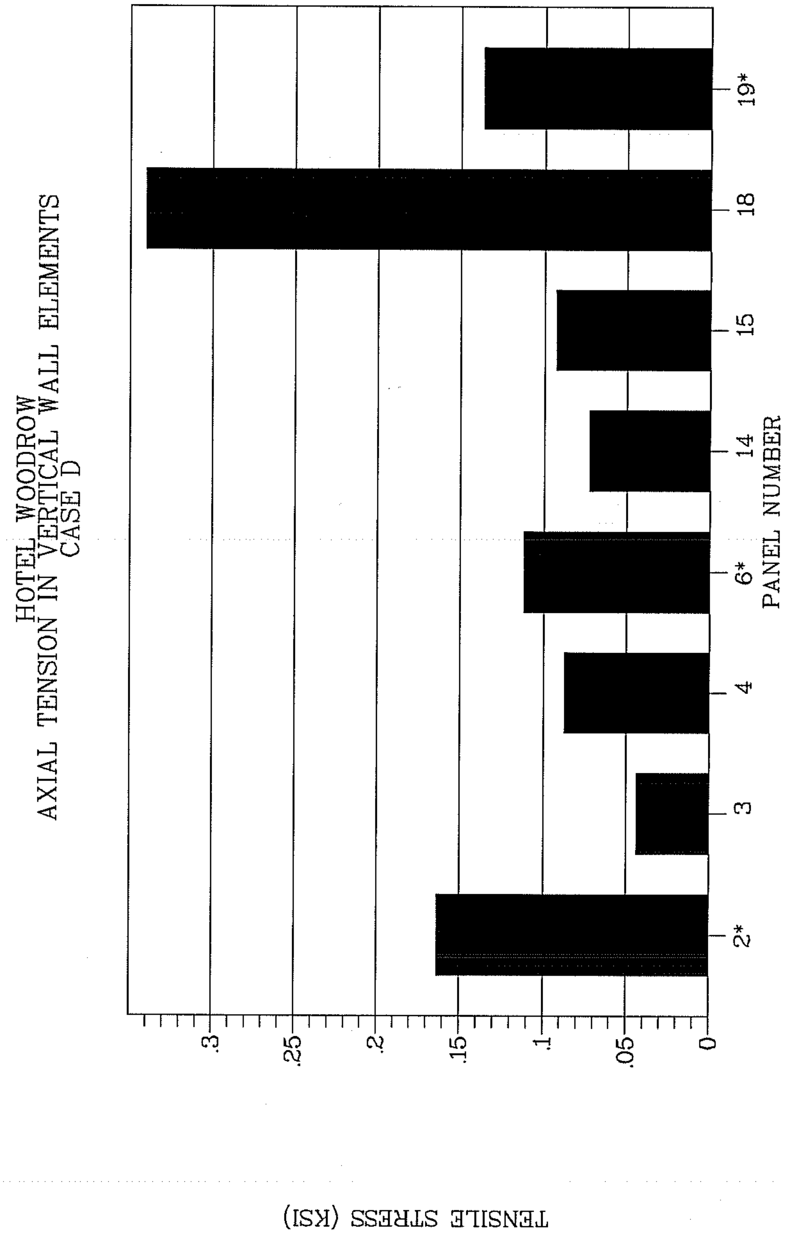

Figure 59 Axial tension in vertical wall elements (Case D)

From Figure 53 it is evident that the ratios of average shear stress to cracking shear stress for vertical wall elements 3, 8 and 15 were very high. However, from Figure 54 it is also obvious that a significant number of panels were still under high axial tensile stress, and therefore the calculated stresses were still not valid.

When a vertical wall element is removed, the vertical shear force in the portion of the wall below the opening goes to zero. As a result, the stress state in the corresponding spandrel wall element cannot be investigated further. Since from the results it is obvious that some vertical wall elements in the back walls would have to be removed, this made Case B the most suitable one for investigation of shear stresses in spandrel wall elements.

The spandrel wall elements were under low axial forces. However, these forces were still considered in the computation of average to cracking shear ratios. The results are shown in Figure 55. It can be seen that if shear cracking is estimated to occur at a ratio of 1.1, all elements between Levels 4 and 5 would have developed shear cracks. Also Panels 1, 2 and 3, between Levels 5 and 6, had developed shear cracks, and Panels 5 and 6 were quite close. Average to cracking shear ratios were also calculated for Elements 1 and 2, between Levels 6 and 7; the values were 0.85 and 1.0 respectively.

The fact that shear cracking occurred generally where it was predicted, and at ratios of actual to cracking stress close to 1.0, implies that the estimated strength of the masonry (f'_m = 3000 psi) was close to the actual value.

Vertical Wall Element 7 (Figure 44) was also investigated for compression failure at the corner; the extreme fiber compression in this panel was calculated as 3.308 ksi on Level 4 and 1.43 ksi on Level 5. Such high compressive stresses imply compressive failure in the extreme fibers of this panel.

Because of the high axial tensions in Panels 5, 7,11 and 12, the radical in the shear equation of Section 6.5.2 became imaginary; ratios of average shear stress to cracking shear stress could not be calculated for those cases.

CASE C: Using the results from the previous case, vertical Wall Elements 5, 7, 8, 11, 12 and 16, all of which had failed in direct tension, were also removed from the model by assigning zero thicknesses to the corresponding sub-elements. The modified model was reanalyzed, with the results shown in Figures 56 and 57.

It can be seen that vertical Wall Elements 1, 3, 6, 9 and 10 have average to cracking shear stress ratios much greater than 1.12. From Figure 56 it is also evident that Panels 1, 9, 10 and 17 had high axial tensile stresses. Keeping in mind the levels of stresses and aspect ratios of individual panels, it can reasonably be stated that Panels 1, 9 and 10 failed in shear, while Panels 2 and 18 needed further investigation. Panel 3 had developed high flexural tensile stresses, and hence also needed further investigation.

At this stage, average to cracking shear ratio for spandrel Wall Element 1, between Levels 6 and 7, was again checked, and had a value of 1.11. As discussed earlier, Panel 2 could not be investigated any more, because part of it had previously been removed.

Because of high axial tension in Panel 17, the radical in the shear equation became imaginary, and ratios of average shear stress to cracking shear stress could not be calculated.

CASE D: Vertical Wall Elements 1, 9 and 10 had failed in shear and vertical Wall Element 17 had failed in direct tension; hence, these elements were further removed from the model. The results for the new model are shown in Figures 58 and 59.

From Figure 56, it is evident that vertical Wall Elements 2, 3, 6, 14, 15 and 19 had average to cracking shear ratios in excess of 1.12. This implied that those panels would have cracked in shear. Panel 18 was under very high direct tension; while its shear ratio could not be calculated because of the negative radicals but it is clear from Figure 59 that the panel would probably have failed in direct tension.

Panels 3, 14 and 15 were under high flexural tensile stresses. This meant that these panels had developed deep cracks due to flexural tension and hence never experienced shears as high as those given by the analysis.

Spalling in vertical wall element 6 (Figure 45) was also investigated, and it was determined that the extreme fiber compression in this panel on Level 4 was 1.08 ksi. This stress was calculated for an 8-in. panel. However, spalling occurred in the 4-in.veneer which was covering the steel column. Therefore, it can be deduced that stress levels for that part of Panel 6 were probably much higher than this calculated value, and that spalling took place as a result.

6.5.4 Drifts: Lateral displacements under various combinations of gravity (static) and spectral loads, for Cases A and D, are shown in Figures 60-65. In these figures, N115W represents the spectrum applied in the global Y direction, and N25W represents the spectrum applied in the global X direction.

It can be seen that drifts in the Y direction at Node 1 (west wall, Figure 39) were less than drifts at Node 5 (east wall, Figure 39). This shows the significance of the flexible floor diaphragm in the lateral response of the structure. Because of the axial stiffness of the floor diaphragm, drifts in X direction were almost the same for all the floor nodes. Although the floor diaphragm at Level 2 had only two floor nodes, which were inside the floor segments, drifts were calculated for points that would correspond to Nodes 1 and 5 on other levels, by assuming a rigid-body motion of Level 2 as defined by its two floor nodes.

6.6 Comparison of Calculated Response with Observed Damage

By comparing stress states in various panels under Cases A through D, it can be seen that as the narrower panels failed in axial tension or flexure, shear stresses were transferred to wider wall panels which eventually failed in shear. This pattern of stress redistribution and failure is in complete harmony with the observed damage, and also gives a plausible explanation for the absence of shear cracks on the street facades. As a result, all the damage due to high shears was confirmed analytically, except in one panel where information was lost on account of removal of adjacent vertical wall elements.

Spot checks were also made to investigate the compression failures at the edges of vertical wall elements. It was determined that if failure was estimated to occur at a flexural compression of 1.4 ksi, all the damage would be confirmed.

The fact that shear cracking occurred generally where it was predicted, at the ratios of actual to cracking stress close to 1.0, implies that the estimated strength of the masonry (f'_m = 3000 psi) was close to the actual value.

6.7 Comments Regarding the Behavior of the Hotel Woodrow

The observed damage to the Hotel Woodrow agreed well with that predicted analytically. The overall seismic response of the Hotel Woodrow was complicated by several factors:

a) The flexible floor diaphragms permitted extensive redistribution of seismic shears among wall elements.

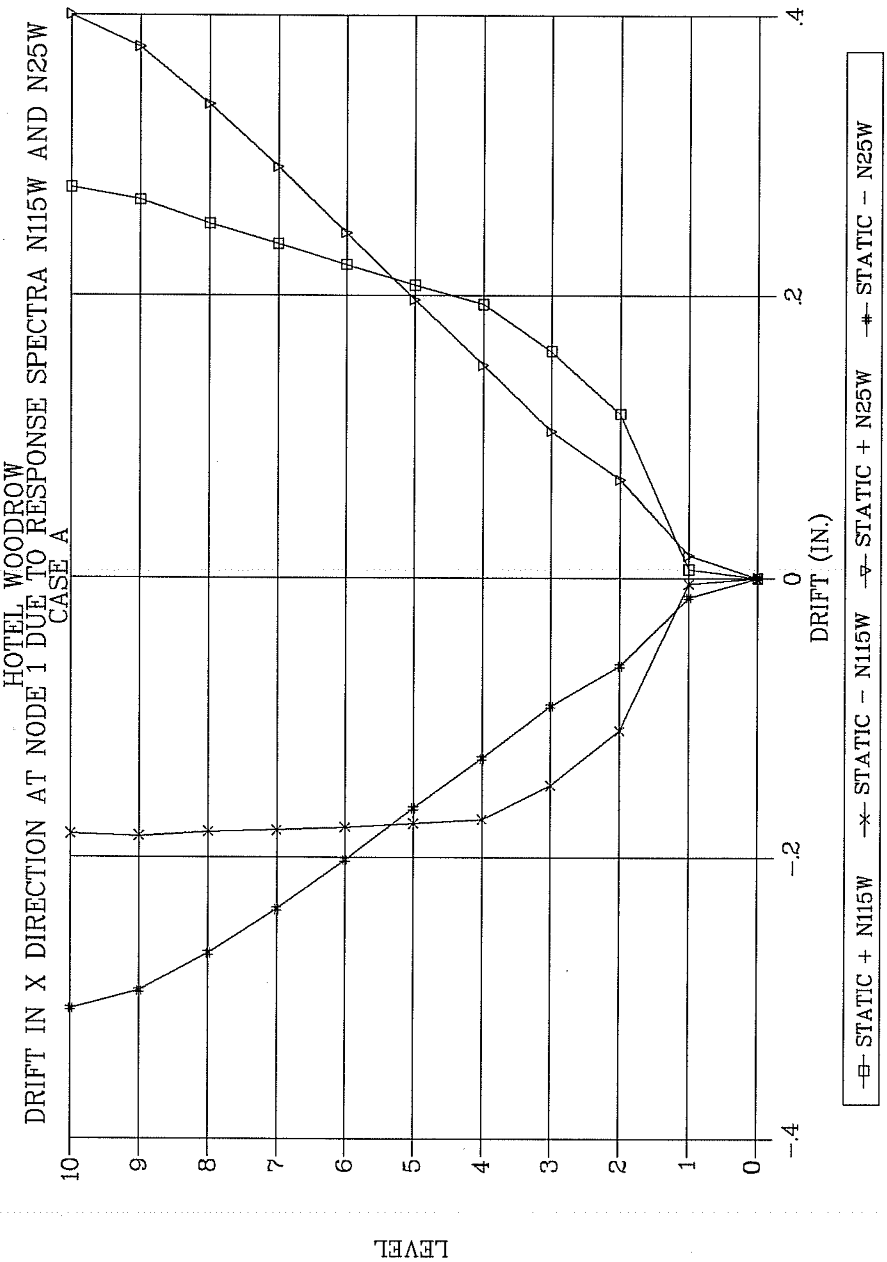

Figure 60 Drift in X direction at Floor Node 1 (Case A)

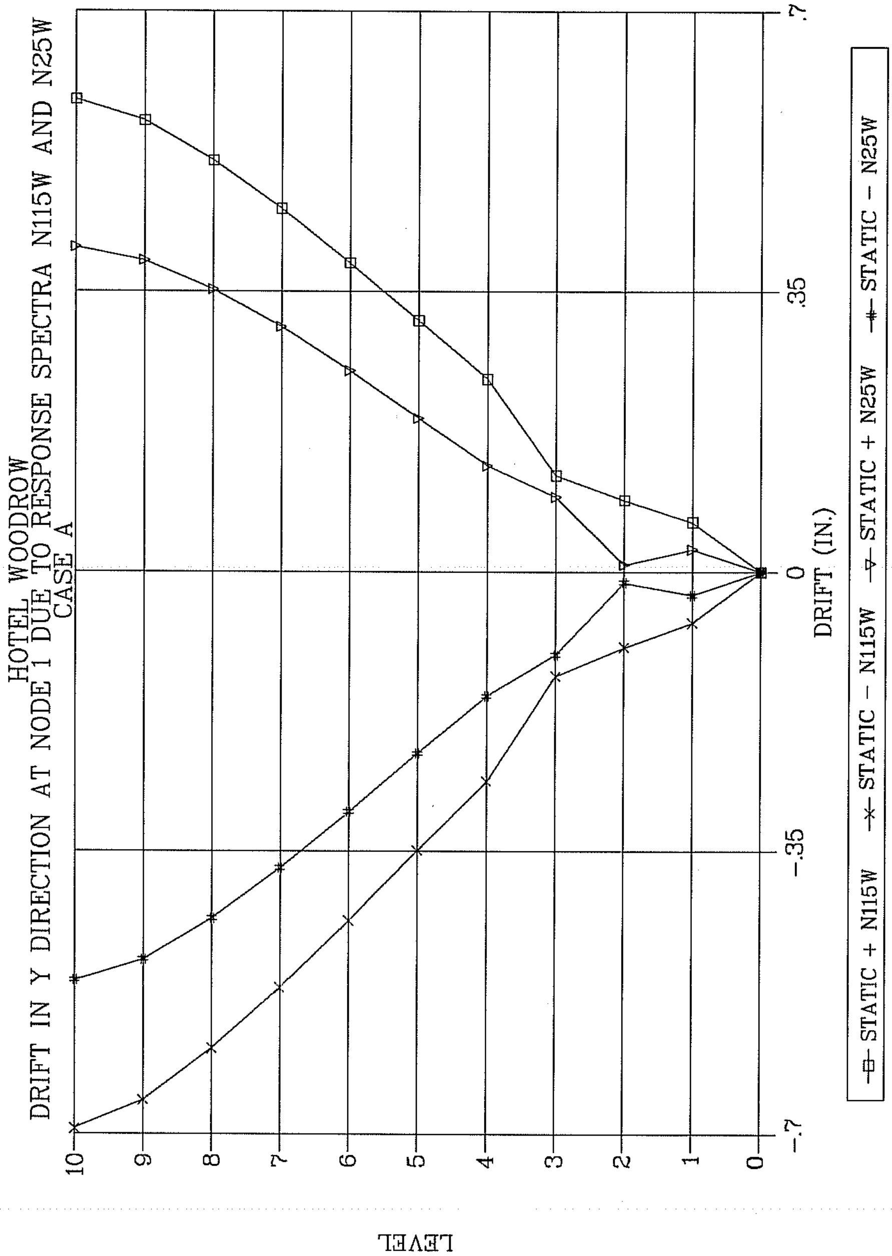

Figure 61 Drift in Y direction at Floor Node 1 (Case A)

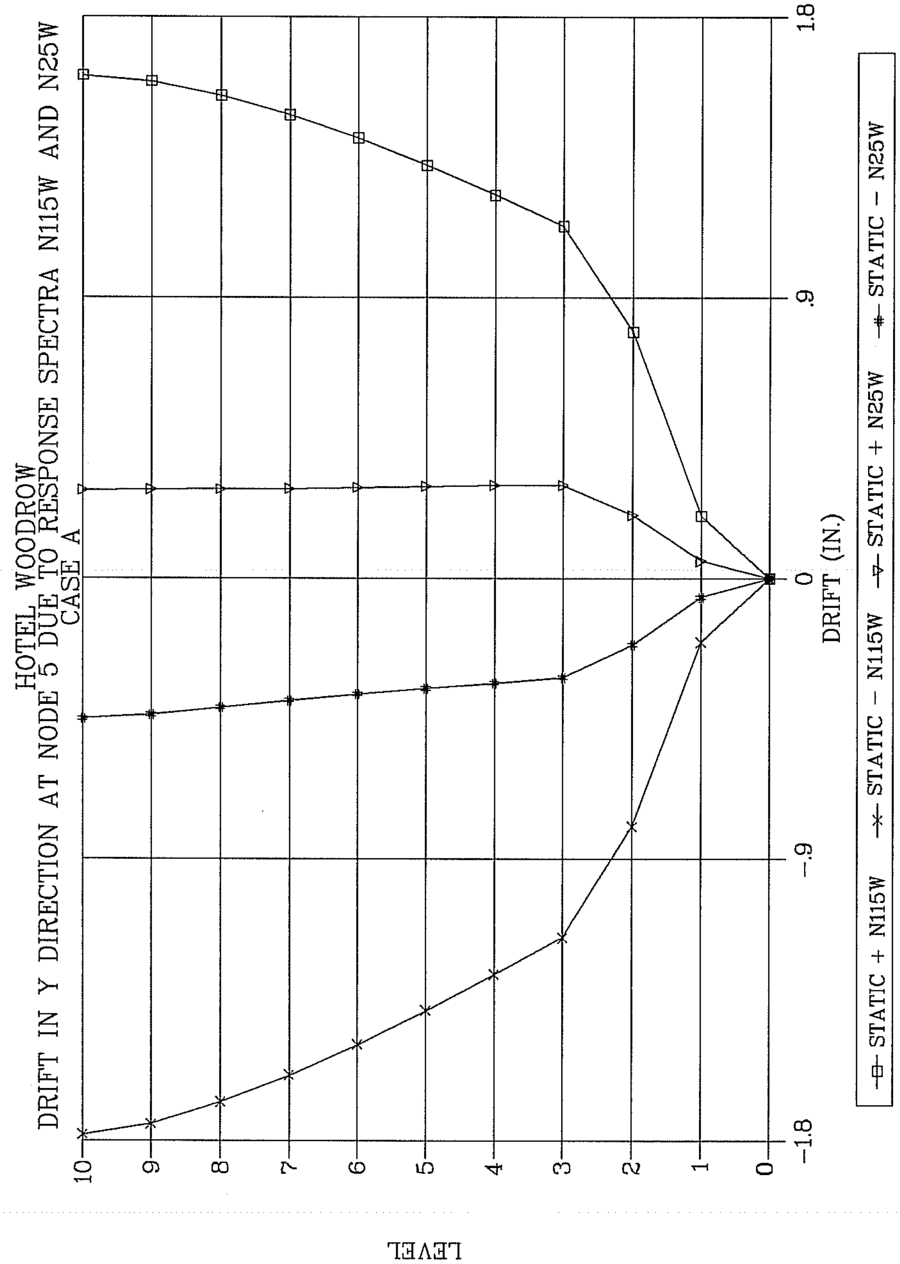

Figure 62 Drift in Y direction at Floor Node 5 (Case A)

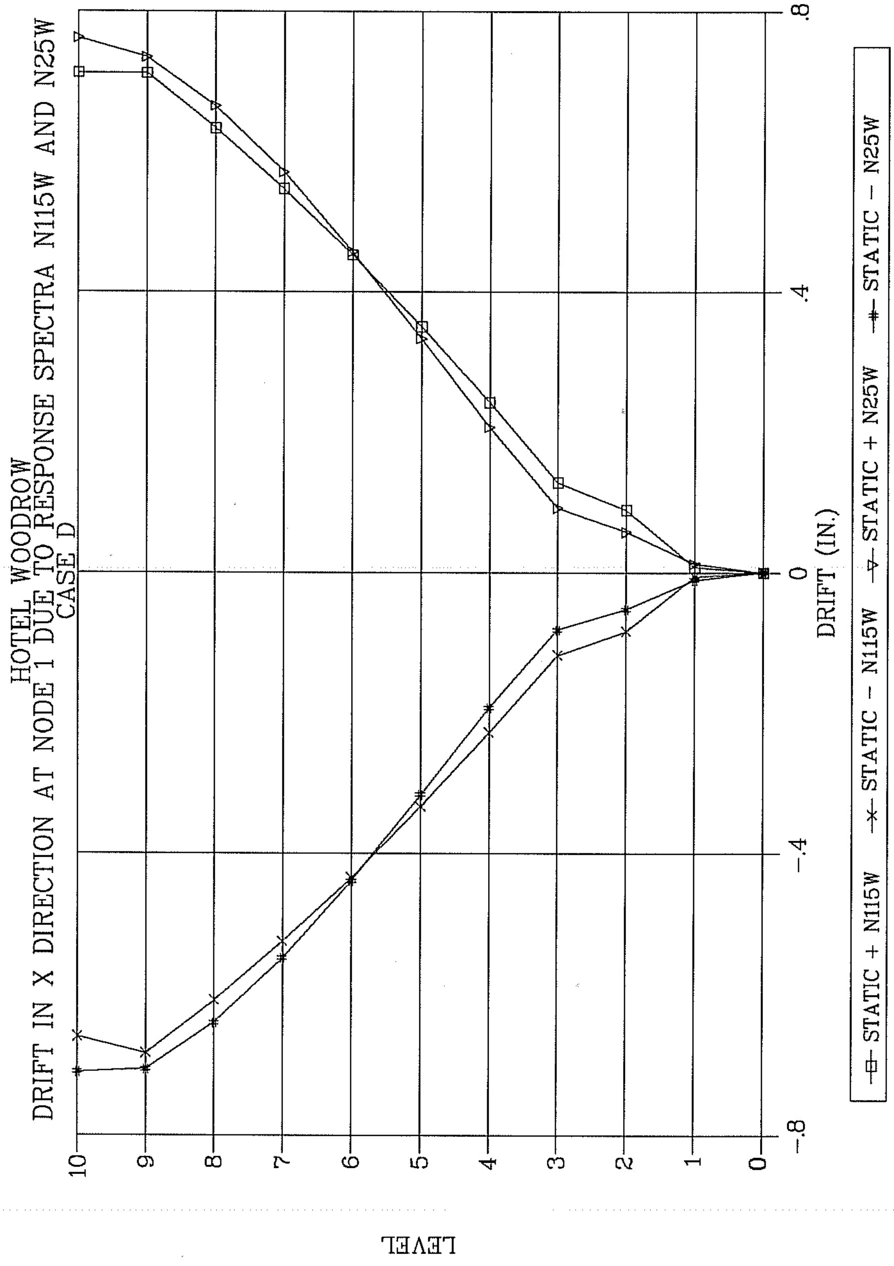

Figure 63 Drift in X direction at Floor Node 1 (Case D)

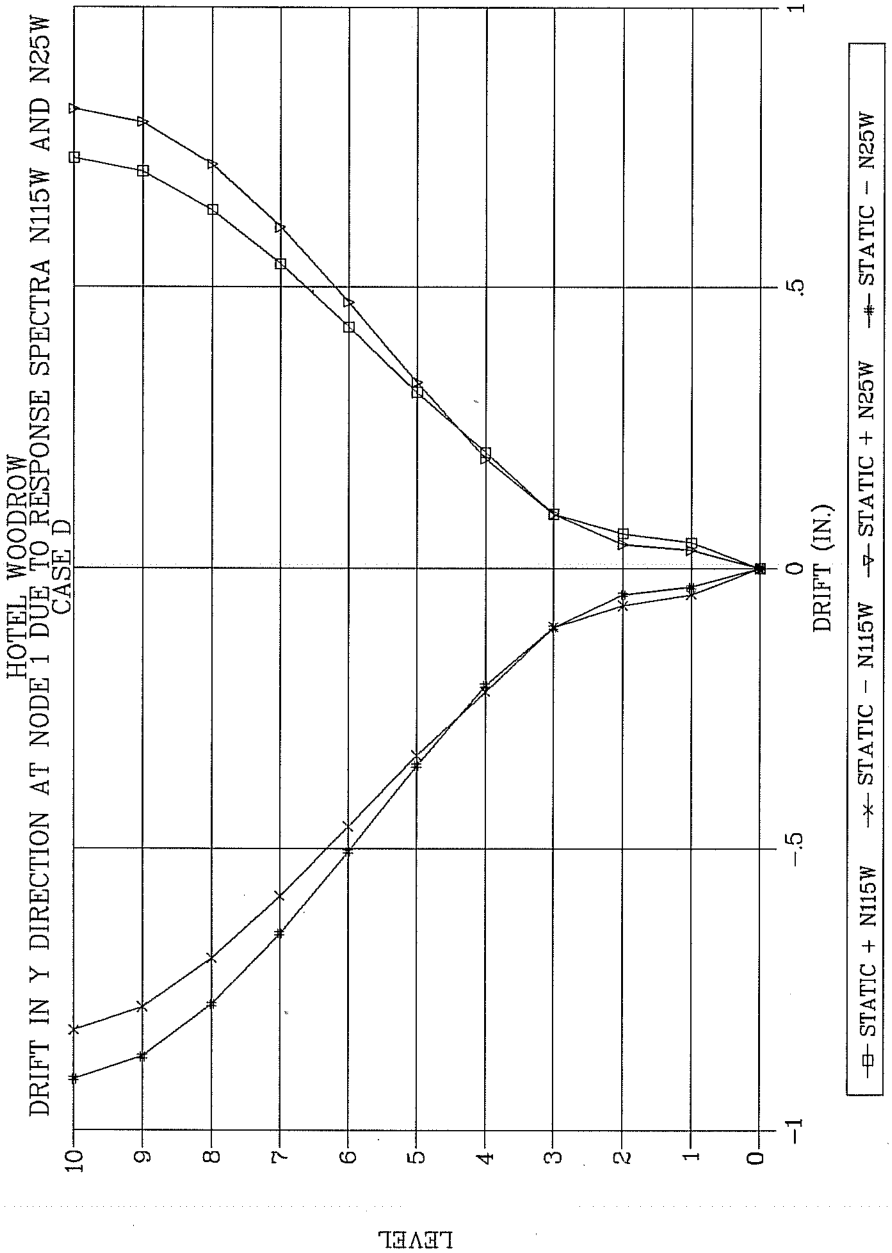

Figure 64 Drift in Y direction at Floor Node 1 (Case D)

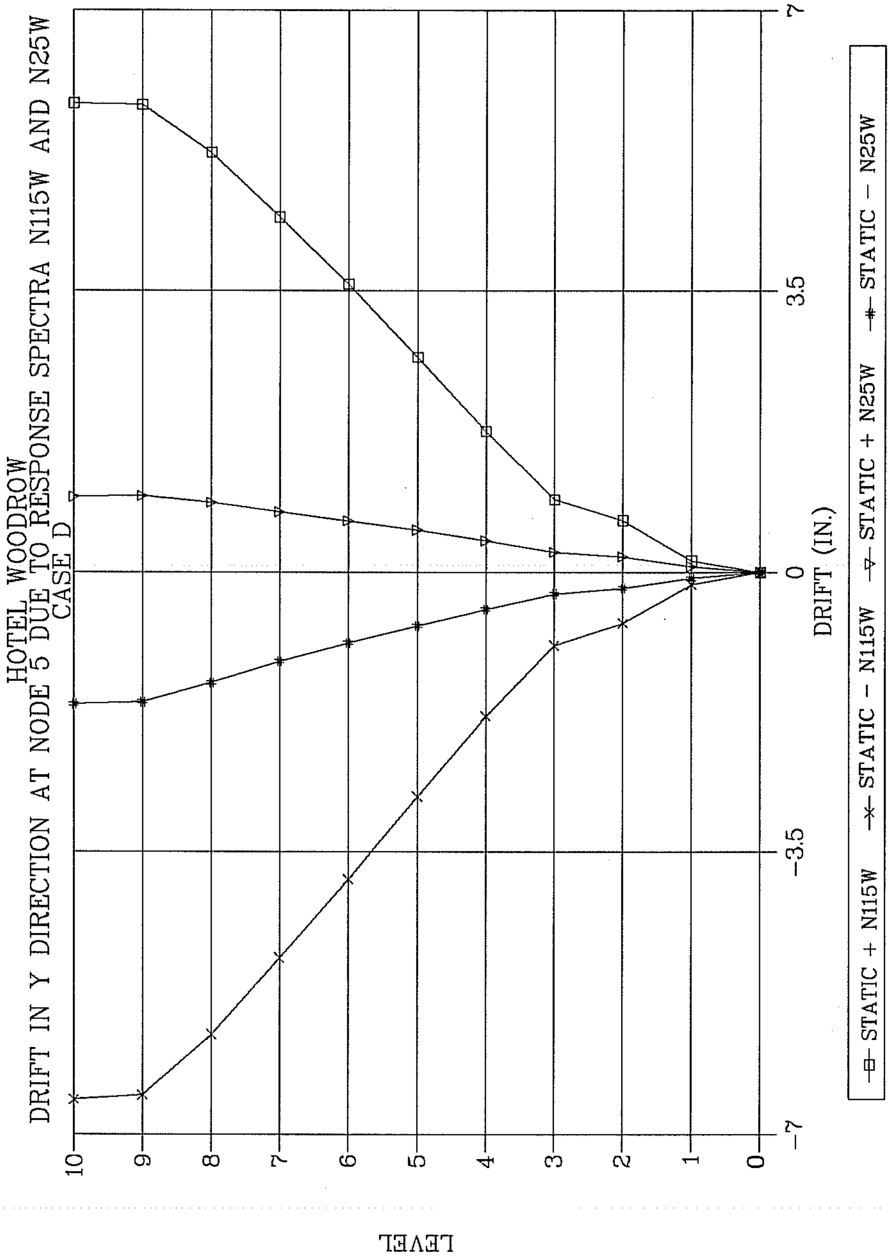

Figure 65 Drift in Y direction at floor node 5 (Case D)

b) While the masonry veneer and infill played a very significant role in the structure's response during the Loma Prieta event, the mechanical properties of the masonry could only be roughly estimated.

c) Observed failure patterns in masonry elements were not consistent with the application of a single failure criterion (for example, shear cracking) for the masonry.

d) Building response was sensitive to the assumed extent of degradation of the masonry elements.

e) Because its plan north and west walls (of infill plus veneer masonry) are stiffer and stronger than the south and east facades, the building can be visualized as a vertical masonry cantilever with an L-shaped cross-section. When such a cantilever is subjected to lateral forces parallel to either leg of the "L," it responds in biaxial bending. That is, forces in the building's transverse direction produce significant in-plane actions in the longitudinal direction, and vice-versa. When the building is loaded simultaneously in the transverse and longitudinal directions, combined actions in individual wall elements are difficult to determine by spectral analysis.

These difficulties were addressed (and hopefully resolved) by the following techniques:

a) The analytical model permitted floor diaphragm flexibility in the plane of the floor.

b) Masonry elements were idealized as substructured finite elements, permitting the application of multiple cracking criteria.

c) In developing the analytical models, masonry infill elements were idealized both as equivalent diagonal struts [21] and as substructured finite elements. No significant differences were noted in overall response calculated using those modelling approaches. Because it permitted the application of multiple cracking criteria, the substructured finite elements approach was preferred.

d) A sequential elastic analysis procedure was used to follow analytically the degradation of the masonry elements under combined in-plane actions.

CHAPTER 7

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

7.1 General Summary

In this report, the response of the following masonry and masonry veneer buildings during the 1989 Loma Prieta Earthquake has been investigated:

Loma Prieta Community Center	Reinforced Masonry Bearing Wall
Peninsula Office Building	Steel Frame with Masonry Veneer
2 Alhambra Street	Apartment Complex Wooden Frame with Masonry Veneer
Hotel Woodrow	Steel Frame with Masonry Infill and Veneer

The buildings were selected based on the following criteria:

1. The building construction should be one of the four types listed above.

2. Ground motion records should be available, either at the building site or reasonably close to it.

3. The building should be undamaged or lightly damaged. This permits more precise estimation of the force levels and deflections during the earthquake.

4. Plans of the building should be available, and the owner and engineer should be amenable to having their building studied.

Each building was modeled analytically, and was studied using linear elastic response spectrum analyses based on representative strong-motion records obtained from the California Strong Motion Instrumentation Program. The effects of soil-foundation flexibility were included in each model, either by placing discrete soil springs beneath the ground floor elements of each model, or by using a fictitious lower story whose column stiffnesses were adjusted to represent soil-foundation flexibilities. In each case, the soil stiffnesses were computed using the method proposed by Dobry and Gazetas.

Ground motion records for use in analyzing each building were selected based on the proximity of available recording instruments with respect to each building, and on the soil conditions underlying each building, compared to those underlying nearby recording instruments.

Results from the dynamic analyses were compared to the observed damage. For the first two buildings, analysis results were compared to the equivalent static lateral force requirements of the 1988 Uniform Building Code, and observations were made regarding the probable response of each building in a stronger earthquake.

Based on these observations, general conclusions are drawn regarding the earthquake response of buildings like those studied here. Finally, recommendations are made regarding changes to building codes which would improve the response of such buildings in future earthquakes.

7.2 Summary of Results for Each Building

7.2.1 Loma Prieta Community Center

Located approximately four miles from the epicenter, The Loma Prieta Community Center was subjected to very strong ground motions. It performed well in the earthquake, and damage was minor. Peak horizontal ground accelerations of 0.64g were recorded at the nearby Corralitos Station, and that record was used for analysis. The Loma Prieta Community Center was analyzed using SAP90, a general-purpose finite element program.

Analytical Model of the Building

The dynamic analysis reasonably modeled the response of the building, as predicted damage was consistent with observed damage.

Soil-foundation flexibility did not have a significant effect on the results. The building's fundamental period increased approximately 5%. The effective damping factor for the structure-foundation system increased from 3%, when not accounting for soil-foundation effects, to 5% including those effects.

Aspects of Behavior

The design intent of the Uniform Building Code was satisfied. The building did not collapse under a strong earthquake, and received only minor non-structural damage.

The computed dynamic shear stresses were significantly less than predicted cracking shear stresses for all of the masonry walls.

The computed dynamic out-of-plane flexural stresses for the north and south walls of the gymnasium were found to be 2 to 4 times greater than the predicted flexural cracking stress resulting in the observed vertical cracks.

The computed dynamic moments in the CMU pilasters would be expected to cause flexural cracking, as observed in several locations.

The long-span trusses supporting the sloping roof of the gymnasium responded strongly to the simultaneous application of one vertical and two horizontal components of ground motion.

The west classroom wing responded strongly in the north-south direction, as can be seen by comparing the magnitude of the response spectrum shear in the west wall to the UBC static lateral shear in the same wall. The dynamic response is roughly 10 times greater than that for the UBC loads. The roof of the west wing pulled slightly away from the gymnasium wall, resulting in water leakage along this joint.

7.2.2 Peninsula Office Building

The Peninsula Office Building, located on the San Francisco Peninsula, also performed well, suffering only minor damage in the Loma Prieta event. Because it was located in the general vicinity of a recording station on the southern Peninsula (Palo Alto), and had similar underlying soil conditions, that record was used for analyses. Peak horizontal ground accelerations of 0.21 g were obtained there. The Peninsula Office Building was analyzed using ETABS, a special-purpose program for building response.

Analytical Model of the Building

The dynamic analysis reasonably modeled the response of the building, as predicted damage was consistent with observed damage.

Soil-foundation flexibility did not have a significant effect on the results. The building's fundamental period increased from 0.72 to 0.75 seconds, and the effective damping factor for the structure-foundation system increased from 2% when soil-foundation flexibility was neglected, to 5% when this flexibility was included.

The insertion of a dummy diagonal into the building model is a simple yet effective method of computing horizontal racking of the structure at any panel location.

<u>Aspects of Behavior</u>

The design intent of the UBC code was satisfied. The building received only minor non-structural damage under a moderate earthquake.

The building is very flexible; its total drift was computed as 1.9 inches under response spectrum loading, neglecting the stiffness contribution from the exterior brick veneer panels. The corresponding maximum story drift ratio was 0.0047.

Including the exterior brick veneer panels in the building model reduces the calculated fundamental period of the structure from 0.75 to 0.55 seconds, and reduces the total building drift from 1.9 to 1.3 inches. The maximum story drift ratio is then 0.0035.

The computed cracking shear stress in the veneer panels under response spectrum loading exceeded the predicted cracking shear stress for all but one panel; this is consistent with the observed damage.

The 1988 UBC requires a clearance of [3 x (R_w / 8)] times the calculated elastic story drift or 1/2 inch, whichever is greater, between the structural frame and exterior nonbearing, nonshear wall panels. This results in a clearance requirement of approximately 1.50 inches in some locations. This is significant, and it may be difficult for the designer to properly detail for this amount of clearance.

In a stronger earthquake, bracing members would likely have buckled and veneer panels would have suffered more serious cracking.

The estimated horizontal racking of the structure in a stronger earthquake is approximately 1.50 inches, consistent with the clearance required by the UBC. However, detailing of a 1-1/2 inch gap may present problems for the designer.

Results from the response spectrum analysis are close to those obtained with the 1988 UBC equivalent static analysis. The peak horizontal ground accelerations at this site were on the order of 0.21g (fairly low). Review of Figure 24 shows that for the building's fundamental period of 0.75 seconds, the response spectrum for this earthquake and the implied 1988 spectrum are fairly close.

The use of thin bricks rather than standard units in the masonry veneer reduces the mass of the structure by approximately 4%, and consequently reduces story shears and overall building drift.

The use of masonry veneer to enclose steel framed structures is a viable type of construction if the veneer is isolated from the structure.

7.2.3 2 Alhambra Building

This building, located in the Marina district of San Francisco, suffered substantial damage to its ground story. Because the building was located on mud fill, a ground motion recorded on a nearby rock outcrop was selected and later modified for soil effects. An analytical model was prepared using the program SK-COMBAT. Effects of masonry cracking were addressed by conducting sequential linear analyses with the damaged elements removed or modified.

The 2 Alhambra building is a wooden frame structure with clay masonry veneer on the ground level. The structure derived most of its shear resistance at the ground level from the masonry veneer. At the time of the earthquake, the masonry veneer probably also carried significant gravity loads because of the deterioration of wood and expansion of masonry over the years.

The effects of soil-foundation flexibility were quite significant because of the soft underlying soils. Ground motion was amplified significantly by the soil, resulting in increased earthquake loads. Following bed joint cracking at the extreme fibers, which reduced the effective area available for shear resistance, the masonry veneer at the ground level lost most of its lateral stiffness. Once the entire masonry veneer was lost at the ground level, the ground floor acted as a soft story during the earthquake, isolating the upper stories and preventing significant damage there. At this stage, the maximum drift at the ground level was about 70% of the roof drift and the structure now relied on the very few wooden stud walls for lateral loads resistance at the ground level. The observed damage to the 2 Alhambra building agreed well with that predicted analytically. The analytical approach used therefore seems reasonable.

Masonry veneer at the street level was essentially destroyed by the earthquake. The remaining stiffness at the ground floor was due only to the wooden siding.

After the loss of the veneer, the 2 Alhambra building probably experienced first-story lateral drifts of more than one inch, corresponding to a first-story lateral drift ratio of about 1%. Given the lack of lateral stiffness without the veneer, had the building been subjected to an event as strong as Loma Prieta but of longer duration, it might well have collapsed. The same comment would apply in the case of a stronger earthquake. Although the masonry veneer of the 2 Alhambra building was not originally conceived as acting structurally, it did cause reductions in first story drift ratios.

Because the 2 Alhambra building was not formally designed, it did not seem necessary to discuss the extent of its compliance with current building codes, nor to discuss ways in which its seismic performance might be improved. However, it is worthwhile to note that the seismic retrofit of the building, involving the installation of shear diaphragms at all levels, and X-braces at the ground floor, seems appropriate in view of the analysis results obtained here.

7.2.4 Hotel Woodrow

This building, located in downtown Oakland, suffered some cracking of masonry shear walls and infills. Ground motion records were available from an instrument located reasonably close by, and founded on soil similar to that underlying the hotel. That record was used for analysis. An analytical model was prepared using the program SK-COMBAT. Effects of masonry cracking were addressed by conducting sequential linear analyses with the damaged elements removed or modified.

The Hotel Woodrow is a steel frame structure with masonry veneer and infill at all levels above the ground. The steel frame carried gravity loads while the masonry also carried lateral loads in addition to the gravity loads. The observed damage to the Hotel Woodrow agreed well with that predicted analytically. The overall seismic response of the Hotel Woodrow was complicated by several factors:

a) The flexible floor diaphragms permitted extensive redistribution of seismic shears among wall elements.

b) While the masonry veneer and infill played a very significant role in the structure's response during the Loma Prieta event, the mechanical properties of the masonry could only be roughly estimated.

c) Observed failure patterns in masonry elements were not consistent with the application of a single failure criterion (for example, shear cracking) for the masonry.

d) Building response was sensitive to the assumed extent of degradation of the masonry elements.

e) Because its plan north and west walls (of infill plus veneer masonry) are stiffer and stronger than the south and east facades, the building can be visualized as a vertical masonry cantilever with an L-shaped cross-section. When such a cantilever is subjected to lateral forces parallel to either leg of the "L," it responds in biaxial bending. That is, forces in the building's transverse direction produce significant in-plane actions in the longitudinal direction, and vice-versa. When the building is loaded simultaneously in the transverse and longitudinal directions, combined actions in individual wall elements are difficult to determine by spectral analysis.

These difficulties were addressed (and hopefully resolved) by the following techniques:

a) The analytical model permitted floor diaphragm flexibility in the plane of the floor.

b) Masonry elements were idealized as substructured finite elements, permitting the application of multiple cracking criteria.

c) In developing the analytical models, masonry infill elements were idealized both as equivalent diagonal struts [21] and as substructured finite elements. No significant differences were noted in overall response calculated using those modelling approaches. Because it permitted the application of multiple cracking criteria, the substructured finite elements approach was preferred.

d) A sequential elastic analysis procedure was used to follow analytically the degradation of the masonry elements under combined in-plane actions.

As a result of earthquake, most of the damage occurred in the wider masonry panels while all the narrower masonry panels remained essentially unscathed. This state of damage in Hotel Woodrow was confirmed analytically. The flexible floor diaphragms allowed significant stress redistribution among wall elements. As a result, when the narrower masonry panels failed in direct tension, their shear forces were transferred to wider masonry panels, which consequently failed in shear.

7.3 Summary of Observations Regarding Analytical Modeling

The analytical models used in the analyses seem to accurately reflect the behavior of the buildings during the earthquake. Provided that cracking was reflected through sequential adjustment of stiffness properties, linear elastic models gave reasonable results. Satisfactory correlation was achieved with respect to the location and the severity of predicted damage, compared to that observed.

The insertion of very flexible diagonal elements into the Peninsula Office Building model provided a simple yet effective method of computing interstory drift of the steel frame at veneer panel locations, enabling the comparison of veneer stresses with estimated cracking capacities.

The effects of soil-foundation flexibility had a significant effect only in the case of the 2 Alhambra building, for which ground motions were amplified by soft soil deposits. This phenomenon should be considered in analyses of buildings founded on soft soil.

In the Hotel Woodrow, the low in-plane stiffness of the wooden plank floor diaphragms permitted significant redistribution of shears and moments among the wall elements at each floor level. This redistribution would not have been so pronounced had the diaphragms been considered rigid. For such buildings in-plane flexibility of floor diaphragms is significant, and should be considered in analytical models.

In the absence of specific information on the elastic stiffness and cracking strength of old masonry, a comparison between observed and predicted damage provided valuable estimated of those properties for the Hotel Woodrow. This approach, involving a series of comparisons between damage observed and that predicted based on various assumed properties, is recommended for such cases.

7.4 Summary of Probable Response of Buildings in Stronger Earthquakes

Of the four buildings studied here, the Loma Prieta Community Center and the Peninsula Office Building were modern structures designed by modern earthquake codes. It was considered appropriate to investigate the probable response of those two buildings in stronger earthquakes.

7.4.1 Loma Prieta Community Center. The Loma Prieta Earthquake represented a significant event for this building, due to its proximity to the epicenter. Peak horizontal ground accelerations of 0.64g were recorded 10 miles southeast, at the Corralitos Station. Due to the magnitude of the Loma Prieta event, a significantly stronger earthquake is not anticipated at this site. However, several observations can be made pertaining to the probable response of the building if subjected to a stronger earthquake.

7.4.2 Peninsula Office Building. The Peninsula Office Building is a flexible structure. If it were subjected to a moderate to strong earthquake, it would undergo lateral drift greater than that experienced in the Loma Prieta event. During a strong earthquake, it is likely that lateral shear forces would be transferred to the non-structural exterior masonry veneer panels, causing significant cracking. The increased lateral forces would probably cause buckling of bracing members. In addition, high lateral drifts would most likely result in noticeable damage to the interior non-loadbearing walls.

Shear cracking of the concrete masonry walls at the perimeter of the gymnasium would not likely be a problem. The predicted cracking stresses for these walls exceeded the computed shear stresses by a factor of five for the Loma Prieta Earthquake. However, some isolated diagonal cracking would be expected around wall openings.

The west classroom wing of the building would most likely be damaged, due to significant response in the north-south direction. Glass would again break along the west wall of the building, accompanied by damage to the interior walls in this wing. Also, significant forces would develop in the connections which tie the east and west classroom wings to the main gymnasium structure. Damage to these connections would likely occur, and the roof over the east and west wings would pull away from the gymnasium walls, as occurred during the Loma Prieta event.

7.5 Conclusions

The models developed here are reasonably good representations of the actual structures. The level of effort involved in this modelling, while considerable, is clearly both possible and economically feasible in a modern structural design office. It is also clear that such understanding of the response of the structures during the earthquake could not have been possible without the use of modern analysis programs.

Bearing wall masonry, as used in construction of the Loma Prieta Community Center, can serve as a good architectural and structural use of masonry. For the Loma Prieta Community Center, the dynamic response spectrum results differed significantly from the results using the equivalent static lateral force procedure of the 1988 UBC. This can be attributed to the mass and stiffness irregularities of this building and to the strong ground motions experienced at this site. The west wing behaved as an appendage to the gymnasium which is much stiffer. The sloped roof over the gymnasium introduced coupling between the horizontal and vertical responses of the roof trusses. This coupling, combined with large horizontal and vertical ground motions, caused significant vertical response of the trusses.

Masonry veneer construction as an architectural cladding will perform well if the veneer is isolated from the structure. As seen for the Peninsula Office Building, the required clearance can be large if the frame is flexible.

7.6 Recommendations for Design

The use of modern computer programs with good dynamic analysis and graphic capabilities is recommended for the seismic analysis and design of buildings with significant irregularities in mass and/or stiffness, as was the case with the Loma Prieta Community Center. The large out-of-plane bending of the walls, the large vertical movements of the roof trusses, and the large north-south displacement of the west classroom wing could not have been predicted using the UBC equivalent static lateral force method. Equivalent static methods should be used with caution in buildings with mass or stiffness irregularities, even if the buildings have only a single story.

Good detailing of structural connections is essential. Analyses may not always accurately predict the response of the building. However, as shown by the Loma Prieta Community Center, good design and detailing of critical connections can result in good behavior in a strong earthquake.

Non-structural masonry which serves as an architectural element must be isolated from the structure. In order to provide the clearance required under the UBC, building drift control is important. Based on the performance of the Peninsula Office Building, current UBC requirements for separation of non-structural elements are adequate. However, architectural details which give such separation may be difficult to develop.

7.7 Recommendations for Further Research

7.7.1 Loma Prieta Community Center

1) Evaluate the response of Loma Prieta Community Center to non-coherent ground motion (out-of-phase ground motion at different points underlying the building).

2) Study the response of the west classroom wing as an appendage to the gymnasium structure.

3) Evaluate the coupled effects of vertical and horizontal ground motion on the sloped roof.

7.7.2 Peninsula Office Building

1) Evaluate the out-of-plane response of the masonry veneer.

2) Develop masonry veneer details which provide the required large clearances from structural elements.

7.7.3 2 Alhambra Building

1) Develop more convenient techniques for incorporating soil-foundation flexibility effects into special-purpose computer programs for building analysis.

2) Improve techniques for estimating the elastic stiffness and cracking strength of old masonry.

7.7.4 Hotel Woodrow

1) Improve techniques for estimating the elastic stiffness and cracking strength of old masonry.

2) Develop techniques for ascertaining the in-plane flexural, shear and axial stiffnesses of flexible floor diaphragms.

3) Continue to develop techniques for practical nonlinear analysis of three-dimensional masonry structures.

REFERENCES

1. American National Standard A58.1-1982, "Minimum Design Loads for Buildings and Other Structures," American National Standards Institute, Inc., 1982.

2. Astaneh, Abolhassan et al, "Preliminary Report on the Seismological and Engineering Aspects of the October 17, 1989 Santa Cruz (Loma Prieta) Earthquake," Report No. UCB/EERC-89/14, Earthquake Engineering Research Center, University of California at Berkeley, October 1989.

3. Blondet, J. Marcial, Mayes, R.L., Kelly, T., Villablanca F., R. and Klingner, R.E., "Performance of Engineered Masonry in the Chilean Earthquake of March 3, 1985: Implications for U.S. Design Practice," Report to the National Science Foundation (Grant Nos. ECE-8613264), PMFSEL Report No. 89-2, University of Texas at Austin and Computech Engineering Services, Inc. (Berkeley, CA), June 1989.

4. Bowles, J.E., Foundation Analysis and Design, McGraw-Hill, Inc., New York, 1977.

5. Breyer, D.E., "Design of Wood Structures,", 2nd Edition, McGraw Hill Book Co., New York, 1980.

6. Building Code Requirements for Masonry Structures (ACI 530-88/ASCE 5-88) and Specifications for Masonry Structures (ACI 530.1-88/ASCE 6-88), Reported by ACI-ASCE Committee 530, American Concrete Institute, Detroit, MI and American Society of Civil Engineers, New York, NY, 1988.

7. COMBAT: Comprehensive Building Analysis Tool, Computech Engineering Services, Inc., 2855 Telegraph Avenue, Suite 410, Berkeley, CA 94705, 1983.

8. CSMIP Strong-Motion Records from the Santa Cruz Mountains (Loma Prieta), California Earthquake of 17 October 1989, California Department of Conservation Division of Mines and Geology, Report OSMS 89-06, 1989.

9. Dobry, R. and Gazetas, G., "Dynamic Response of Arbitrarily Shaped Foundations," Journal of Geotechnical Engineering, Geotechnical Engineering Division, American Society of Civil Engineers, Volume 112, No. 2, February 1986.

10. Dreyer, R.C., Personal Communication, September, 1990.

11. Earthquake Spectra, Earthquake Engineering Research Institute, El Cerrito, CA, Supplement to Volume 6, May 1990.

12. Garcia-Delgado, V., "Soil Effects on Building Earthquake Response," Dissertation in Partial Fulfillment of the Requirements for the Degree of Doctor of Philosophy, The University of Texas at Austin, December 1986, pp. 65-95.

13. Geotechnical Investigation, Existing Residential Structure 2 Alhambra Street, San Francisco, California, Harding Lawson Associates, San Francisco, CA 94106, 1990.

14. Habibullah, A., ETABS Users Manual - Three Dimensional Analysis of Building Systems, Computers & Structures, Inc., Berkeley, CA, 1988-1990.

15. Habibullah, A. and Wilson, E.L., SAP90 Users Manual - A Series of Computer Programs for the Static and Dynamic Finite Element Analysis of Structures, Computers & Structures, Inc., Berkeley, CA, 1988-1990.

16. Klingner, R.E., "CE 383L - Advanced Reinforced Concrete Members, Lecture Notes," Fall 1989.

17. Klingner, R., "ARE W383 - Masonry, Lecture Notes," The University of Texas at Austin, Summer 1988.

18. Klingner, R., "CE 384P - Structural Dynamics, Lecture Notes," The University of Texas at Austin, Spring 1990.

19. Kreger, M.E., and Jirsa, J.O., "CE 384R - Earthquake Engineering, Lecture Notes," The University of Texas at Austin, Summer, 1990.

20. Langenbach, R., Personal Communication, March-April, 1991.

21. Loma Prieta Earthquake Reconnaissance Report, (Earthquake Spectra, Supplement to Volume 6), Earthquake Engineering Research Institute, El Cerrito, CA, May, 1990.

22. Loma Prieta Earthquake, October 17, 1989: Preliminary Reconnaissance Report, Report No. 89-03, Earthquake Engineering Research Institute, El Cerrito, CA, 1989.

23. "NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings," FEMA Report No. 95 Building Seismic Safety Council, Federal Emergency Management Agency, Washington, D.C., 1988.

24. O'Neil, Owen, Personal Communication, January, 1991.

25. Performance of Masonry Buildings in the October 17, 1989 Loma Prieta Earthquake, International Masonry Institute, Washington, D.C., February 1990.

26. Recommended Lateral Force Requirements and Commentary, Structural Engineers Association of California, San Francisco, CA, 1990.

27. Roesset, J.M., Personal Communication, November, 1990.

28. Simpkins, K., Personal Communication, September 1990.

29. SK-COMBAT: Program Description and User's Manual, SK-Computing Services, 810 Hidden Lakes, Martinez, CA 94553, 1988.

30. Soil Profile: Oakland 2-Story Recording Station, Majia, L., Woodward-Clyde Consultants, Oakland, CA 94607.

31. Specifications for Masonry Structures (ACI 530.1-88/ASCE 6-88), ACI-ASCE Committee 530, ASCE, Detroit, MI and ASCE, New York, NY, 1988.

32. The October 17, 1989 Loma Prieta Earthquake: A Quick Look Report, EQE Engineering, San Francisco, CA, 1989.

33. Thomas, R. and Klingner, R.E., "Infilled Frames," TMS Resource Document, The Masonry Society, 1991. Material adapted from Thomas, R., "Concrete Infilled Frames: The Efficient Alternative," M.S. Thesis, The University of Texas at Austin, Austin, TX, August, 1989.

34. Tissell, J.R., Shear Wall Load-Deflection Curves per ASTM E72 Test Method, American Plywood Association, Tacoma, WA, December 1982.

35. Tissell, J.R. and Elliott, J.R., Report 138 - Plywood Diaphragms, American Plywood Association, Tacoma, WA, April 1990.

36. Tissell, J.R., Report 154 - Structural Panel Shear Walls, American Plywood Association, Tacoma, WA, July 1990.

37. Uniform Building Code, 1988 Edition, International Conference of Building Officials, Whittier, CA,1988.

38. Vossbrinck, D., Personal Communication, September 1990. See Appendix A for detailed list.

APPENDIX A
BUILDING NO. 1 - LOMA PRIETA COMMUNITY CENTER

A.1 Design Criteria

This section presents information on the material properties and seismic dead loads used in the analyses. The size, location and types of materials used in construction were obtained from the original plans and specifications for the building. In addition, a copy of the original Foundation Investigation Report was available which provided information on the subsurface conditions at the site. The following documentation was obtained:

a. "Architectural Drawings A-1 thru A-25," dated August 11, 1986, and prepared by Warren B. Heid, AIA and Associates, Saratoga, CA.

b. "Structural Drawings S-1 thru S-11," dated August 13, 1986, and prepared by Vossbrinck Associates, Palo Alto, CA.

c. "Foundation Investigation Report," prepared by Peter Kaldveer and Associates, Inc., and dated February 1986.

I. Materials

A. Structural Steel

1. Structural Steel Shapes — ASTM Carbon Steel, Grade 36
   E_s = 29,000 ksi, — Poisson's Ratio = 0.30

B. Cast-in-Place Concrete — f'_c = 3,000 psi @ 28 days
   E_c = 3,122 ksi, — Poisson's Ratio = 0.20

C. Masonry

1. Masonry Slump Block Units — ASTM C90, Grade N, 6" units, f'_m = 2,000 psi
2. Mortar — ASTM 270, Type S, 2,000 psi
3. Grout — f'_c = 2,000 psi

D. Timber — Douglas Fir (Coast Region) No. 1, Moisture Content 19%

E. Soils

1. Unit Weight of Soil — 100 pcf
2. Poisson's Ratio for Soil — 0.333
3. Shear Wave Velocity, v_s — 847 ft/sec @ 6-ft. depth

II. Seismic Dead Loads (Used in lateral analyses)

Roof over Gymnasium:

Dead Load =	5/8-inch CDX Plywood Sheathing	= 2 psf
	Composition Shingles over 30# Felt	= 2 psf
	2x6 Wood Purlins @ 24" o.c.	= 2 psf
	4x10 & 6x10 Wood Purlins @ 8 ft. o.c.	= 2 psf
	Main Structural Steel Roof Trusses	= 10 psf
	Mechanical, Electrical and Plumbing	= 3 psf
		21 psf

Roof over Classrooms (west side):

Dead Load =	1/2-inch CDX Plywood Sheathing	= 1.5 psf
	Composition Shingles over 30# Felt	= 2 psf
	2x6 Wood Purlins @ 24" o.c.	= 2 psf
	5 1/8 x 12" Glued-Laminated Beams @ 8 ft. o.c.	= 2.5 psf
	Mechanical, Electrical and Plumbing	= 4 psf
	Accoustical Ceiling & Grid	= 2 psf
	Insulation	= 2 psf
		16 psf
	Interior Partitions	= 10 psf
		26 psf

Roof over Stage & Locker Rooms (east side):

Dead Load =	1/2-inch CDX Plywood Sheathing	= 1.5 psf
	Composition Shingles over 30# Felt	= 2 psf
	2x6 Wood Purlins @ 24" o.c.	= 2 psf
	5 1/8 x 12" Glued-Laminated Beams @ 8 ft. o.c.	= 2.5 psf
	Mechanical, Electrical and Plumbing	= 3 psf
	Accoustical Ceiling & Grid	= 2 psf
	Insulation	= 2 psf
		15 psf
	Interior Partitions & Additional Roof Loads over Stage	= 5 psf
		20 psf

Roof over Open Patio at West End (South wallway similar):

Dead Load =	1/2-inch CDX Plywood Sheathing	= 2 psf
	Composition Shingles over 30# Felt	= 2 psf
	2x8 Wood Purlins @ 16" o.c.	= 2.5 psf
	5 1/8 x 13 1/2" GLB @ 17.5 ft. o.c.	= 1.5 psf
		8 psf

Floor at Stage & Locker Areas (east side):

Live Load =	5 psf (toliets, sinks, benches, partitions)	
Dead Load =	3/4-inch Tongue & Groove Plywood	= 3 psf
	2x6 Wood Joists @ 16" o.c.	= 2 psf
	6x10 Wood Beams @ 4 ft. o.c.	= 4 psf
	Insulation	= 2 psf
		11 psf

Exterior Wall System:

1. West End Classrooms

Perimeter Walls:	2x6 studs @ 24" o.c.	= 1 psf
	6" RW-19 Batt Insulation	= 2 psf
	3/8" Exterior Plywood Siding	= 1 psf
	1/2" Gyp Board Sheathing	= 2 psf
	1/2" Plywood	= 1.5 psf
	1/2" Interior Gyp Board	= 2 psf
		9.5 psf

2. Gymnasium:

a. N & S Walls:	8" Grouted CMU	= 85 psf
	16" x 40" Pilasters @ 16 ft. o.c.	= 36 psf
		121 psf
b. E & W Walls:	12" Grouted CMU	= 130 psf

3. N, S & E Stage Walls: 8" Grouted CMU = 85 psf

4. Perimeter Wall at East End Lockers/Storage Area = 9.5 psf
   (Same as west end classroom area)

5. Glazing (5/8-inch) at N & S Gymnasium Walls = 8 psf

A.2 Design Calculations - Loma Prieta Community Center

Sample calculations are presented below for soil properties, effective damping, static stiffness coefficients and estimated in-plane stiffness of plywood roof diaphragms.

I. Soil Properties

Shear Wave Velocity (p. 80 Garcia-Delgado [12])

For γ_d = 100 pcf and Medium Soil

$$v_s \doteq (159 - 32.1)\ [66.666\ (z + 23.71)]^{0.25}$$

where z = 6 ft, based on an average footing size of 7.50 ft^2

$$v_s = (159 - 32.1)\ [66.666\ (6 + 23.71)]^{0.25} = 847\ \text{ft/sec}$$

II. Effective Damping

T' and T were obtained from SAP90 computer runs with and without equivalent columns to model soil flexibility.

$$\frac{T'}{T} = \frac{0.304}{0.290} = 1.048$$

Effective Damping (Ref p. 67 NEHRP [23])

where

$\beta_o = 0.015$ (Figure 6A-1 [23])

$$\beta = \beta_o + 0.05/\left(\frac{T'}{T}\right)3$$

$$\beta = 0.015 + 0.05/(1.048)^3$$

$$\beta = 0.015 + 0.43$$

$$\beta = 0.058$$

III. Static Stiffness Coefficients (Dobry & Gazetas [7])

$\underline{K_z}$:

$$K_z = K_{surface} \times I_{trench} \times I_{wall}$$

$$K_{surface} = \frac{2LG}{(1-\gamma)}\left[0.73 + 1.54\left(\frac{A_b}{4L^2}\right)^{0.75}\right] \quad 0.02 \le \frac{A_b}{4L^2} \le 1$$

$$I_{trench} = 1 + \frac{1}{21}\frac{D}{B}\left(1 + \frac{4}{3}\frac{A_b}{4L^2}\right)$$

$$I_{wall} = 1 + 0.19\left(\frac{A_s}{A_b}\right)^{2/3}$$

where

D = embedment depth

A_s = total embedded side area

$\underline{K_y}$: (Short Direction)

$$K_y = S_y\frac{(2LG)}{2-\gamma}$$

$$S_y = 2.24 \text{ for } \frac{A_b}{4L^2} \le 0.175$$

$$S_y = 4.5\left(\frac{A_b}{4L^2}\right)^{0.4} \text{ for } \frac{A_b}{4L^2} > 0.175$$

$\underline{K_x}$: (Long Direction)

$$K_x = S_x\frac{2LG}{(2-\gamma)}$$

$$S_x = S_y - \frac{3}{8}\left(1-\frac{A_b}{4L^2}\right)$$

$\underline{K_{\theta x}}$: (Short Direction)

$$K_{\theta x} = S_{rx} \frac{G}{1-\gamma} I_{xx}^{0.75}$$

$$S_{rx} = 2.54 \text{ for } B/L < 0.4$$

$$S_{rx} = 3.2(B/L) \text{ for } 0.4 \leq B/L \leq 1$$

$\underline{K_{\theta y}}$: (Long Direction)

$$K_{\theta y} = S_{ry} \frac{G}{1-\gamma} I_{yy}^{0.75}$$

$$S_{ry} = 3.2 \text{ for } 0.2 \leq B/L \leq 1$$

Refer to Appendix B for sample calculation of static stiffness coefficients.

IV. Estimated In-Plane Stiffness of Plywood Roof Diaphragms

The in-plane stiffness of the plywood roof diaphragms was estimated using diaphragm deflection formulas presented in APA <u>Publication No. 138</u>, "Plywood Diaphragms" [35]:

$$\Delta = \frac{5VL^3}{8EA_b} + \frac{VL}{4G_t} + 0.188\ Le_n + \frac{\sum(\Delta_c X)}{2_b}$$

where

V	=	shear (plf)
L	=	diaphragm length (ft)
b	=	diaphragm width (ft)
A	=	area of chord cross section (in^2)
E	=	elastic modulus of chords (psi)
G	=	shearing modulus of the webs (psi)
t	=	effective plywood thickness of shear (in)
e_n	=	nail deformation (in) from Appendix Table B-4 (Ref. 20) at calculated load per nail on perimeter of interior panels, based on shear per foot divided by number of nails per foot. If the nailing is not the same in both directions, use the greater spacing for calculations.
$\Sigma(\Delta_c X)$	=	sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by its distance (ft) to the nearest support.

The stiffness was computed as the ratio (V/Δ).

A.3 Computer Results

I. Eigenvalues and Frequencies

MODE NUMBER	EIGENVALUE (rad/sec)^2	CIRCULAR FREQ (RAD/SEC)	FREQUENCY (CYCLES/SEC)	PERIOD (SEC)
1	.426388E+03	.206492E+02	3.286418	.304283
2	.509032E+03	.225617E+02	3.590814	.278488
3	.580892E+03	.241017E+02	3.835905	.260695
4	.813975E+03	.285302E+02	4.540729	.220229
5	.898255E+03	.299709E+02	4.770017	.209643
6	.904423E+03	.300736E+02	4.786365	.208927
7	.108073E+04	.328745E+02	5.232132	.191127
8	.115424E+04	.339741E+02	5.407151	.184940
9	.122129E+04	.349470E+02	5.561992	.179792
10	.122990E+04	.350699E+02	5.581545	.179162
11	.136214E+04	.369071E+02	5.873951	.170243
12	.149849E+04	.387104E+02	6.160947	.162313
13	.164846E+04	.406013E+02	6.461896	.154753
14	.166772E+04	.408378E+02	6.499532	.153857
15	.184109E+04	.429079E+02	6.829007	.146434
16	.185975E+04	.431248E+02	6.863523	.145698
17	.195870E+04	.442572E+02	7.043759	.141970
18	.209790E+04	.458028E+02	7.289740	.137179
19	.220158E+04	.469210E+02	7.467711	.133910
20	.235262E+04	.485038E+02	7.719625	.129540
21	.252879E+04	.502870E+02	8.003433	.124946
22	.262214E+04	.512068E+02	8.149822	.122702
23	.275778E+04	.525146E+02	8.357952	.119647
24	.283104E+04	.532075E+02	8.468241	.118088
25	.289113E+04	.537692E+02	8.557642	.116855
26	.317681E+04	.563632E+02	8.970483	.111477
27	.370539E+04	.608719E+02	9.688062	.103220
28	.387625E+04	.622596E+02	9.908916	.100919
29	.395091E+04	.628562E+02	10.003880	.099961
30	.478240E+04	.691549E+02	11.006341	.090857
31	.489058E+04	.699327E+02	11.130133	.089846
32	.501243E+04	.707985E+02	11.267938	.088747
33	.697448E+04	.835133E+02	13.291563	.075236
34	.722859E+04	.850211E+02	13.531534	.073901
35	.755004E+04	.868910E+02	13.829126	.072311
36	.108583E+05	.104203E+03	16.584455	.060297
37	.159366E+05	.126240E+03	20.091745	.049772
38	.171840E+05	.131088E+03	20.863282	.047931
39	.217862E+05	.147601E+03	23.491492	.042569
40	.242558E+05	.155743E+03	24.787209	.040343
41	.264786E+05	.162722E+03	25.898069	.038613
42	.365811E+05	.191262E+03	30.440281	.032851
43	.503368E+05	.224359E+03	35.707782	.028005
44	.574203E+05	.239625E+03	38.137552	.026221
45	.116715E+06	.341636E+03	54.373008	.018391

II. Participating Mass (percent)

MODE	X-DIR	Y-DIR	Z-DIR	X-SUM	Y-SUM	Z-SUM
1	.001	35.144	.000	.001	35.144	.000
2	.043	8.721	.000	.044	43.865	.000
3	30.755	.027	.000	30.799	43.892	.000
4	1.194	.015	.000	31.994	43.908	.000
5	1.619	.010	.000	33.613	43.918	.000
6	.003	.582	.000	33.616	44.500	.000
7	.322	.022	.000	33.938	44.522	.000
8	.004	.302	.000	33.942	44.824	.000
9	.112	.049	.000	34.054	44.873	.001
10	.307	.813	.000	34.361	45.686	.001
11	.556	.295	.000	34.916	45.982	.001
12	.003	2.806	.000	34.920	48.788	.001
13	.424	.001	.000	35.344	48.789	.001
14	.001	.419	.000	35.345	49.207	.001
15	.273	.000	.000	35.618	49.207	.001
16	.000	.182	.000	35.618	49.389	.001
17	.089	.011	.000	35.707	49.401	.001
18	.277	.000	.000	35.984	49.401	.001
19	.050	.003	.000	36.034	49.403	.001
20	.006	.044	.000	36.040	49.447	.001
21	.003	.535	.000	36.043	49.982	.001
22	.240	.711	.001	36.283	50.693	.002
23	.405	.076	.000	36.688	50.769	.002
24	.263	.870	.001	36.951	51.639	.003
25	.217	.010	.000	37.168	51.649	.003
26	.299	.244	.000	37.467	51.893	.003
27	.003	.092	.001	37.470	51.985	.004
28	.235	.705	.000	37.705	52.690	.004
29	.080	1.968	.000	37.785	54.658	.004
30	.024	.180	.000	37.809	54.838	.004
31	.067	1.823	.000	37.876	56.661	.004
32	.416	.361	.000	38.293	57.022	.004
33	.012	3.523	.000	38.304	60.546	.004
34	26.973	.020	.073	65.278	60.566	.078
35	1.220	.051	.000	66.498	60.617	.078
36	.001	1.738	.000	66.499	62.355	.078
37	1.200	.002	.024	67.699	62.357	.102
38	.103	.060	64.698	67.801	62.417	64.799
39	12.004	.041	.754	79.806	62.458	65.554
40	.251	11.185	.000	80.057	73.643	65.554
41	9.859	.143	.242	89.916	73.786	65.797
42	7.932	.163	.409	97.849	73.948	66.205
43	.628	.182	1.214	98.476	74.131	67.419
44	.010	21.871	.056	98.486	96.002	67.475
45	.040	.118	27.017	98.526	96.120	94.492

III. Maximum Lateral Displacements

Location	Max. Deflection X-Direction (E-W)	Max. Deflection Y-Direction (N-S)
West Classroom Wing	0.67 inches	1.32 inches
Gymnasium	1.47	2.43
Stage/Locker Rooms	0.67	0.11

A.4 SAP90 Data File

LOMA62 - LOMA PRIETA COMMUNITY CENTER
SYSTEM
Z=45 L=1 : NUMBER OF MODES USED IN RITZ ANALYSIS
JOINTS : SPECIFYING JOINT LOCATIONS

1	X=0	Y=0	Z=0	S=12
2	X=0	Y=6.5	Z=0	
3	X=0	Y=24.83	Z=0	
4	X=0	Y=35.83	Z=0	
5	X=0	Y=49.67	Z=0	
6	X=0	Y=55.67	Z=0	
7	X=0	Y=64.67	Z=0	
8	X=0	Y=74.51	Z=0	
9	X=0	Y=81.09	Z=0	
10	X=0	Y=90.09	Z=0	
11	X=0	Y=99.34	Z=0	
12	X=0	Y=0	Z=11	
13	X=0	Y=6.5	Z=12.83	
14	X=0	Y=24.83	Z=18	
15	X=45	Y=0	Z=0	
16	X=45	Y=24.83	Z=0	
17	X=45	Y=49.67	Z=0	
18	X=45	Y=74.51	Z=0	
19	X=45	Y=99.34	Z=0	
20	X=45	Y=0	Z=11	
21	X=45	Y=24.83	Z=18	
22	X=45	Y=49.67	Z=11	
23	X=45	Y=74.51	Z=18	
24	X=45	Y=99.34	Z=11	
25	X=63	Y=0	Z=0	
	:Joint Nos. 26, 28, 30, 32 & 37 not used			
27	X=63	Y=24.83	Z=0	
29	X=63	Y=49.67	Z=0	
31	X=63	Y=74.51	Z=0	
33	X=63	Y=99.34	Z=0	
34	X=63	Y=0	Z=11	
35	X=63	Y=24.83	Z=11	
36	X=63	Y=24.83	Z=18	
38	X=63	Y=49.67	Z=11	
39	X=63	Y=49.67	Z=18	
40	X=63	Y=74.51	Z=11	
41	X=63	Y=74.51	Z=18	
42	X=63	Y=99.34	Z=11	
43	X=63	Y=0	Z=20	
44	X=63	Y=22	Z=20	
45	X=63	Y=34.67	Z=20	
46	X=63	Y=64.67	Z=20	
47	X=63	Y=77.34	Z=20	
48	X=63	Y=99.34	Z=20	
49	X=78.5	Y=0	Z=0	
50	X=78.5	Y=99.34	Z=0	
51	X=78.5	Y=0	Z=11	
52	X=78.5	Y=99.34	Z=11	
53	X=78.5	Y=0	Z=18	
54	X=78.5	Y=99.3	Z=18	
55	X=78.5	Y=0	Z=25	
56	X=78.5	Y=22	Z=25	
57	X=78.5	Y=34.67	Z=25	
58	X=78.5	Y=64.67	Z=25	
59	X=78.5	Y=77.34	Z=25	
60	X=78.5	Y=99.34	Z=25	
61	X=94.5	Y=0	Z=0	
62	X=94.5	Y=99.34	Z=0	
63	X=94.5	Y=0	Z=11	
64	X=94.5	Y=99.34	Z=11	
65	X=94.5	Y=0	Z=18	
66	X=94.5	Y=99.34	Z=18	
67	X=94.5	Y=0	Z=30	
68	X=94.5	Y=22	Z=30	
69	X=94.5	Y=34.67	Z=30	
70	X=94.5	Y=64.67	Z=30	
71	X=94.5	Y=77.34	Z=30	
72	X=94.5	Y=99.34	Z=30	
73	X=110.5	Y=0	Z=0	
74	X=110.5	Y=99.34	Z=0	
75	X=110.5	Y=0	Z=11	
76	X=110.5	Y=99.34	Z=11	
77	X=110.5	Y=0	Z=35	
78	X=110.5	Y=22	Z=35	
79	X=110.5	Y=34.67	Z=35	
80	X=110.5	Y=64.67	Z=35	
81	X=110.5	Y=77.34	Z=35	
82	X=110.5	Y=99.34	Z=35	
83	X=126.5	Y=0	Z=0	
84	X=126.5	Y=99.34	Z=0	
85	X=126.5	Y=0	Z=11	
86	X=126.5	Y=99.34	Z=11	
87	X=126.5	Y=0	Z=18	
88	X=126.5	Y=99.34	Z=18	
89	X=126.5	Y=0	Z=30	
90	X=126.5	Y=22	Z=30	
91	X=126.5	Y=34.67	Z=30	
92	X=126.5	Y=64.67	Z=30	
93	X=126.5	Y=77.34	Z=30	
94	X=126.5	Y=99.34	Z=30	
95	X=142.5	Y=0	Z=0	
96	X=142.5	Y=99.34	Z=0	
97	X=142.5	Y=0	Z=11	
98	X=142.5	Y=99.34	Z=11	
99	X=142.5	Y=0	Z=18	
100	X=142.5	Y=99.34	Z=18	

101	X=142.5	Y=0	Z=25
102	X=142.5	Y=22	Z=25
103	X=142.5	Y=34.67	Z=25
104	X=142.5	Y=64.67	Z=25
105	X=142.5	Y=77.34	Z=25
106	X=142.5	Y=99.34	Z=25
107	X=158	Y=0	Z=0
108	X=158	Y=22	Z=0
109	X=158	Y=34.67	Z=0
110	X=158	Y=64.67	Z=0
111	X=158	Y=77.34	Z=0
112	X=158	Y=99.34	Z=0
113	X=158	Y=0	Z=11
114	X=158	Y=22	Z=11
115	X=158	Y=34.67	Z=11
116	X=158	Y=64.67	Z=11
117	X=158	Y=77.34	Z=11
118	X=158	Y=99.34	Z=11
119	X=158	Y=0	Z=18
120	X=158	Y=99.34	Z=18
121	X=158	Y=0	Z=20
122	X=158	Y=22	Z=20
123	X=158	Y=34.67	Z=20
124	X=158	Y=64.67	Z=20
125	X=158	Y=77.34	Z=20
126	X=158	Y=99.34	Z=20
127	X=187	Y=22	Z=0
128	X=187	Y=77.34	Z=0
129	X=187	Y=22	Z=11
130	X=187	Y=77.34	Z=11
131	X=187	Y=22	Z=20
132	X=187	Y=77.34	Z=20
133	X=63	Y=0	Z=18
134	X=63	Y=99.34	Z=18
135	X=158	Y=22	Z=18
136	X=158	Y=34.67	Z=18
137	X=158	Y=64.67	Z=18
138	X=158	Y=77.34	Z=18
139	X=10	Y=0	Z=11
140	X=10	Y=24.83	Z=18
141	X=10	Y=49.67	Z=11
142	X=10	Y=74.51	Z=18
143	X=10	Y=99.34	Z=11
144	X=10	Y=0	Z=0
145	X=10	Y=49.67	Z=0
146	X=10	Y=99.34	Z=0
	Joints 147 thru 151 not used		
152	X=0	Y=35.83	Z=14.90
153	X=0	Y=49.67	Z=11
154	X=0	Y=55.67	Z=12.69
155	X=0	Y=64.67	Z=13.73
156	X=0	Y=74.51	Z=18
157	X=0	Y=81.09	Z=16.14
158	X=0	Y=90.09	Z=13.61
159	X=0	Y=99.34	Z=11
	: Joint 160 not used		
161	X=0	Y=0	Z=2.5
162	X=0	Y=0	Z=9.5
163	X=0	Y=6.5	Z=2.5
164	X=0	Y=6.5	Z=9.5
165	X=0	Y=24.83	Z=2.5
166	X=0	Y=24.83	Z=9.5
167	X=0	Y=35.83	Z=2.5
168	X=0	Y=35.83	Z=9.5
169	X=0	Y=49.67	Z=9.5
170	X=0	Y=55.67	Z=2.5
171	X=0	Y=55.67	Z=9.5
172	X=0	Y=64.67	Z=2.5
173	X=0	Y=64.67	Z=9.5
174	X=0	Y=74.51	Z=2.5
175	X=0	Y=74.51	Z=9.5
176	X=0	Y=81.09	Z=2.5
177	X=0	Y=81.09	Z=9.5
178	X=0	Y=90.09	Z=2.5
179	X=0	Y=90.09	Z=9.5 :
Joints 180 & 181 not used			
182	X=0	Y=99.34	Z=2.5
183	X=0	Y=99.34	Z=9.5
184	X=-17.5	Y=-8.0	Z=0
185	X=-17.5	Y=0	Z=0
186	X=-17.5	Y=24.83	Z=0
187	X=-17.5	Y=35.83	Z=0
188	X=-17.5	Y=49.67	Z=0
189	X=-17.5	Y=64.67	Z=0
190	X=-17.5	Y=74.51	Z=0
191	X=-17.5	Y=99.34	Z=0
192	X=-17.5	Y=-8.0	Z=11
193	X=-17.5	Y=0	Z=11
194	X=-17.5	Y=6.5	Z=12.83
195	X=-17.5	Y=24.83	Z=18
196	X=-17.5	Y=35.83	Z=14.90
197	X=-17.5	Y=49.67	Z=11
198	X=-17.5	Y=55.67	Z=12.69
199	X=-17.5	Y=64.67	Z=13.73
200	X=-17.5	Y=74.51	Z=18
201	X=-17.5	Y=81.09	Z=16.14
202	X=-17.5	Y=90.09	Z=13.61
203	X=-17.5	Y=99.34	Z=11
204	X=0	Y=-8	Z=0
205	X=10	Y=-8	Z=0
206	X=45	Y=-8	Z=0
207	X=63	Y=-8	Z=0
208	X=78.5	Y=-8	Z=0
209	X=94.5	Y=-8	Z=0
210	X=110.5	Y=-8	Z=0
211	X=126.5	Y=-8	Z=0
212	X=142.5	Y=-8	Z=0
213	X=158	Y=-8	Z=0
214	X=170	Y=-8	Z=0

215	X=170	Y=0	Z=0
216	X=187	Y=0	Z=0
217	X=199.5	Y=0	Z=0
218	X=199.5	Y=22	Z=0
219	X=199.5	Y=49.67	Z=0
220	X=199.5	Y=77.34	Z=0
221	X=199.5	Y=99.34	Z=0
222	X=187	Y=49.67	Z=0
223	X=187	Y=99.34	Z=0
224	X=170	Y=99.34	Z=0
225	X=170	Y=22	Z=0
226	X=170	Y=77.34	Z=0
	Joints 227, 228 & 229 not used		
230	X=0	Y=-8	Z=11
231	X=10	Y=-8	Z=11
232	X=45	Y=-8	Z=11
233	X=63	Y=-8	Z=11
234	X=78.5	Y=-8	Z=11
235	X=94.5	Y=-8	Z=11
236	X=110.5	Y=-8	Z=11
237	X=126.5	Y=-8	Z=11
238	X=142.5	Y=-8	Z=11
239	X=158	Y=-8	Z=11
240	X=170	Y=-8	Z=11
241	X=170	Y=0	Z=11
242	X=187	Y=0	Z=11
243	X=199.5	Y=0	Z=11
244	X=199.5	Y=22	Z=18
245	X=199.5	Y=49.67	Z=11
246	X=199.5	Y=77.34	Z=18
247	X=199.5	Y=99.34	Z=11
248	X=187	Y=99.34	Z=11
249	X=170	Y=99.34	Z=11
250	X=170	Y=22	Z=18
251	X=187	Y=22	Z=18
252	X=187	Y=49.67	Z=11
253	X=170	Y=77.34	Z=18
254	X=187	Y=77.34	Z=18
255	X=187	Y=49.67	Z=18
256	X=187	Y=49.67	Z=20
257	X=170	Y=22	Z=11
258	X=170	Y=22	Z=20
259	X=170	Y=77.34	Z=11
260	X=170	Y=77.34	Z=20
	: Joint 261 not used		
262	X=170	Y=34.67	Z=20
263	X=170	Y=64.67	Z=20
	: Joints 264 & 265 not used		
266	X=63	Y=0	Z=14.5
267	X=70.75	Y=0	Z=11
268	X=70.75	Y=0	Z=14.5
269	X=70.75	Y=0	Z=18
270	X=78.5	Y=0	Z=14.5
271	X=86.5	Y=0	Z=11
272	X=86.5	Y=0	Z=14.5
273	X=86.5	Y=0	Z=18
274	X=94.5	Y=0	Z=14.5

:Blank Line

RESTRAINTS

2	2	1	R=1,1,1,1,1,1
4	4	1	R=1,1,1,1,1,1
6	7	1	R=1,1,1,1,1,1
9	10	1	R=1,1,1,1,1,1
27	27	1	R=1,1,1,1,1,1
29	29	1	R=1,1,1,1,1,1
31	31	1	R=1,1,1,1,1,1
144	146	1	R=1,1,1,1,1,1
184	191	1	R=1,1,1,1,1,1
204	215	1	R=1,1,1,1,1,1
224	226	1	R=1,1,1,1,1,1
1	11	10	R=0,0,0,0,0,1
3	5	2	R=0,0,0,0,0,1
8	8	1	R=0,0,0,0,0,1
15	19	2	R=0,0,0,0,0,1
16	18	2	R=1,1,1,1,1,1
25	33	8	R=0,0,0,0,0,1
49	50	1	R=0,0,0,0,0,1
61	62	1	R=0,0,0,0,0,1
73	74	1	R=0,0,0,0,0,1
83	84	1	R=0,0,0,0,0,1
95	96	1	R=0,0,0,0,0,1
107	112	5	R=0,0,0,0,0,1
108	111	3	R=0,0,0,0,0,1
109	110	1	R=0,0,0,0,0,1
216	223	7	R=0,0,0,0,0,1
127	128	1	R=0,0,0,0,0,1
222	222	1	R=0,0,0,0,0,1
217	221	4	R=0,0,0,0,0,1
218	220	2	R=0,0,0,0,0,1
219	219	1	R=0,0,0,0,0,1

: Blank Line

SPRINGS

1	11	10	K=3634,3634,6332,7581313,7581313,0
3	5	2	K=3057,3057,5551,4515646,4515646,0
8	8	1	K=3057,3057,5551,4515646,4515646,0
15	19	2	K=3433,3433,6061,6394434,6394434,0
25	33	8	K=2706,2706,5074,3132649,3132649,0
49	50	1	K=3784,3784,6535,8561922,8561922,0
61	62	1	K=3784,3784,6535,8561922,8561922,0
73	74	1	K=3784,3784,6535,8561922,8561922,0
83	84	1	K=3784,3784,6535,8561922,8561922,0
95	96	1	K=3784,3784,6535,8561922,8561922,0
107	112	5	K=4210,4210,7110,11791478,11791478,0
108	111	3	K=3609,3609,6298,7425537,7425537,0
109	110	1	K=2180,2180,4355,1637564,1637564,0
216	223	7	K=2807,2807,5210,3493771,3493771,0
127	128	1	K=4611,4611,7649,15491515,15491515,0
222	222	1	K=4561,4561,7581,14991828,14991828,0
217	221	4	K=2556,2556,4869,2639009,2639009,0
218	220	2	K=3057,3057,5551,4515646,4515646,0

```
219 219 1    K=3233,3233,5789,5338381,5338831,0
                   :  Blank Line
MASSES
34   42   8     M=0,.014915,0,0,0,0
         : N & S Gymnasium Walls
133 134 1     M=0,.008493,0,0,0,0
43   48   5     M=0,.002693,0,0,0,0
51   52   1     M=0,.030310,0,0,0,0
53   54   1     M=0,.035360,0,0,0,0
55   60   5     M=0,.023580,0,0,0,0
63   64   1     M=0,.024500,0,0,0,0
65   66   1     M=0,.024380,0,0,0,0
67   72   5     M=0,.018390,0,0,0,0
75   76   1     M=0,.018820,0,0,0,0
85   86   1     M=0,.024500,0,0,0,0
87   88   1     M=0,.024380,0,0,0,0
89   94   5     M=0,.018390,0,0,0,0
97   98   1     M=0,.030310,0,0,0,0
99   100 1     M=0,.035360,0,0,0,0
101 106 5     M=0,.023580,0,0,0,0
113 118 5     M=0,.014915,0,0,0,0
119 120 1     M=0,.008493,0,0,0,0
121 126 5     M=0,.002693,0,0,0,0
115 116 1     M=.018664,0,0,0,0,0
         : East Gymnasium Wall
136 137 1     M=.011406,0,0,0,0,0
113 118 5     M=.034320,0,0,0,0,0
119 120 1     M=.020970,0,0,0,0,0
114 117 3     M=.052980,0,0,0,0,0
135 138 3     M=.032377,0,0,0,0,0
35   38   3     M=.075200,0,0,0,0,0
         : West Gymnasium Wall
40   40   1     M=.075200,0,0,0,0,0
36   39   3     M=.045960,0,0,0,0,0
41   41   1     M=.045960,0,0,0,0,0
34   42   8     M=.037600,0,0,0,0,0
133 134 1     M=.022980,0,0,0,0,0
250 253 3     M=0,.017054,0,0,0,0
         : N & S Stage Walls
257 259 2     M=0,.027906,0,0,0,0
114 117 3     M=0,.011547,0,0,0,0
135 138 3     M=0,.007057,0,0,0,0
129 130 1     M=0,.016360,0,0,0,0
251 254 3     M=0,.009997,0,0,0,0
129 130 1     M=.026309,0,0,0,0,0
         : East Stage Wall
252 252 1     M=.052598,0,0,0,0,0
251 254 3     M=.016078,0,0,0,0,0
255 255 1     M=.032140,0,0,0,0,0
51   52   1     M=.128,0,0,0,0,0
         : In-Plane Nodal Masses\ N&S Gym Walls
97   98   1     M=.128,0,0,0,0,0
53   54   1     M=.064,0,0,0,0,0
99   100 1     M=.064,0,0,0,0,0
114 117 3     M=0,.1138,0,0,0,0
         : In-Plane Nodal Masses\ E&W Gym Walls
135 138 3     M=0,.0569,0,0,0,0
35   40   5     M=0,.1615,0,0,0,0
36   41   5     M=0,.0808,0,0,0,0
257 259 2     M=.05995,0,0,0,0,0
         : In-Plane Nodal Masses\ N&S Stage Walls
250 253 3     M=.02997,0,0,0,0,0
129 130 1     M=.05651,0,0,0,0,0
         : In-Plane Nodal Masses\ East Stage Wall
251 254 3     M=.02825,0,0,0,0,0
195 197 2     M=.01670,.01670,0,0,0,0
         : Section #1 Nodal Masses (West Wing)
200 200 1     M=.01670,.01670,0,0,0,0
14   14   1     M=.04123,.04123,0,0,0,0
153 153 1     M=.03355,.03355,0,0,0,0
142 156 14   M=.025880,.025880,0,0,0
140 140 1     M=.04123,.04123,0,0,0,0
141 141 1     M=.03355,.03355,0,0,0,0
21   23   1     M=.06019,.06019,0,0,0,0
36   38   2     M=.03009,.03009,0,0,0,0
41   41   1     M=.03009,.03009,0,0,0,0
56   59   1     M=.026933,.026933,0,0,0,0
         : Section #2 Nodal Roof Masses (Gym)
68   71   1     M=.026933,.026933,0,0,0,0
78   81   1     M=.026933,.026933,0,0,0,0
90   93   1     M=.026933,.026933,0,0,0,0
102 105 1     M=.026933,.026933,0,0,0,0
122 125 3     M=.020689,.020689,0,0,0,0
         : Section #3 Nodal Masses (East Wing)
131 132 1     M=.020689,.020689,0,0,0,0
256 256 1     M=.040625,.040625,0,0,0,0
113 118 5     M=.014896,.014896,0,0,0,0
243 247 4     M=.014896,.014896,0,0,0,0
135 138 3     M=.011887,.011887,0,0,0,0
252 252 1     M=.017905,.017905,0,0,0,0
244 246 2     M=.018056,.018056,0,0,0,0
245 245 1     M=.024074,.024074,0,0,0,0
                   : Blank Line
CONSTRAINTS
21   24   1     C=0,0,20,0,0,0
140 143 1     C=0,0,139,0,0,0
262 263 1     C=0,0,258,0,0,0
44   48   1     C=0,43,0,0,0,0
56   60   1     C=0,55,0,0,0,0
68   72   1     C=0,67,0,0,0,0
78   82   1     C=0,77,0,0,0,0
90   94   1     C=0,89,0,0,0,0
102 106 1     C=0,101,0,0,0,0
122 126 1     C=0,121,0,0,0,0
78   102 12   C=56,0,0,0,0,0
68   68   1     C=56,0,0,0,0,0
79   103 12   C=57,0,0,0,0,0
69   69   1     C=57,0,0,0,0,0
80   104 12   C=58,0,0,0,0,0
81   105 12   C=59,0,0,0,0,0
```

```
71  71  1   C=59,0,0,0,0,0
            : Blank Line

FRAME
NM=8
1   A=27.6    J=.01      I=16283,109    AS=12.52,13.22    \
    E=29000   G=37638    W=.009917M     M=0                 : Steel Trusses
2   A=619     J=3221     I=81011,12596  AS=516,516        \
    E=2460    G=984      W=.0502        M=.00013            : CMU Pilasters
3   A=31.6    J=141.0    I=83.4,83.4    AS=26.4,26.4      \
    E=1800    G=720      W=.000623      M=0                 : 6x6 Wood Posts
4   A=50.3    J=402      I=201,201      AS=45.2,45.2      \
    E=3122    G=1301     W=.00437       M=0                 : 8"-dia Conc Cols
5   A=113.1   J=2036.0   I=1018,1018    AS=101.8,101.8    \
    E=3122    G=1301     W=.008837      M=0                 : 12"-dia Conc Cols
6   A=27.6    J=83.2     I=134,30.3     AS=23,23            \
    E=1800    G=720      W=.000544      M=0                 : 4x8 Wood Beam
7   A=619     J=3221     I=81011,12596  AS=516,516        \
    E=2460    G=984      W=.055087      M=.0001426          : CMU Pilasters @ J & L
8   A=619     J=3221     I=81011,12596  AS=516,516        \
    E=2460    G=984      W=.05998       M=.0001553          : CMU Pilaster @ K

1    55    56    M=1,1,1   LP=3,0   LR=1,0,0,0,0,0
2    56    57    M=1,1,1   LP=3,0   LR=0,0,0,0,0,0
3    57    58    M=1,1,1   LP=3,0   LR=0,0,0,0,0,0
4    58    59    M=1,1,1   LP=3,0   LR=0,0,0,0,0,0
5    59    60    M=1,1,1   LP=3,0   LR=0,1,0,0,0,0
6    67    68    M=1,1,1   LP=3,0   LR=1,0,0,0,0,0
7    68    69    M=1,1,1   LP=3,0   0,0,0,0,0,0
8    69    70    M=1,1,1   LP=3,0   LR=0,0,0,0,0,0
9    70    71    M=1,1,1   LP=3,0   R=0,0,0,0,0,0
10   71    72    M=1,1,1   LP=3,0   0,1,0,0,0,0
11   77    78    M=1,1,1   LP=3,0   LR=1,0,0,0,0,0
12   78    79    M=1,1,1   LP=3,0   LR=0,0,0,0,0,0
13   79    80    M=1,1,1   LP=3,0   LR=0,0,0,0,0,0
14   80    81    M=1,1,1   LP=3,0   LR=0,0,0,0,0,0
15   81    82    M=1,1,1   LP=3,0   LR=0,1,0,0,0,0
16   89    90    M=1,1,1   LP=3,0   LR=1,0,0,0,0,0
17   90    91    M=1,1,1   LP=3,0   LR=0,0,0,0,0,0
18   91    92    M=1,1,1   LP=3,0   LR=0,0,0,0,0,0
19   92    93    M=1,1,1   LP=3,0   LR=0,0,0,0,0,0
20   93    94    M=1,1,1   LP=3,0   LR=0,1,0,0,0,0
21   101   102   M=1,1,1   LP=3,0   LR=1,0,0,0,0,0
22   102   103   M=1,1,1   LP=3,0   LR=0,0,0,0,0,0
23   103   104   M=1,1,1   LP=3,0   LR=0,0,0,0,0,0
24   104   105   M=1,1,1   LP=3,0   LR=0,0,0,0,0,0
25   105   106   M=1,1,1   LP=3,0   LR=0,1,0,0,0,0    : 26 thru 30 not used
31   25    34    M=2,2,1   LP=3,0   LR=0,0,0,0,0,0
32   34    133   M=2,2,1   LP=3,0   LR=0,0,0,0,0,0
33   133   43    M=2,2,1   LP=3,0   LR=0,0,0,0,0,0
34   49    51    M=2,2,1   LP=3,0   LR=0,0,0,0,0,0
35   51    53    M=2,2,1   LP=3,0   LR=0,0,0,0,0,0
36   53    55    M=2,2,1   LP=3,0   LR=0,0,0,0,0,0
37   61    63    M=2,2,1   LP=3,0   LR=0,0,0,0,0,0
38   63    65    M=7,7,1   LP=3,0   LR=0,0,0,0,0,0
```

39	65	67	M=7,7,1	LP=3,0	LR=0,0,0,0,0,0	
40	73	75	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
41	75	77	M=8,8,1	LP=3,0	LR=0,0,0,0,0,0	
42	83	85	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
43	85	87	M=7,7,1	LP=3,0	LR=0,0,0,0,0,0	
44	87	89	M=7,7,1	LP=3,0	LR=0,0,0,0,0,0	
45	95	97	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
46	97	99	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
47	99	101	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
48	107	113	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
49	113	119	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
50	119	121	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
51	33	42	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
52	42	134	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
53	134	48	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
54	50	52	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
55	52	54	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
56	54	60	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
57	62	64	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
58	64	66	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
59	66	72	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
60	74	76	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
61	76	82	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
62	84	86	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
63	86	88	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
64	88	94	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
65	96	98	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
66	98	100	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
67	100	106	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
68	112	118	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
69	118	120	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
70	120	126	M=2,2,1	LP=3,0	LR=0,0,0,0,0,0	
71	16	21	M=3,3,1	LP=3,0	LR=0,0,0,0,0,0	
72	18	23	M=3,3,1	LP=3,0	LR=0,0,0,0,0,0	: Element 73 not used
74	185	193	M=5,5,1	LP=3,0	LR=0,0,0,0,0,0	
75	186	195	M=5,5,1	LP=3,0	LR=0,0,0,0,0,0	
76	187	196	M=5,5,1	LP=3,0	LR=0,0,0,0,0,0	
77	188	197	M=5,5,1	LP=3,0	LR=0,0,0,0,0,0	
78	189	199	M=5,5,1	LP=3,0	LR=0,0,0,0,0,0	
79	190	200	M=5,5,1	LP=3,0	LR=0,0,0,0,0,0	
80	191	203	M=5,5,1	LP=3,0	LR=0,0,0,0,0,0	
81	184	192	M=4,4,1	LP=3,0	LR=0,0,0,0,0,0	
82	204	230	M=4,4,1	LP=3,0	LR=0,0,0,0,0,0	
83	205	231	M=4,4,1	LP=3,0	LR=0,0,0,0,0,0	
84	206	232	M=4,4,1	LP=3,0	LR=0,0,0,0,0,0	
85	207	233	M=4,4,1	LP=3,0	LR=0,0,0,0,0,0	
86	208	234	M=4,4,1	LP=3,0	LR=0,0,0,0,0,0	
87	209	235	M=4,4,1	LP=3,0	LR=0,0,0,0,0,0	
88	210	236	M=4,4,1	LP=3,0	LR=0,0,0,0,0,0	
89	211	237	M=4,4,1	LP=3,0	LR=0,0,0,0,0,0	
90	212	238	M=4,4,1	LP=3,0	LR=0,0,0,0,0,0	
91	213	239	M=4,4,1	LP=3,0	LR=0,0,0,0,0,0	
92	214	240	M=4,4,1	LP=3,0	LR=0,0,0,0,0,0	
93	15	20	M=3,3,1	LP=3,0	LR=0,0,0,0,0,0	
94	17	22	M=3,3,1	LP=3,0	LR=0,0,0,0,0,0	

95	19	24	M=3,3,1	LP=3,0	LR=0,0,0,0,0,0
96	193	194	M=6,6,1	LP=3,0	LR=1,0,0,0,0,0
97	194	195	M=6,6,1	LP=3,0	LR=0,1,0,0,0,0
98	195	196	M=6,6,1	LP=3,0	LR=1,1,0,0,0,0
99	196	197	M=6,6,1	LP=3,0	LR=1,1,0,0,0,0
100	197	198	M=6,6,1	LP=3,0	LR=1,0,0,0,0,0
101	198	199	M=6,6,1	LP=3,0	LR=0,1,0,0,0,0
102	199	200	M=6,6,1	LP=3,0	LR=1,1,0,0,0,0
103	200	201	M=6,6,1	LP=3,0	LR=1,0,0,0,0,0
104	201	202	M=6,6,1	LP=3,0	LR=0,0,0,0,0,0
105	202	203	M=6,6,1	LP=3,0	LR=0,1,0,0,0,0
106	6	170	M=3,3,1	LP=3,0	LR=0,0,0,0,0,0
107	170	171	M=3,3,1	LP=3,0	LR=0,0,0,0,0,0
108	171	154	M=3,3,1	LP=3,0	LR=0,0,0,0,0,0
109	9	176	M=3,3,1	LP=3,0	LR=0,0,0,0,0,0
110	176	177	M=3,3,1	LP=3,0	LR=0,0,0,0,0,0
111	177	157	M=3,3,1	LP=3,0	LR=0,0,0,0,0,0

SHELL
NM=8

1	E=2460	U=.25	W=0	M=0	: Masonry Shear Walls
2	E=156	U=.25	W=0	M=0	: CDX Plywood Shear Walls
3	E=160	U=.25	W=0	M=0	: CDX - W. Classrm/E. Sloped
4	E=140	U=.25	W=0	M=0	: CDX - Gymnasium Roof
5	E=160	U=.25	W=0	M=0	: CDX - Locker/Stage Roof
6	E=160	U=.25	W=0	M=0	: CDX - Patio/Walkway Roof
7	E=2460	U=.25	W=0	M=0	: CMU Walls at Stage
8	E=2460	U=.25	W=0	M=0	: CMU - South Wall of Gym

1	JQ=20,34,21,36	ETYPE=0	M=3	TH=.5,.5	LP=2 : Membrane Element
2	JQ=21,36,22,38	ETYPE=0	M=3	TH=.5,.5	LP=2
3	JQ=22,38,23,41	ETYPE=0	M=3	TH=.5,.5	LP=2
4	JQ=23,41,24,42	ETYPE=0	M=3	TH=.5,.5	LP=2
5	JQ=43,55,44,56	ETYPE=1	M=4	TH=.625,.625	LP=2
6	JQ=44,56,45,57	ETYPE=1	M=4	TH=.625,.625	LP=2
7	JQ=45,57,46,58	ETYPE=1	M=4	TH=.625,.625	LP=2
8	JQ=46,58,47,59	ETYPE=1	M=4	TH=.625,.625	LP=2
9	JQ=47,59,48,60	ETYPE=1	M=4	TH=.625,.625	LP=2
10	JQ=55,67,56,68	ETYPE=1	M=4	TH=.625,.625	LP=2
11	JQ=56,68,57,69	ETYPE=1	M=4	TH=.625,.625	LP=2
12	JQ=57,69,58,70	ETYPE=1	M=4	TH=.625,.625	LP=2
13	JQ=58,70,59,71	ETYPE=1	M=4	TH=.625,.625	LP=2
14	JQ=59,71,60,72	ETYPE=1	M=4	TH=.625,.625	LP=2
15	JQ=67,77,68,78	ETYPE=1	M=4	TH=.625,.625	LP=2
16	JQ=68,78,69,79	ETYPE=1	M=4	TH=.625,.625	LP=2
17	JQ=69,79,70,80	ETYPE=1	M=4	TH=.625,.625	LP=2
18	JQ=70,80,71,81	ETYPE=1	M=4	TH=.625,.625	LP=2
19	JQ=71,81,72,82	ETYPE=1	M=4	TH=.625,.625	LP=2
20	JQ=77,89,78,90	ETYPE=1	M=4	TH=.625,.625	LP=2
21	JQ=78,90,79,91	ETYPE=1	M=4	TH=.625,.625	LP=2
22	JQ=79,91,80,92	ETYPE=1	M=4	TH=.625,.625	LP=2
23	JQ=80,92,81,93	ETYPE=1	M=4	TH=.625,.625	LP=2
24	JQ=81,93,82,94	ETYPE=1	M=4	TH=.625,.625	LP=2
25	JQ=89,101,90,102	ETYPE=1	M=4	TH=.625,.625	LP=2
26	JQ=90,102,91,103	ETYPE=1	M=4	TH=.625,.625	LP=2

27	JQ=91,103,92,104	ETYPE=1	M=4	TH=.625,.625	LP=2	
28	JQ=92,104,93,105	ETYPE=1	M=4	TH=.625,.625	LP=2	
29	JQ=93,105,94,106	ETYPE=1	M=4	TH=.625,.625	LP=2	
30	JQ=101,121,102,122	ETYPE=1	M=4	TH=.625,.625	LP=2	
31	JQ=102,122,103,123	ETYPE=1	M=4	TH=.625,.625	LP=2	
32	JQ=103,123,104,124	ETYPE=1	M=4	TH=.625,.625	LP=2	
33	JQ=104,124,105,125	ETYPE=1	M=4	TH=.625,.625	LP=2	
34	JQ=105,125,106,126	ETYPE=1	M=4	TH=.625,.625	LP=2	: 35 not used
36	JQ=144,15,139,20	ETYPE=1	M=2	TH=.875,.875	LP=3	: Plate Element
37	JQ=145,17,141,22	ETYPE=1	M=2	TH=1.0,1.0	LP=3	
38	JQ=146,19,143,24	ETYPE=1	M=2	TH=.875,.875	LP=3	
39	JQ=1,2,161,163	ETYPE=0	M=2	TH=.875,.875	LP=3	
40	JQ=161,163,162,164	ETYPE=0	M=2	TH=.875,.875	LP=3	: 41 & 42 not used
43	JQ=133,53,43,55	ETYPE=0	M=1	TH=8,8	LP=3	: 44 & 45 not used
46	JQ=53,65,55,67	ETYPE=0	M=1	TH=8,8	LP=3	
47	JQ=61,73,63,75	ETYPE=0	M=1	TH=8,8	LP=3	
48	JQ=73,83,75,85	ETYPE=0	M=1	TH=8,8	LP=3	
49	JQ=83,95,85,97	ETYPE=0	M=1	TH=8,8	LP=3	
50	JQ=85,97,87,99	ETYPE=0	M=1	TH=8,8	LP=3	
51	JQ=87,99,89,101	ETYPE=0	M=1	TH=8,8	LP=3	
52	JQ=95,107,97,113	ETYPE=0	M=1	TH=8,8	LP=3	
53	JQ=97,113,99,119	ETYPE=0	M=1	TH=8,8	LP=3	
54	JQ=99,119,101,121	ETYPE=0	M=1	TH=8,8	LP=3	
55	JQ=33,50,42,52	ETYPE=0	M=1	TH=8,8	LP=3	
56	JQ=42,52,134,54	ETYPE=0	M=1	TH=8,8	LP=3	
57	JQ=134,54,48,60	ETYPE=0	M=1	TH=8,8	LP=3	
58	JQ=50,62,52,64	ETYPE=0	M=1	TH=8,8	LP=3	
59	JQ=52,64,54,66	ETYPE=0	M=1	TH=8,8	LP=3	
60	JQ=54,66,60,72	ETYPE=0	M=1	TH=8,8	LP=3	
61	JQ=62,74,64,76	ETYPE=0	M=1	TH=8,8	LP=3	
62	JQ=74,84,76,86	ETYPE=0	M=1	TH=8,8	LP=3	
63	JQ=84,96,86,98	ETYPE=0	M=1	TH=8,8	LP=3	
64	JQ=86,98,88,100	ETYPE=0	M=1	TH=8,8	LP=3	
65	JQ=88,100,94,106	ETYPE=0	M=1	TH=8,8	LP=3	
66	JQ=96,112,98,118	ETYPE=0	M=1	TH=8,8	LP=3	
67	JQ=98,118,100,120	ETYPE=0	M=1	TH=8,8	LP=3	
68	JQ=100,120,106,126	ETYPE=0	M=1	TH=8,8	LP=3	
69	JQ=25,27,34,35	ETYPE=0	M=1	TH=12,12	LP=3	
70	JQ=34,35,133,36	ETYPE=0	M=1	TH=12,12	LP=3	
71	JQ=133,36,43,44	ETYPE=0	M=1	TH=12,12	LP=3	
72	JQ=27,29,35,38	ETYPE=0	M=1	TH=12,12	LP=3	
73	JQ=35,38,36,39	ETYPE=0	M=1	TH=12,12	LP=3	
74	JQ=44,45,36	ETYPE=0	M=1	TH=12,12	LP=3	
75	JQ=36,39,45	ETYPE=0	M=1	TH=12,12	LP=3	
76	JQ=45,46,39	ETYPE=0	M=1	TH=12,12	LP=3	
77	JQ=29,31,38,40	ETYPE=0	M=1	TH=12,12	LP=3	
78	JQ=38,40,39,41	ETYPE=0	M=1	TH=12,12	LP=3	
79	JQ=39,41,46	ETYPE=0	M=1	TH=12,12	LP=3	
80	JQ=46,47,41	ETYPE=0	M=1	TH=12,12	LP=3	
81	JQ=107,108,113,114	ETYPE=0	M=1	TH=12,12	LP=3	
82	JQ=113,114,119,135	ETYPE=0	M=1	TH=12,12	LP=3	
83	JQ=108,109,114,115	ETYPE=0	M=1	TH=12,12	LP=3	
84	JQ=114,115,122,123	ETYPE=0	M=1	TH=12,12	LP=3	
85	JQ=110,111,116,117	ETYPE=0	M=1	TH=12,12	LP=3	
86	JQ=116,117,124,125	ETYPE=0	M=1	TH=12,12	LP=3	

87	JQ=111,112,117,118	ETYPE=0	M=1	TH=12,12	LP=3	
88	JQ=117,118,125,126	ETYPE=0	M=1	TH=12,12	LP=3	: 89 thru 94 not used
95	JQ=119,135,121,122	ETYPE=0	M=1	TH=12,12	LP=3	
96	JQ=135,136,122,123	ETYPE=0	M=1	TH=12,12	LP=3	
97	JQ=137,138,124,125	ETYPE=0	M=1	TH=12,12	LP=3	
98	JQ=138,120,125,126	ETYPE=0	M=1	TH=12,12	LP=3	
99	JQ=31,33,40,42	ETYPE=0	M=1	TH=12,12	LP=3	
100	JQ=40,42,41,134	ETYPE=0	M=1	TH=12,12	LP=3	
101	JQ=41,134,47,48	ETYPE=0	M=1	TH=12,12	LP=3	
102	JQ=139,20,140,21	ETYPE=0	M=3	TH=.5,.5	LP=2	: Membrane Element
103	JQ=140,21,141,22	ETYPE=0	M=3	TH=.5,.5	LP=2	
104	JQ=141,22,142,23	ETYPE=0	M=3	TH=.5,.5	LP=2	
105	JQ=142,23,143,24	ETYPE=0	M=3	TH=.5,.5	LP=2	
106	JQ=13,12,139	ETYPE=0	M=3	TH=.5,.5	LP=1	
107	JQ=13,139,14,140	ETYPE=0	M=3	TH=.5,.5	LP=2	
108	JQ=152,14,140	ETYPE=0	M=3	TH=.5,.5	LP=1	
109	JQ=152,140,153,141	ETYPE=0	M=3	TH=.5,.5	LP=2	
110	JQ=154,153,141	ETYPE=0	M=3	TH=.5,.5	LP=1	
111	JQ=154,141,155,142	ETYPE=0	M=3	TH=.5,.5	LP=2	
112	JQ=156,155,142	ETYPE=0	M=3	TH=.5,.5	LP=1	
113	JQ=157,156,142	ETYPE=0	M=3	TH=.5,.5	LP=1	
114	JQ=157,142,158,143	ETYPE=0	M=3	TH=.5,.5	LP=2	
115	JQ=159,158,143	ETYPE=0	M=3	TH=.5,.5	LP=1	
116	JQ=1,144,161	ETYPE=1	M=2	TH=.875,.875	LP=1	
117	JQ=161,144,162,139	ETYPE=1	M=2	TH=.875,.875	LP=3	
118	JQ=12,139,162	ETYPE=1	M=2	TH=.875,.875	LP=1	
119	JQ=5,145,169,141	ETYPE=1	M=2	TH=1.0,1.0	LP=3	
120	JQ=192,230,193,12	ETYPE=0	M=6	TH=.5,.5	LP=2	
121	JQ=193,12,194,13	ETYPE=0	M=6	TH=.5,.5	LP=2	
122	JQ=194,13,195,14	ETYPE=0	M=6	TH=.5,.5	LP=2	
123	JQ=195,14,196,152	ETYPE=0	M=6	TH=.5,.5	LP=2	
124	JQ=196,152,197,153	ETYPE=0	M=6	TH=.5,.5	LP=2	
125	JQ=197,153,198,154	ETYPE=0	M=6	TH=.5,.5	LP=2	
126	JQ=198,154,199,155	ETYPE=0	M=6	TH=.5,.5	LP=2	
127	JQ=199,155,200,156	ETYPE=0	M=6	TH=.5,.5	LP=2	
128	JQ=200,156,201,157	ETYPE=0	M=6	TH=.5,.5	LP=2	
129	JQ=201,157,202,158	ETYPE=0	M=6	TH=.5,.5	LP=2	
130	JQ=202,158,203,159	ETYPE=0	M=6	TH=.5,.5	LP=2	
131	JQ=162,164,12,13	ETYPE=0	M=2	TH=.875,.875	LP=3	
132	JQ=2,3,163,165	ETYPE=0	M=2	TH=.375,.37	LP=3	
133	JQ=164,166,13,14	ETYPE=0	M=2	TH=.375,.375	LP=3	
134	JQ=3,4,165,167	ETYPE=0	M=2	TH=.875,.875	LP=3	
135	JQ=165,167,166,168	ETYPE=0	M=2	TH=.875,.875	LP=3	
136	JQ=166,168,14,152	ETYPE=0	M=2	TH=.875,.875	LP=3	
137	JQ=168,169,152,153	ETYPE=0	M=2	TH=.375,.375	LP=3	
138	JQ=169,171,153,154	ETYPE=0	M=2	TH=.375,.375	LP=3	
139	JQ=6,7,170,172	ETYPE=0	M=2	TH=.375,.375	LP=3	
140	JQ=171,173,154,155	ETYPE=0	M=2	TH=.375,.375	LP=3	
141	JQ=7,8,172,174	ETYPE=0	M=2	TH=.875,.875	LP=3	
142	JQ=172,174,173,175	ETYPE=0	M=2	TH=.875,.875	LP=3	
143	JQ=173,175,155,156	ETYPE=0	M=2	TH=.875,.875	LP=3	
144	JQ=175,177,156,157	ETYPE=0	M=2	TH=.375,.375	LP=3	
145	JQ=9,10,176,178	ETYPE=0	M=2	TH=.375,.375	LP=3	
146	JQ=177,179,157,158	ETYPE=0	M=2	TH=.375,.375	LP=3	
147	JQ=10,11,178,182	ETYPE=0	M=2	TH=.875,.875	LP=3	

148	JQ=178,182,179,183	ETYPE=0	M=2	TH=.875,.875	LP=3	
149	JQ=179,183,158,159	ETYPE=0	M=2	TH=.875,.875	LP=3	
150	JQ=230,231,12,139	ETYPE=1	M=6	TH=.5,.5	LP=2	
151	JQ=231,232,139,20	ETYPE=1	M=6	TH=.5,.5	LP=2	
152	JQ=232,233,20,34	ETYPE=1	M=6	TH=.5,.5	LP=2	
153	JQ=233,234,34,51	ETYPE=1	M=6	TH=.5,.5	LP=2	
154	JQ=234,235,51,63	ETYPE=1	M=6	TH=.5,.5	LP=2	
155	JQ=235,236,63,75	ETYPE=1	M=6	TH=.5,.5	LP=2	
156	JQ=236,237,75,85	ETYPE=1	M=6	TH=.5,.5	LP=2	
157	Q=237,238,85,97	ETYPE=1	M=6	TH=.5,.5	LP=2	
158	JQ=238,239,97,113	ETYPE=1	M=6	TH=.5,.5	LP=2	
159	JQ=239,240,113,241	ETYPE=1	M=6	TH=.5,.5	LP=2	
160	JQ=113,241,135,250	ETYPE=1	M=6	TH=.5,.5	LP=2	
161	JQ=241,242,250,251	ETYPE=1	M=6	TH=.5,.5	LP=2	
162	JQ=242,243,251,244	ETYPE=1	M=6	TH=.5,.5	LP=2	
163	JQ=251,244,252,245	ETYPE=1	M=6	TH=.5,.5	LP=2	
164	JQ=252,245,254,246	ETYPE=1	M=6	TH=.5,.5	LP=2	
165	JQ=138,253,118,249	ETYPE=1	M=6	TH=.5,.5	LP=2	
166	JQ=253,254,249,248	ETYPE=1	M=6	TH=.5,.5	LP=2	
167	JQ=254,246,248,247	ETYPE=1	M=6	TH=.5,.5	LP=2	: Elements 168-170
171	JQ=108,225,114,257	ETYPE=0	M=7	TH=8,8	LP=3	: Not Used
172	JQ=114,257,135,250	ETYPE=0	M=7	TH=8,8	LP=3	
173	JQ=135,250,122,258	ETYPE=0	M=7	TH=8,8	LP=3	
174	JQ=225,127,257,129	ETYPE=0	M=7	TH=8,8	LP=3	
175	JQ=257,129,250,251	ETYPE=0	M=7	TH=8,8	LP=3	
176	JQ=250,251,258,131	ETYPE=0	M=7	TH=8,8	LP=3	
177	JQ=111,226,117,259	ETYPE=0	M=7	TH=8,8	LP=3	
178	JQ=117,259,138,253	ETYPE=0	M=7	TH=8,8	LP=3	
179	JQ=138,253,125,260	ETYPE=0	M=7	TH=8,8	LP=3	
180	JQ=226,128,259,130	ETYPE=0	M=7	TH=8,8	LP=3	
181	JQ=259,130,253,254	ETYPE=0	M=7	TH=8,8	LP=3	
182	JQ=253,254,260,132	ETYPE=0	M=7	TH=8,8	LP=3	
183	JQ=127,222,129,252	ETYPE=0	M=7	TH=8,8	LP=3	
184	JQ=129,252,251,255	ETYPE=0	M=7	TH=8,8	LP=3	
185	JQ=251,255,131,256	ETYPE=0	M=7	TH=8,8	LP=3	
186	JQ=222,128,252,130	ETYPE=0	M=7	TH=8,8	LP=3	
187	JQ=252,130,255,254	ETYPE=0	M=7	TH=8,8	LP=3	
188	JQ=255,254,256,132	ETYPE=0	M=7	TH=8,8	LP=3	
189	JQ=122,258,123,262	ETYPE=0	M=5	TH=.5,.5	LP=2	
190	JQ=123,262,124,263	ETYPE=0	M=5	TH=.5,.5	LP=2	
191	JQ=124,263,125,260	ETYPE=0	M=5	TH=.5,.5	LP=2	
192	JQ=258,131,262,256	ETYPE=0	M=5	TH=.5,.5	LP=2	
193	JQ=263,262,256	ETYPE=0	M=5	TH=.5,.5	LP=1	
194	JQ=263,256,260,132	ETYPE=0	M=5	TH=.5,.5	LP=2	
195	JQ=217,218,243,244	ETYPE=0	M=2	TH=.375,.375	LP=3	
196	JQ=218,219,244,245	ETYPE=0	M=2	TH=.375,.375	LP=3	
197	JQ=219,220,245,246	ETYPE=0	M=2	TH=.375,.375	LP=3	
198	JQ=220,221,246,247	ETYPE=0	M=2	TH=.375,.375	LP=3	
199	JQ=153,141,169	ETYPE=0	M=2	TH=1.0,1.0	LP=1	
200	JQ=11,146,182	ETYPE=0	M=2	TH=.875,.875	LP=1	
201	JQ=182,146,183,143	ETYPE=0	M=2	TH=.875,.875	LP=3	
202	JQ=159,143,183	ETYPE=0	M=2	TH=.875,.875	LP=1	
203	JQ=107,215,113,241	ETYPE=0	M=2	TH=.375,.375	LP=3	
204	JQ=215,216,241,242	ETYPE=0	M=2	TH=.375,.375	LP=3	
205	JQ=216,217,242,243	ETYPE=0	M=2	TH=.375,.375	LP=3	

```
206   JQ=112,224,118,249     ETYPE=0  M=2  TH=.375,.375  LP=3
207   JQ=224,223,249,248     ETYPE=0  M=2  TH=.375,.375  LP=3
208   JQ=223,221,248,247     ETYPE=0  M=2  TH=.375,.375  LP=3
209   JQ=34,267,266,268      ETYPE=0  M=8  TH=8,8        LP=3
210   JQ=266,268,133,269     ETYPE=0  M=8  TH=8,8        LP=3
211   JQ=267,51,268,270      ETYPE=0  M=8  TH=8,8        LP=3
212   JQ=268,270,269,53      ETYPE=0  M=8  TH=8,8        LP=3
213   JQ=51,271,270,272      ETYPE=0  M=8  TH=8,8        LP=3
214   JQ=270,272,53,273      ETYPE=0  M=8  TH=8,8        LP=3
215   JQ=271,63,272,274      ETYPE=0  M=8  TH=8,8        LP=3
216   JQ=272,274,273,65      ETYPE=0  M=8  TH=8,8        LP=3
217   JQ=25,49,34,51         ETYPE=0  M=8  TH=8,8        LP=3
218   JQ=49,61,51,63         ETYPE=0  M=8  TH=8,8        LP=3
                               : Blank Line
SPEC
A=42.5   S=386.4   D=.05  : Corralitos Record (Angle, Scale Factor, Damping)
 .0       .478     .629    .431 : (Time Period, E-W Motion, N-S Motion, Up-Dn)
 .040     .479     .657    .463
 .042     .482     .657    .495
 .044     .484     .656    .517
 .046     .494     .663    .537
 .048     .503     .671    .557
 .050     .506     .677    .583
 .055     .496     .692    .613
 .060     .492     .723    .620
 .065     .528     .735    .610
 .070     .566     .740    .589
 .075     .578     .753    .570
 .080     .585     .741    .599
 .085     .578     .727    .607
 .090     .536     .757    .607
 .095     .546     .782    .758
 .100     .575     .833    .865
 .110     .621     .773    .920
 .120     .744     .727    .899
 .130     .891     .845    .743
 .140     .807     .890    .663
 .150     .830     .931    .766
 .160     .925     .992    .832
 .170     .989    1.090    .961
 .180     .927    1.080   1.030
 .190     .985    1.070   1.150
 .200    1.010     .988   1.300
 .220     .942    1.300   1.330
 .240    1.020    1.680   1.270
 .260     .970    1.950   1.140
 .280     .901    2.100    .843
 .300     .978    2.120    .663
 .320     .936    2.040    .486
 .400     .799    1.650    .301
 .500    1.020    1.410    .443
 .600    1.370    1.100    .261
 .700    1.330    1.090    .287
 .800    1.330     .614    .275
 .900     .900     .519    .197
```

1.000	.387	.412	.180
1.500	.343	.185	.0924
2.000	.122	.173	.0754

: Blank Line

COMBO
1 C=1.0D=1.0

SELECT
NT=1 ID=1,274,1 SW=1
NT=2 ID=1,274,1 SW=1
NT=5 ID=1,111,1 SW=1
NT=6 ID=1,218,1 SW=1

APPENDIX B

BUILDING NO. 2 - PENINSULA OFFICE BUILDING

B.1 Design Criteria

This section presents information on the material properties and seismic dead loads used in the analyses. The size, location and types of materials used in construction were obtained from the original plans and specifications for the building. In addition, a copy of the original Foundation Investigation Report was available which provided information on the subsurface conditions at the site.

I. Materials

A.	Structural Steel	
	1. Structural Steel Shapes	ASTM Carbon Steel, Grade 36
	- For W8x31 & W8x48 only	ASTM A441
	2. Steel Tubes	ASTM A500, Grade B (Fy = 46 ksi)
	E_s = 29,000 ksi,	Poisson's Ratio = 0.30
B.	Cast-in-Place Concrete	
	1. Footings & Slab-on-Grade	f'_c = 2,500 psi @ 28 days
	E_c = 2,850 ksi,	Poisson's Ratio =0 .20
	2. Columns	f'_c = 4,000 psi @ 28 days
	E_c = 3,605 ksi,	Poisson's Ratio = 0.20
	3. All Other Concrete	f'_c = 3,000 psi @ 28 days
	E_c = 3,122 ksi,	Poisson's Ratio = 0.20
C.	Masonry	
	1. Face Brick Masonry Units	ASTM C216, Type FBS, Grade SW Std size 3-5/8 x 2-1/4 x 7-5/8"
	2. Thin Brick Masonry Units	ASTM C216, Type FBS, Grade SW Size 1 x 2-1/4 x 7-5/8"
	3. Mortar	ASTM 270, Type S
	4. Epoxy Adhesive	"Latapoxy 210" Epoxy Adhesive
	f'_m = 3,350 psi,	Poisson's Ratio = 0.20
D.	Soils	
	1. Unit Weight of Soil	100 pcf
	2. Poisson's Ratio for Soil	0.333
	3. Shear Wave Velocity	922 ft/sec @ 18 ft. depth

II. Seismic Dead Loads (Used in lateral analyses)

First Floor:

Dead Load =	12" Concrete Slab + 1" for Drop Panels & Depressed Tile Areas	= 163 psf
	Mech, Elect & Plumbing (Below Grade Parking Area)	= 2 psf
	Carpet	= 1 psf
		166 psf
	Interior Partitions	= 10 psf
		176 psf
Additional Loads (file cabinets, bookshelves, desks)		= 15 psf
		191 psf

Floors Two thru Four:

Dead Load =	4" N.W. Concrete Slab Over 1 1/2-inch Composite Steel Deck	= 40 psf
	Steel Framing (Beams & Girders)	= 8 psf
	Mech, Elect & Plumbing	= 6 psf
	Acoustical Lay-in Ceiling/Grid	= 2 psf
	Carpet	= 1 psf
		57 psf
	Interior Partitions	= 10 psf
		67 psf
Additional Loads (file cabinets, bookshelves, desks)		= 15 psf
		82 psf

Roof:

Dead Load =	1 1/2-inch, 20-Ga Steel Deck	= 2 psf
	Steel Framing (Beams & Girders)	= 6 psf
	Built-up-Roof w/ Ballast	= 10 psf
	Insulating Fill (4" avg thickness)	= 16 psf
	Mech, Elect & Plumbing	= 4 psf
	Accoustical Lay-in Ceiling/Grid	= 2 psf
		40 psf

Exterior Wall System:

1. Full Brick Veneer over Metal Studs

- Brick Veneer		= 40 psf
- 1" Slushed Mortar Joint	= 12 psf	
- 4" - 16 Ga Steel Studs @ 16" o.c.	= 2 psf	
- Insulation		= 2 psf
- 5/8" Gypsum Board		= 3 psf
Wall Weight =		59 psf

2. Thin Brick over Stucco & Metal Studs

- Thin Brick		= 10 psf
- 1" Mortar Bed Over Wire Mesh	= 12 psf	
- 4" - 16 Ga Steel Studs @ 16" o.c.	= 2 psf	
- Insulation		= 2 psf
- 5/8" Gypsum Board		= 3 psf
Wall Weight =		29 psf

3. Stucco Over Wire Lath & Metal Studs

- 1" Stucco Over Wire Mesh		= 10 psf
- 4" - 16 Ga Steel Studs @ 16" o.c.	= 2 psf	
- Insulation		= 2 psf
- 5/8-inch Gypsum Board	= 3 psf	
Wall Weight =		17 psf

4. Perimeter Balcony Railings - 2'-6" Height (Weight/Ft.)

- Full Brick Veneer at Exterior	40 psf x 2'	= 80 lb/ft
- 1" Slushed Exterior Mortar Joint	12 psf x 2'	= 24 lb/ft
- Thin Brick at Interior	10 psf x 2'	= 20 lb/ft
- 3/4" Interior Mortar Bed	9 psf x 2'	= 18 lb/ft
- 8" - 16 Ga Steel Studs @ 16" o.c.	3 psf x 2'	= 6 lb/ft
- Rowlock Course	40 psf x 1.33'	= 53 lb/ft
- Vert 2 1/2"-Dia Posts & Hardware		= 6 lb/ft
- 5"-Dia Std Pipe Handrail		= 15 lb/ft
- Structural Channel C10x15.3		= 15 lb/ft
Wall Weight =		237 lb/ft

5. Exterior Glazing (Windows, Glass & Frame) 8 psf

B.2 Design Calculations - Peninsula Office Building

Sample calculations are presented below for soil properties, effective damping, static stiffness coefficients, equivalent column properties and the computation of horizontal racking at shear panels.

I. Soil Properties

Dry Unit Weight (Ref. p. 12 Bowles [4])

(p. 14 Bowles)
$G_s = 2.65$ for sands

$$\gamma_{Dry} = \frac{\gamma_W \, G_s}{1 + e}$$

For medium soil assume e = 0.60 (p. 79 Garcia-Delgado [12])

$$\gamma_D = \frac{(62.4)(2.65)}{1 + .60} = 103.4 \approx 100 \text{ PCF}$$

Shear Wave Velocity v_s (Ft/Sec) (p. 80 Garcia-Delgado [12])

For $\gamma_d = 100$ PCF & Medium Soil

$$v_s = (159 - 32.1) \, [66.666 \, (z + 23.71)]^{0.25}$$

z = Depth of Influence used in Computing v_s

Effective depth of significant influence can be determined as follows: (p. 141 NEHRP Commentary [23])

- Horizontal & Vertical Motion $4 \, r_a$
- Rocking Motion $1.5 \, r_m$

where

$$r_a = \sqrt{\frac{A_o}{\pi}}$$

$$r_m = \left[\frac{4 I_o}{\pi}\right]^{.25}$$

A_o = Area of Foundation

I_o = Moment of Inertia of Foundation

$A_o = 134\ ft^2$ (average value)

$I_o = 686{,}620\ ft^4$ (I_o for perimeter foundation wall/footings)

$$r_a = \sqrt{\frac{134}{3.14}} = 6.5\ ft$$

$$r_m = \left[\frac{4\ (686{,}620)}{3.14}\right]^{0.25} = 30.6\ ft$$

Effective Depth of Influence	Avg. Depth of Influence
- Horizontal & Vertical Motion - 4 x 6.5 ft. = 26 ft.	26/2 = 13'
- Rocking Motion - 1.5 x 30.6 ft. = 46 ft.	46/2 = 23'

$$\text{Depth to Compute } v_s = \frac{13 + 23}{2} = 18\ ft$$

Shear Modulus, G (psf) (C6A-11, p. 138, NEHRP Commentary [23])

$$G = \gamma\, v_s^2/g$$

$$= 100\ (922)^2/32.2$$

$$= 2{,}640{,}012\ lb/ft^2$$

$$v_s \ @\ 18\ ft. = (159-32.1)\ [66.666\ \ (18 + 23.71)]^{0.25} = 922\ ft/sec$$

II. Effective Damping

No Shear Panels

$$\frac{T'}{T} = \frac{0.746 \text{ sec}}{0.718 \text{ sec}} = 1.039$$

Including Shear Panels

$$\frac{T'}{T} = \frac{0.552 \text{ sec}}{0.507 \text{ sec}} = 1.089$$

T′ & T were obtained from ETABS computer runs with and without equivalent columns to model soil flexibility.

Effective Damping (Ref. NEHRP [23] p. 67)

No Shear Panels

$\beta = \beta_o + 0.05/(T'/T)^3$ where $\beta_o = 0.015$ (Figure 6A-1 [23])

$\beta = 0.015 + 0.05/1.039^3$

$\beta = 0.015 + 0.0446$

$\beta = 0.059$

With Shear Panels

$\beta' = 0.025 + 0.05/(1.089)^3$ $\beta_o = 0.025$ (Figure 6A-1 [23])

$= 0.025 + 0.039$

$\beta' = 0.064$

III. Static Stiffness Coefficients

Footing ID: Grids C-4, E-4 & E-5

Footing Size: 11 x 11 x 2 ft. Embedment: 2.33 ft. $g = 32.2 \text{ ft/sec}^2$

Assumptions: Poisson's Ratio = .333 sec

v_s = 922 ft/sec

$$\gamma = 100 \text{ PCF}$$

Compute G, A_b, A_s & $A_b/4L^2$ $e = \gamma/g$ (Mass Density)

a. $$G = \frac{\gamma v_s^2}{g} = \frac{100(922)^2}{32.2} = 2{,}640{,}012 \text{ lb/ft}^2$$ (Shear Modulus)

b. $$A_b = 11 \times 11 = 121 \text{ ft}^2$$ (Bearing Area)

c. $$A_s = 4(11 \times 2) = 88 \text{ ft}^2$$ (Embedded Soil Area)

d. $$\frac{A_b}{4L^2} = \frac{121}{(4 \times 5.5^2)} = 1.0$$

Compute K_z:

$$K_{surface} = \frac{2LG}{1-v}\left[.73 + 1.54\left(\frac{A_b}{4L^2}\right)^{0.75}\right] = 3$$

$$= \frac{(2{,}640{,}012)(2)(5.5)}{(1-.333)}\left[0.73 \text{ x } 1.54\ (1.0)^{0.75}\right]$$

$$= 98{,}832{,}251 \text{ lb/ft}$$

$$I_{trench} = 1 + \left(\frac{1}{21}\right)\left(\frac{D}{B}\right)\left(1 + \frac{4}{3} \text{ x } \frac{A_b}{4L^2}\right)$$

$$= 1 + \left(\frac{1}{21}\right)\left(\frac{2.33}{5.5}\right)\left(1 + \frac{4}{3} \text{ x } 1.0\right)$$

$$= 1.04713$$

$$I_{wall} = 1 + .19\left(\frac{A_s}{A_b}\right)^{2/3}$$

$$= 1 + .19\left(\frac{88}{121}\right)^{2/3}$$

$$= 1.15366$$

$$K_z = K_{surface} \text{ x } I_{trench} \text{ x } I_{wall}$$

$$= 98{,}832{,}251 \text{ x } 1.04713 \text{ x } 1.15366$$

$$= 119{,}392{,}521 \text{ lb/ft}$$

Compute K_y: (Short Direction)

$$K_y = S_y\,\frac{(2LG)}{2-v}$$

For $\frac{A_b}{4L^2} = 1.0 > 0.175,\ S_y = 4.5 \text{ x } (1.0)^{0.40} = 4.5$

$$K_y = 4.5 \text{ x } \frac{(2)(5.5)(2{,}640{,}012)}{(2 - 0.333)} = 78{,}392{,}690 \text{ lb/ft}$$

Compute K_x: (Long Direction)

$$K_x = (S_x)\,\frac{2LG}{(2-\nu)}$$

$$= (4.5)\,\frac{2(5.5)2{,}640{,}012}{(2-0.333)} = 78{,}392{,}690 \text{ lb/ft}$$

Compute $K_{\theta x}$ (Short Direction)

$$K_{\theta x} = S_{rx} \text{ x } \frac{G}{1-\nu} \text{ x } \frac{I_{x-x}^{0.75}}{\left(\frac{A_b}{4L^2}\right)^{.75}}$$

$$= 3.2 \text{ x } \frac{2{,}640{,}012}{(1-0.333)} \text{ x } \frac{(1{,}220.1)^{0.75}}{(1.0)^{0.75}}$$

$$= 2{,}614{,}729{,}009 \text{ lb-ft}$$

Compute $K_{\theta y}$ (Long Direction)

$$K_{\theta y} = S_{ry} \left[\frac{6}{1-\nu} \right] I_{y-y}^{0.75}$$

$$= 3.2 \left[\frac{2,640,012}{1-0.333} \right] (1,220.1)^{0.75}$$

$$= 2,614,729,009 \text{ lb-ft}$$

IV. Compute Equivalent Column Properties

$$K_{xi} = \frac{12\ EI_y}{L^3\ (1 + 2\beta)}$$

$$K_{yi} = \frac{12\ EI_x}{L^3\ (1 + 2\beta)}$$

$$K_{zi} = \frac{AE}{L}$$

$$K_{\theta x} = \frac{2EI_x\ (2 + \beta)}{L(1 + 2\beta)}$$

$$K_{\theta y} = \frac{2EI_y\ (2 + \beta)}{L(1 + 2\beta)}$$

Set L = 1 in E = 3,605 ksi

Compute Area A

$$K_z = 119,392,521 \text{ lb /ft} = 9,949 \text{ k/in}$$

$$K_z = \frac{AE}{L}$$

$$9,949 = \frac{A\ (3,605)}{1}$$

$$A = 2.76 \text{ in}^2$$

Set β From

$$\frac{K_{\theta y}}{K_x} = \frac{2,614,729,009}{78,392,690}$$

$$33.4 = \frac{(2 + \beta)(0.0833)^2}{6}$$

$$\beta = \left[\frac{(6)(33.4)}{(0.833)^2} \right] - 2$$

$$\beta = 28,858$$

Compute EI

$$K_y = \frac{12\ EI_x}{L^3(1 + 2\beta)}$$

$$K_y = 78{,}392{,}690\ \text{lb/ft} \times \frac{1}{12{,}000} = 6{,}533\ \text{k/in}$$

$$6{,}533 = \frac{12\ EI_x}{1^3\ (1 + 2 \times 28{,}858)}$$

$$EI_x = 31{,}422{,}097$$

Compute I & A_v

$$I = \frac{31{,}422{,}097}{3{,}605} = 8{,}716\ \text{in}^4$$

$$\beta = 28{,}858 = \frac{12I}{L^2 A_v}$$

$$A_v = \frac{12I}{L^2\beta} \times \frac{12(8{,}716)}{(1)^2(28{,}858)} = 3.62\ \text{in}^2$$

V. Computation of Horizontal Racking at Shear Panels

To compute the amount of horizontal racking which occurred at shear panel locations, dummy diagonal elements were inserted into the model. The resulting force in the diagonal was used to compute its elongation from which the horizontal frame movement at each panel location was determined.

Horizontal Racking = γH

where

$$\gamma = \frac{ZL}{BH}$$

$$Z = \text{Elongation of Diagonal} = \frac{PL}{AE}$$

L = Original Length of Diagonal

B = Bay Width

H = Story Height

A typical calculation is shown below for the East Building elevation between column lines 6 and 7.

Level 2-3:

$$Z = \frac{(0.49)(253.45)}{(1)(500)} = 0.248$$

L = 253.45 in

B = 205.75 in

H = 148 in

$$\gamma = \frac{(0.248)(253.45)}{(205.75)(148)} = 0.00206$$

Horizontal Racking = γH = 0.00206 x 148 = 0.31 inches

B.3 Computer Results for the Peninsula Office Building

I. Structural Time Periods and Frequencies (Brick Shear Panels not Included)

MODE NUMBER	PERIOD (SEC)	FREQUENCY (CYCLES/SEC)	CIRCULAR/FREQ (RADIANS/SEC)
1	.74650	1.33958	8.41685
2	.68659	1.45648	9.15135
3	.43855	2.28023	14.32712
4	.24683	4.05132	25.45522
5	.21709	4.60643	28.94304
6	.15252	6.55643	41.19524
7	.14295	6.99550	43.95402
8	.13174	7.59083	47.69459
9	.11095	9.01318	56.63151
10	.10314	9.69590	60.92115
11	.09139	10.94195	68.75033
12	.07629	13.10804	82.36027
13	.07189	13.91076	87.40390
14	.06057	16.50990	103.73475
15	.05654	17.68516	111.11912
16	.01919	52.10152	327.36352
17	.01696	58.97622	370.55853
18	.00963	103.79988	652.19391

Effective Mass Factors (Brick Shear Panels not Included)

MODE NUMBER	X-TRANSLATION %-MASS	<%-SUM>	Y-TRANSLATION %-MASS	<%-SUM>	Z-ROTATION %-MASS	<%-SUM>
1	37.35	< 37.4>	.37	< .4>	.36	< .4>
2	.14	< 37.5>	36.64	< 37.0>	3.08	< 3.4>
3	1.17	< 38.7>	3.87	< 40.9>	33.56	< 37.0>
4	8.40	< 47.1>	.00	< 40.9>	1.38	< 38.4>
5	.09	< 47.2>	6.68	< 47.6>	.48	< 38.9>
6	.42	< 47.6>	1.83	< 49.4>	5.86	< 44.7>
7	1.57	< 49.1>	.00	< 49.4>	.08	< 44.8>
8	.22	< 49.4>	1.05	< 50.5>	2.36	< 47.2>
9	1.19	< 50.6>	.02	< 50.5>	.34	< 47.5>
10	.00	< 50.6>	1.10	< 51.6>	.92	< 48.4>
11	.03	< 50.6>	.21	< 51.8>	3.69	< 52.1>
12	.07	< 50.7>	4.21	< 56.0>	9.50	< 61.6>
13	.15	< 50.8>	15.41	< 71.4>	14.98	< 76.6>
14	47.86	< 98.7>	.00	< 71.4>	.42	< 77.0>
15	.28	< 99.0>	27.21	< 98.6>	22.38	< 99.4>
16	.24	< 99.2>	1.27	< 99.9>	.00	< 99.4>
17	.80	<100.0>	.12	<100.0>	.60	<100.0>
18	.00	<100.0>	.00	<100.0>	.04	<100.0>

II. Structural Time Periods and Frequencies (Brick Shear Panels Included)

MODE NUMBER	PERIOD (SEC)	FREQUENCY (CYCLES/SEC)	CIRCULAR/FREQ (RAD/SEC)
1	.55187	1.81203	11.38530
2	.48855	2.04689	12.86100
3	.29590	3.37947	21.23383
4	.18564	5.38681	33.84633
5	.13887	7.20112	45.24595
6	.12075	8.28141	52.03364
7	.09702	10.30677	64.75935
8	.07869	12.70838	79.84908
9	.07603	13.15351	82.64592
10	.06950	14.38828	90.40423
11	.06393	15.64201	98.28162
12	.05752	17.38658	109.24312
13	.05563	17.97539	112.94271
14	.05068	19.73081	123.97232
15	.03611	27.69326	174.00187
16	.01895	52.78268	331.64339
17	.01676	59.67301	374.93661
18	.00962	103.94226	653.08850

Effective Mass Factors (Brick Shear Panels Included)

MODE	X-TRANSLATION		Y-TRANSLATION		Z-ROTATION	
NUMBER	%-MASS	<%-SUM>	%-MASS	<%-SUM>	%-MASS	<%-SUM>
1	11.44	< 11.4>	25.80	< 25.8>	1.80	< 1.8>
2	23.93	< 35.4>	16.34	< 42.1>	.27	< 2.1>
3	1.28	< 36.7>	2.38	< 44.5>	36.07	< 38.1>
4	6.36	< 43.0>	1.85	< 46.4>	2.85	< 41.0>
5	7.21	< 50.2>	2.29	< 48.7>	2.27	< 43.3>
6	1.46	< 51.7>	3.46	< 52.1>	.28	< 43.5>
7	.01	< 51.7>	1.19	< 53.3>	10.12	< 53.6>
8	11.54	< 63.2>	.67	< 54.0>	4.55	< 58.2>
9	.47	< 63.7>	1.90	< 55.9>	19.70	< 77.9>
10	.58	< 64.3>	19.57	< 75.5>	2.58	< 80.5>
11	.24	< 64.5>	.44	< 75.9>	.11	< 80.6>
12	28.56	< 93.1>	.38	< 76.3>	.12	< 80.7>
13	2.18	< 95.3>	19.70	< 96.0>	13.75	< 94.5>
14	3.77	< 99.0>	2.68	< 98.7>	4.90	< 99.4>
15	.00	< 99.0>	.02	< 98.7>	.01	< 99.4>
16	.21	< 99.2>	1.20	< 99.9>	.00	< 99.4>
17	.76	<100.0>	.11	<100.0>	.59	<100.0>
18	.00	<100.0>	.00	<100.0>	.04	<100.0>

III. **Response Spectrum Lateral Story Displacements (Brick Shear Panels not Included; Displacements are at the Centers of Mass of the Respective Story Levels)**

LEVEL	DIRN	/-------LOAD CONDITIONS-------/ DYN-1	DYN-2	DYN-3
ROOF	X	1.9034	.0000	.0000
4TH	X	1.3797	.0000	.0000
3RD	X	.8539	.0000	.0000
2ND	X	.4110	.0000	.0000
1ST	X	.0169	.0000	.0000
BASE	X	.0075	.0000	.0000

Dynamic Story Shears

LEVEL	DIRN	/--------LOAD CONDITIONS--------/ DYN-1	DYN-2	DYN-3
ROOF	X	148.51	.00	.00
4TH	X	329.70	.00	.00
3RD	X	455.12	.00	.00
2ND	X	550.02	.00	.00
1ST	X	723.05	.00	.00
BASE	X	845.98	.00	.00

IV. **Response Spectrum Lateral Story Displacements (Brick Shear Panels Included; Displacements are at the Centers of Mass of the Respective Story Levels)**

LEVEL	DIRN	/--------LOAD CONDITIONS--------/ DYN-1	DYN-2	DYN-3
ROOF	X	1.3009	.0000	.0000
4TH	X	.9250	.0000	.0000
3RD	X	.5257	.0000	.0000
2ND	X	.2172	.0000	.0000
1ST	X	.0202	.0000	.0000
BASE	X	.0078	.0000	.0000

Dynamic Story Shears (Brick Shear Panels Included)

LEVEL	DIRN	/--------LOAD CONDITIONS--------/ DYN-1	DYN-2	DYN-3
ROOF	X	189.94	.00	.00
4TH	X	470.36	.00	.00
3RD	X	658.43	.00	.00
2ND	X	757.84	.00	.00
1ST	X	848.31	.00	.00
BASE	X	914.42	.00	.00

B.4 ETABS Data File

```
Peninsula Office Building - 4-Level K-Braced Frame w/Basement (etab1ac)
Units:  Kip-Inches
$
$  Equivalent columns used to model soil flexibility
$  No shear panels
$
$                                                              Control Data
 6 1 1 5 1 18 3 15 9 1 3 0 2 0 0 0 1 0 1
 386.4
$                                                              Story Mass Data - 1st Floor
 1  10  1/386.4
           .191/144    49.5*12    36*12     99*12     72*12
           .191/144    81.25*12   82.5*12   35.5*12   21.2*12
           2.00/144    49.5*12    3         99*12     6
           1.25/144    3          36*12     6         72*12
           2.48/144    30.25*12   71.8*12   60.5*12   6
            .62/144    60.5*12    82.4*12   6         21.2*12
           2.11/144    79.7*12    93*12     38.4*12   6
           1.96/144    99*12      46.5*12   6         93*12
           .248/144    8.25*12    36*12     6         72*12
$
          278.2/144    81.0*12    81.8*12   12        12
$                                                              Story Mass Data - Flrs 2 & 3
 2  8   1/386.4
           .082/144    53.5*12    36*12     99*12     72*12
           .082/144    58.5*12    82.5*12   81.1*12   21.2*12
           .838/144    53.5*12    3         91*12     6
           .838/144    8.25*12    36*12     6         72*12
          1.450/144    12.9*12    72*12     9.9*12    6
           .838/144    17.9*12    82.4*12   6         21.2*12
           .838/144    58.5*12    93*12     81.1*12   6
           .715/144    99*12      46.5*12   6         93*12
$                                                              Story Mass Data - 4th Floor
 3  11  1/386.4
           .082/144    69.1*12    9*12      59.8*12   18*12
           .082/144    53.5*12    44.9*12   91*12     53.8*12
           .082/144    58.5*12    82.6*12   81.1*12   21.2*12
           .838/144    69.1*12    3         59.8*12   6
           .838/144    39.1*12    9*12      6         18*12
           .838/144    28.5*12    18*12     31.2*12   6
           .838/144    44.9*12    8*12      6         53.8*12
          1.455/144    12.9*12    71.8*12   9.9*12    6
           .715/144    17.9*12    82.4*12   6         21.2*12
           .838/144    58.5*12    93*12     81.1*12   6
           .715/144    99*12      46.5*12   6         93*12
$                                                              Story Mass Data - Roof
 4  14  1/386.4
           .040/144    69.1*12    9*12      59.8*12   18*12
           .040/144    58.5*12    44.9*12   81.1*12   53.8*12
           .040/144    12.9*12    63.3*12   9.9*12    17.5*12
```

```
           .040/144    90.4*12    82.4*12    17.1*12    21.2*12
            .52/144    69.1*12    3          59.8*12    6
            .52/144    39.2*12    9*12       6          18*12
            .52/144    28.6*12    18*12      21.33*12   6
            .52/144    17.9*12    36.1*12    6          36.33*12
            .52/144    12.9*12    54.3*12    9.9*12     6
            .52/144    8*12       63.1*12    6          17.5*12
            .52/144    44.9*12    71.8*12    73.9*12    6
$                                                                  Story Mass Data - Basement
 5  9  1/386.4
           .060/144    49.8*12    36.5*12    99.6*12    73.0*12
           .175/144    20*12      83.6*12    40*12      21.2*12
           .060/144    69.8*12    83.6*12    59.6*12    21.2*12
           .875/144    49.8*12    6          99.6*12    12
           .625/144    6          47.1*12    12         94.2*12
           .875/144    49.8*12    94.2*12    99.6*12    12
           .875/144    99.6*12    47.1*12    12         94.2*12
           .875/144    49.8*12    94.2*12    99.6*12    12
           .875/144    87.6*12    25*12      24*12      12
$                                                                  Story Data
 Roof                  12.00*12  4
 4th                   12.00*12  3
 3rd                   12.33*12  2
 2nd                   12.33*12  2
 1st                   10.1*12    1  0    0    0     0    4780   4780
 Base       1          5  0    0    0     0    9559  9559
$                                                                  Frame Member Material
$                                                                  - Steel Bms & Cols

 1  S  29000   .0002836  .3  36  .66*36  .75*36
$                                                                  - Concrete Cols
 2  C  3605    .0000868  .2  60  4       60
$                                                                  - Concrete Walls
 3  W  3122    .0000868  .2  60  3       60
$                                                                  Frame Member Sections
$                                                                  - Steel Cols.
          1    1    W8X31
          2    1    TS6X6X1/4
          3    1    W8X24
          4    1    W8X48
$                                                                  - Concrete Cols.
          5    2    RECT   14   14     0     0
          6    2    RECT   15   15     0     0
          7    2    RECT   24   14     0     0
          8    2    RECT   28   14     0     0
$                                                                  - Equivalent Columns
          9    2    USER   0    0      0     0
2.760          3.624       3.624   0        8703    8703
10        2    USER        0    0      0     0
3.193          4.283       4.283   0        14366   14366
         11    2    USER   0    0      0     0
2.340          2.965       2.965   0        4767    4767
```

```
        12    2   USER   0    0     0     0
1.996         2.471      2.471   0       2758    2758
        13    2   USER   0    0     0     0
1.474         1.812      1.812   0       1088    1088
        14    2   USER   0    0     0     0
2.977         3.954      3.954   0       11299   11299
        15    2   USER   0    0     0     0
.220          0    0     0          .01657  .01657
$                                                                   - Steel Beams
 1  1  W12x26
 2  1  W14x30
 3  1  W14x38
 4  1  W16x36
 5  1  TS8x6x1/4
$                                                                   - Concrete Grade Beams
 6  2  RECT   20   4   60    0   0
 7  2  RECT   14   4   60    0   0
 8  2  RECT   14   4   96    0   0
 9  2  RECT   14   4   120   0   0
$                                                                   - Steel Brace Members
 1  1  TS6x6x1/4
$                                                                   - Concrete Wall Panels
 1  3  10
 2  3  12
 3  3  14
$                                                                   Data for Frame #1
 4-Level K-Braced 3-D Frame w/ Basement
 1  6  48  50  55  38  0  32  0
$                                                                   Column Line Coordinates
 1            17.86*12      93*12        0
 2            39.19*12      93*12        0
 3            60.52*12      93*12        0
 4            81.85*12      93*12        0
 5            90.39*12      93*12        0
 6            99*12         93*12        0
 7             7.98*12      71.8*12      90
 8            17.86*12      71.8*12      90
 9            39.19*12      71.8*12      90
 10           50.35*12      71.8*12      0
 11           81.85*12      71.8*12      0
 12           90.39*12      71.8*12      0
 13           99*12         71.8*12      0
 14           7.98*12       63.05*12     0
 15           17.86*12      63.05*12     0
 16           99*12         63.05*12     0
 17            7.98*12      54.3*12      90
 18           17.86*12      54.3*12      90
 19           39.19*12      54.3*12      90
 20           60.52*12      54.3*12      90
 21           81.85*12      54.3*12      90
 22           99*12         54.3*12      90
 23           99*12         45.22*12     0
```

24	7.98*12	36.14*12	0
25	17.86*12	36.14*12	90
26	99*12	36.14*12	0
27	7.98*12	17.98*12	0
28	17.86*12	17.98*12	90
29	39.19*12	17.98*12	90
30	60.52*12	17.98*12	90
31	76*12	17.98*12	90
32	86*12	17.98*12	90
33	7.98*12	0	0
34	17.86*12	0	0
35	39.19*12	0	0
36	60.52*12	0	0
37	81.85*12	0	0
38	90.39*12	0	0
39	99*12	0	0
40	81.85*12	25.56*12	0
41	90.39*12	25.56*12	0
42	99*12	25.56*12	0
43	0	93*12	0
44	0	71.8*12	0
45	0	0	0
46	0	54.3	
47	0	36.14	
48	0	25.56	

$

Bay Connectivity - Frame #1

1	1	2	26	3334
2	2	3	27	3435
3	3	4	28	3536
4	4	5	29	3637
5	5	6	30	3738
6	21	11	31	3839
7	7	8	32	3327
8	8	9	33	2724
9	9	10	34	2417
10	10	11	35	1714
11	11	12	36	14 7
12	12	13	37	2825
13	17	18	38	2518
14	18	19	39	1815
15	19	20	40	15 8
16	20	21	41	8 1
17	21	22	42	3529
18	24	25	43	11 4
19	27	28	44	3942
20	28	29	45	4226
21	29	30	46	2623
22	30	31	47	2322
23	31	32	48	2216
24	40	41	49	1613
25	41	42	50	13 6

$ Beam Span Vertical Loading Patterns

```
1  0    0  03.0
2  0    0  020.4
3  0    0  06.1
4  0    0  018.7
5  0    0  4.90
6  0    0  15.10
7  0    0  2.50
8  00  29.1  0
9  00  23.3  0
10 00  24.0  0
11 0    0  0 19.2
12 0    0  0 13.3
13 0    0  0 14.1
14 00  14.9  0
15 00  30.6  0
16 00  14.0  0
17 00  28.7  0
18 0    0  0  6.2
19 0    0  0 19.1
20 00  11.5  0
21 00  21.9  0
22 0    0  0  6.6
23 0    0  0 17.4
24 0    0  0  3.8
25 0    0  0 10.5
26 0    0  5.80
27 00  14.2  0
28 0    0  0  5.4
29 0    0  0 25.4
30 0    0  0 28.3
31 0    0  5.40
32 0    0  22.80
```

$Column Location Data

```
1  0  4th 1   1  0
1  0  2nd     1  01
1  0  Base    15 00
2  1
3  1
4  0  Roof    1  10
4  0  3rd 4   0  0
4  0  2nd     4  01
4  0  Base    15 00
$ Column 5 is a dummy column (zero properties)
6  4
7  0  Roof    1  20
7  0  2nd     1  01
7  0  Base    14 00
8  7
9  0  Roof    1  20
9        02nd101
9        0Base1300
10       9
11       0Roof120
11       02nd101
11       0Base1000
$ Column 12 is a dummy column
13       4
14       04th203
$ Columns 15 & 16 are dummy columns
17       7
18       7
19       0Roof120
19       02nd101
19       01st800
19       0Base900
20       0Roof120
20       02nd101
20       01st800
20       0Base1000
21       0Roof120
21       02nd101
21       01st600
21       0Base1100
22       0Roof310
22       03rd100
22       02nd101
$ Column 23 is a dummy column
24       04th110
24       02nd101
24       01st600
24       0Base1300
25       0Roof120
25       02nd101
25       01st600
25       0Base1200
26       4
27       24
28       0Roof120
28       02nd101
28       01st600
28       0Base1200
29       19
30       19
31       0Roof120
31       02nd101
31       01st500
31       0Base1300
32       31
33       03rd100
```

```
33 0    2nd     1    01
33 0    Base    15   00
34 33
35 4
36 4
37 4
$ Column 38 is a dummy column
39 4
40 4
41 0    Roof    2    03
42 4
$ Columns 43 thru 45 are dummy columns
46 0    Base    15   00
47 46
48 46

$                         Beam Location Data
1  0    4th 2   2    3
2  1
3  1
4  0    Roof    5    00
4  0    4th 2   2    1
5  0    Roof    5    00
5  0    4th 2   2    2
6  0    Roof    2    43
7  0    Roof    2    33
8  0    Roof    2    33
8  0    Base    7    00
9  8
10 7
11 0    Roof    1    01
11 0    4th 2   2    1
11 0    Base    8    00
12 0    Roof    1    02
12 0    4th 2   2    2
12 0    Base    8    00
13 0    Roof    2    33
13 0    Base    6    00
14 6
15 6
16 6
17 6
18 6
19 6
20 6
21 6
22 6
23 6
24 0    Roof    3    01
24 0    4th 2   2    1
24 0    Base    9    00
25 0    Roof    3    02
```

```
25      04th222
25      0Base900
26      03rd213
27      26
28      7
29      7
30      0Roof100
30      04th200
30      03rd401
30      02nd201
31      0Roof100
31      04th200
31      03rd402
31      02nd202
32      26
33      0Roof243
34      33
35      0Roof100
35      04th221
35      0Base600
36      0Roof100
36      04th222
36      0Base600
37      6
38      6
39      0Roof101
39      04th421
39      0Base600
40      0Roof102
40      04th422
40      0Base600
41      04th223
42      6
43      6
44      7
45      7
46      0Roof101
46      04th421
47      0Roof102
47      04th422
48      46
49      47
50      7

$       Brace Location Data
$       - Brace VB1
1       Roof2326133
5       Roof2322133
9       Roof1622133
13      Roof1613133
$       - Brace VB2
17      4th54123
```

```
17        4th   5    4    1    2    3
20        4th   5    6    1    2    3
$                                 -
Brace VB3
23        Roof  12   11   1    3    3
27        Roof  12   13   1    3    3
$                                 -
Brace VB4
31        Roof  41   40   1    3    3
35        Roof  41   42   1    3    3
$                                 -
Brace VB5
39        3rd   38   37   1    1    3
41        3rd   38   39   1    1    3
$                                 -
Brace VB6
43        4th   14   17   1    2    3
46        4th   14   7    1    2    3
$                                 -
Brace VB7
49        Roof  18   8    1    0    3
50        4th   15   18   1    2    3
53        4th   15   8    1    2    3
$
Panel Location Data
 1        1st   43   1    3    0
 2        1st   1    2    3    0
 3        1st   2    3    3    0
 4        1st   3    4    3    0
 5        1st   4    5    3    0
 6        1st   5    6    3    0
 7        1st   44   7    3    0
 8        1st   7    8    3    0
 9        1st   8    9    3    0
 10  1st   9    10   3    0
 11  1st   11   12   3    0
 12  1st   12   13   3    0
 13  1st   17   18   3    0
 14  1st   45   33   3    0
 15  1st   33   34   3    0
 16  1st   34   35   3    0
 17  1st   35   36   3    0
 18  1st   36   37   3    0
 19  1st   37   38   3    0
 20  1st   38   39   3    0
 21  1st   40   41   3    0
 22  1st   41   42   3    0
 23  1st   45   48   1    0
 24  1st   48   47   1    0
 25  1st   17   14   3    0
 26  1st   14   7    3    0
 27  1st   18   15   3    0
 28 1st 15 830
 29 1st 394220
 30 1st 422630
 31 1st 262330
 32 1st 232230
 33 1st 221630
 34 1st 161330
 35 1st 13630
 36 1st 474610
 37 1st 464410
 38 1st 444310
$     Assignment of Beam Span Vertical
Loading Patterns to Beams
 49     0Roof1000
 49     04th 2002
 47     0Roof3000
 47     04th 4002
 46     0Roof5000
 46     04th 6002
 4      0Roof7000
 4      04th 8000
 4      03rd 9000
 4      02nd10000
 5      04th11000
 5      03rd12000
 5      02nd13000
 11     0Roof14000
 11     04th15002
 24     0Roof16000
 24     04th17002
 25     0Roof18000
 25     04th19002
 30     0Roof20000
 30     04th21002
 31     0Roof22000
 31     04th23002
 36     0Roof24000
 36     04th25002
 35     0Roof26000
 35     04th27002
 40     0Roof28000
 40     04th29000
 40     03rd30001
 39     0Roof31000
 39     04th32002

$      Frame Location Data
 1  0  0  0  0  Frame #1/ K-Braced Frame
$      Response Spectrum
$      - N-S Direction Only
Response Spectrum - Peninsula Office Bldg
(0-Deg)
```

0	.202
.04	.204
.05	.211
.06	.215
.08	.211
.10	.248
.12	.261
.14	.292
.16	.365
.18	.468
.20	.443
.22	.355
.24	.346
.26	.457
.28	.408
.30	.390
.32	.391
.34	.389
.36	.459
.38	.527
.40	.523
.50	.404
.60	.302
.70	.220
.80	.236
.90	.224
1.00	.237
1.10	.280
1.20	.201
1.30	.131
1.40	.110
1.50	.126
2.00	.151

$ Load Case #1

$ - Response Spectrum + Loading on Columns at Brace Locations

1 0 1 0 0 0 0 1 0 0

APPENDIX C
PROGRAM FOR MODIFICATION OF GROUND MOTION FOR SOIL EFFECTS

C.1 Scope

In the research described here, a ground motion recorded on a rock outcrop was modified for soil effects, so as to estimate the actual earthquake motion experienced by a structure located on an adjacent soil deposit (Fig. C.1). This process was used in the case of the 2 Alhambra building which was located on a sand fill while the available ground motion was recorded on a rock outcrop. The process was carried out using the program TIMOD.FOR [18, 27], whose mathematical basis and use are reviewed here.

C.2 Mathematical Basis of Program

Assumptions:

a) The soil is linear, elastic, and horizontally layered

b) The ground motion consists of vertically propagating shear waves

The model is shown in Fig. C.2.

The governing differential equation for vertically propagating shear waves is given by

$$Gv'' = \rho\ddot{v} \tag{C.1}$$

where

G = Shear modulus of the soil layer

v = Horizontal displacement of a soil particle

$$v'' = \frac{d^2v}{dz^2}$$

$$\ddot{v} = \frac{d^2v}{dt^2}$$

$$but \quad \ddot{v} = -\omega^2 v$$

ω = Frequency

therefore Eq.(C.1) can be written as

$$Gv'' = -\rho\omega^2 v$$

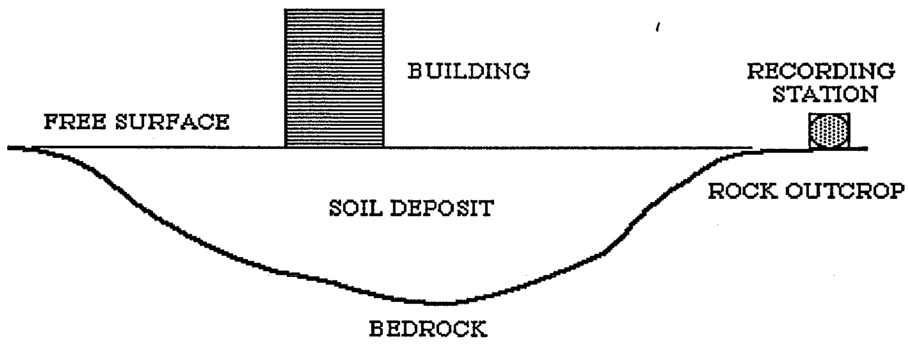

Fig. C.1 Location of the building and recording instrument

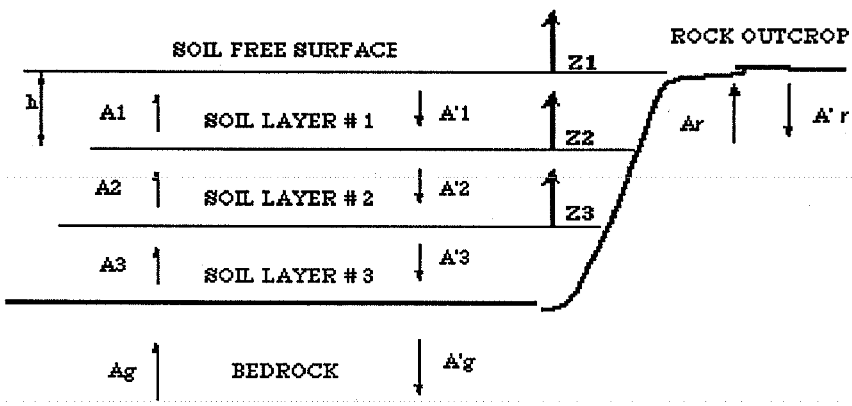

Fig. C.2 Soil model used in the program TIMOD.FOR or

$$v'' = \frac{-\omega^2 v}{\frac{G}{\rho}} \qquad \text{(C.2)}$$

where

ρ = Mass density of the soil

define

$$C_s^2 = \frac{G}{\rho}$$

$$\alpha^2 = \left[\frac{\omega}{C_s}\right]^2$$

where

C_s = Shear wave velocity

α = Wave number

therefore Eq. (C.2) can be written as

$$v'' + \alpha^2 v = 0$$

the solution of the above equation is of the form

$$v = \left(Ae^{-\iota\alpha z} + A' e^{\iota\alpha z}\right) e^{\iota\omega t} \qquad \text{(C.3)}$$

$$v' = \iota\alpha\left(-Ae^{-\iota\alpha z} + A'^{\iota\alpha z}\right) \qquad \text{(C.4)}$$

where A and A' are the amplitudes of propagated and reflected shear waves respectively.

Shear stress is given by τ = Gv'. Therefore at any level,

$$\tau(z) = \iota G\alpha(-A + A') \qquad \text{(C.5)}$$

at any depth h

Z = -h

$$v(-h) = Ae^{\iota\alpha h} + A'e^{-\iota\alpha h}$$

$$\tau(-h) = \iota G\alpha(-Ae^{\iota\alpha h} + A'e^{-\iota\alpha h})$$

At the free surface $Z_1 = 0$. Therefore

$$v(0) = A + A'$$

and $\tau_1(0) = 0$

therefore from Eq. (C.5)

$$A_1 = A'_1$$

and $v_1(0) = 2A_1$

Imposing compatibility of displacements at the junction of layers 1 and 2

$$v_2(0) = v_1(-h)$$

Therefore,

$$A_2 + A'_2 = A_1 e^{\iota\alpha 1 h1} + A'_1 e^{-\iota\alpha 1 h1} \tag{C.6}$$

Also, equilibrium of shear stresses must hold:

$$\tau_2(0) = \tau_1(-h)$$

therefore

$$\iota G_2\alpha_2(-A_2 + A'_2) = \iota G_1\alpha_1(-A_1 e^{\iota\alpha 1 h1} + A'_1 e^{-\iota\alpha 1 h1})$$

and

$$-A_2 + A'_2 = \frac{G_1 \alpha 1}{G_2\alpha_2}\left(-A_1 e^{\iota\alpha 1 h1} + A'_1 e^{-\iota\alpha 1 h1}\right) \tag{C.7}$$

let

$$K = \frac{G_1\alpha_1}{G_2\alpha_2}$$

Simultaneous solution of Eqs. (C.6) and (C.7) gives

$$A_2 = \frac{1}{2} A_1 e^{\iota\alpha 1 h1}(1 + K) + \frac{1}{2} A'_1 e^{-\iota\alpha 1 h1}(1 - K)$$

$$A'_2 = \frac{1}{2} A_1 e^{\iota\alpha 1 h1}(1 - K) + \frac{1}{2} A'_1 e^{-\iota\alpha 1 h1}(1 + K)$$

Now that A_2 and A'_2 are known, A_3 and A'_3 can be calculated in terms of A_2 and A'_2, and hence in terms of A_1 and A'_1. Therefore for the bedrock A_g and A'_g can be expressed as functions of A_1 and A'_1.

Since the damping in the rock is negligible, the amplitude of the shear wave in the rock outcrop will be the same as in the bedrock. Therefore, for the rock outcrop. That is,

$$A_r = A_g$$

For rock outcrop

$$\tau_r(0) = 0$$

hence

$$v_r = 2A_r$$

let

$$A_r = X\,A_1$$

The amplification of the amplitude of the horizontal displacement at the free surface, with respect to the ground motion on the rock outcrop, is given by:

$$\frac{v_1}{v_r} = \frac{2A_1}{2A_r} = \frac{1}{X}$$

Amplification in ground acceleration at each frequency will also be in the same proportion. Therefore this ratio, multiplied by the ground accelerations recorded on the rock free surface, will give the accelerations on the soil free surface. This analysis must be repeated for each frequency. A direct Fourier transform of the amplified motion must be performed to get the time history of the desired seismic motion.

Damping in the system is assumed to be hysteretic and independent of frequency. In order to introduce damping, G^* and C_s^* are used instead of G and C_s respectively, where asterisks imply complex numbers. The relationships for G^* and C_s^* are given below.

$$G^* = G\sqrt{1 + 2\imath D}$$

$$C_s^* = C_s(1 + \imath D)$$

where

D = Damping ratio

C.4 Description of Program

The program requires two input files. The first file contains information about the properties of the soils in various layers; and the second input file consists of the ground accelerations recorded on the rock outcrop. The program generates two output files, Soil.ech and Timod.ijk. The first file is an echo of the first input file, and the second output file contains the modified ground acceleration records. The reason for this arrangement is that this program is part of a package that would modify the ground motion for soil effects and then generate spectra for the modified ground motion. For the same reason, some of the input data required in the second input file are not used in this program.

C.5 Input Files

The two input files required by the program are described in this section. Both of the files can be in free format.

C.5.1 INPUT FILE No. 1: This file contains information about soils. All the quantities are required in the units of feet, pounds (force) and seconds. The format of the file is as follows.

Line No.1	Number of soil layers.
Line No. 2 (One for each layer)	Thickness; Specific Weight; Shear modulus; Damping in Soil Layer.

The following input file that was used for the 2 Alhambra building.

SAMPLE FOR INPUT FILE NO. 1

```
4,
40.,80.,224000.,0.07
40.,100.,280000.,0.07
180.,110.,307600.,0.07
100.,150.,4.24E10,0.0
```

The last layer in the above input file is always bedrock whose depth does not affect the results and which can be any arbitrary number.

C.5.2 INPUT FILE No. 2: In the CSMIP ground motion records the units used for acceleration are cm/s^2, therefore for the sake of compatibility, the units for acceleration in the input file are also the same and the program itself changes them to the units of in./sec^2. The format is given below:

Line No. 1	Title (up to 80 characters)
Line No. 2	Time Increment; Number of Records; Structural Damping

Line No. 3 (As many cards as there are records.)	Acceleration

The structural damping required here is not used in this program. However, this value is passed on to the output file which can then be used directly for the generation of spectra. A portion of the input file that was used in the case of 2 Alhambra is given below.

SAMPLE FOR INPUT FILE NO. 2:
THE N45E COMPONENT OF N00E AND N90E RECORDS

```
.02,     2000,     .05,
   .742
   .686
   1.176
   -.255
   -.707
   -.366
   -2.116
   -1.470
```

And so on until the last record.

C.6 SOURCE CODE OF PROGRAM TIMOD.FOR

```
PROGRAM TIMOD.FOR
C....PROGRAM TO MODIFY TIME-HISTORY ON ROCK OUTCROP
C.....FOR SOIL EFFECTS.  LINEAR SOILS CONSIDERED.
C
C***********************************************************  IMPLICIT
REAL*8(A-H,O-Z),INTEGER*4(I-N)
      CHARACTER*15 EQ,SOIL
      CHARACTER*1 INFO(80)
      COMPLEX*16 GL1(50),AUX,G(50),CON
      COMPLEX*16 E1,E2,TMAT(2,2),DMAT(2,2),EM(2,2)
      COMPLEX*16 CEEE(2050),TF(200),AA
      DIMENSION HH(50),W(50),GS(50),AT(50),DAUX(2)
      DIMENSION EEE(4100),FR(200)
      EQUIVALENCE (AUX,DAUX(1))
      EQUIVALENCE (EEE(1),CEEE(1))
      NPP=4096
      NP2=NPP/2
      WRITE(*,*) ' ENTER FILE NAME FOR SOIL PROFILE '
      READ(*,1000) SOIL
      WRITE (*,*) ' ENTER FILE NAME FOR EQ. TIME-HISTORY'
      READ (*,1000) EQ
1000  FORMAT (A)
      OPEN(1,FILE=SOIL,STATUS='OLD')
      READ(1,*)  NLAY
      READ(1,*) (HH(I),W(I),GS(I),AT(I),I=1,NLAY)
      CLOSE(1)
C.    WRITING INPUT TO THE ECHO FILE
      OPEN (4,FILE='SOIL.ECH',STATUS='UNKNOWN')
      WRITE (4,1800)
1800  FORMAT
(6X,'LAYER',6X,'HEIGHT',3X,'SP.WEIGHT',3X,'SHEAR MOD.'
   +  ,7X,'DAMPING')
      DO 222 IE=1,NLAY
222   WRITE (4,223) IE,HH(IE),W(IE),GS(IE),AT(IE)
223   FORMAT (1X,I10,2(2X,F10.2),2X,E10.4,2X,F10.2)
      DO 10 I=1,NLAY
      W(I)=W(I)/32.17
      DAUX(1)=1.
      DAUX(2)=2.*AT(I)
10    G(I)=GS(I)*AUX
      NL=NLAY-1
      DO 11 I=1,NLAY
11    GL1(I)=CDSQRT(G(I)/W(I))
      NOM=200
      OM=0.
      TF(1)=1.
      DOM=0.125
      DO 30 L=2,NOM
30    FR(L)=OM+(L-1)*DOM
      INDEX=0
      DO 50 L=1,NOM
      OM=6.28318*FR(L)
      DO 52 I=1,2
      DO 53 J=1,2
53    TMAT(I,J)=0.
52    TMAT(I,I)=1.
      DO 51 I=1,NL
      CON=OM/GL1(I)
      AUX=(0.,1.)*CON*HH(I)
      E1=CDEXP(AUX)
      E2=1./E1
      AA=W(I)*GL1(I)/(W(I+1)*GL1(I+1))
      DMAT(1,1)=E1*(1.+AA)/2.
      DMAT(1,2)=E2*(1.-AA)/2.
      DMAT(2,1)=E1*(1.-AA)/2.
      DMAT(2,2)=E2*(1.+AA)/2.
      DO 54 II=1,2
      DO 54 JJ=1,2
      AA=0.
      DO 56 KK=1,2
56    AA=AA+DMAT(II,KK)*TMAT(KK,JJ)
54    EM(II,JJ)=AA
      DO 57 II=1,2
      DO 57 JJ=1,2
57    TMAT(II,JJ)=EM(II,JJ)
51    CONTINUE
      AA=TMAT(1,1)+TMAT(1,2)
50    TF(L)=1./AA
      DO 55 J=1,4100
55    EEE(J)=0.0
      OPEN(1,FILE=EQ,STATUS='OLD')
      READ (1,1801) (INFO(I),I=1,80)
1801  FORMAT (80A1)
      READ (1,*) DT,NPTS,DMTS
```

```
      READ (1,*) (EEE(I),I=1,NPTS)
      CLOSE(1)
      WRITE (4,1560) EEE(NPTS)
1560  FORMAT (/,1X,'THE LAST RECORD IN THE TIME-HISTORY
INPUT WAS',
   +  F10.3)
      CLOSE (4)
C     CHANGING UNITS FRON CM/S^2 TO IN/S^2
      DO 1156 IU=1,NPTS
1156  EEE(IU)=EEE(IU)/(2.54)
      CALL FOUR2(EEE,NPP,1,-1,0)
      T=DT*NPP
      DF=1./T
      DO 80 I=1,NP2
      F=(I-1)*DF
      CALL INTPOL(AA,TF,FR,F,NOM)
80    CEEE(I)=AA*CEEE(I)
      CEEE(NP2+1)=0.
      CALL FOUR2(EEE,NPP,1,1,-1)
      OPEN(1,FILE='TIMOD.IJK',STATUS='UNKNOWN')
      WRITE (1,1801) (INFO(I),I=1,80)
      WRITE (1,*) DT,NPTS,DMTS
      DO 90 I=1,NPTS
      EEE(I)=EEE(I)/NPP
90    WRITE(1,100) EEE(I)
100   FORMAT(2X,F10.3)
      CLOSE(1)
      END
C****************************************************
      SUBROUTINE INTPOL(AA,V,FR,F,NOF)
      IMPLICIT REAL*8(A-H,O-Z),INTEGER*4(I-N)
      DIMENSION  FR(1)
      COMPLEX*16 AA,V(1)
      IF(F.GT.FR(1)) GO TO 10
      AA=V(1)
      GO TO 20
10    F(F.GE.FR(NOF)) GO TO 12
      DO 11 I=2,NOF
      IF(F.GT.FR(I)) GO TO 11
       AA=V(I-1)+(V(I)-V(I-1))*(F-FR(I-1))/(FR(I)-FR(I-1))
      GO TO 20
11    CONTINUE
12    AA=0.
20    RETURN
      END
C****************************************************
      SUBROUTINE FOUR2 (DATA,N,NDIM,ISIGN,IFORM)
      IMPLICIT REAL*8(A-H,O-Z),INTEGER*4(I-N)
      DIMENSION DATA(1), N(1)
      NTOT=1
      DO 10 IDIM=1,NDIM
10    NTOT=NTOT*N(IDIM)
      IF (IFORM) 70,20,20
20    NREM=NTOT
      DO 60 IDIM=1,NDIM
      NREM=NREM/N(IDIM)
      NPREV=NTOT/(N(IDIM)*NREM)
      NCURR=N(IDIM)
      IF (IDIM-1+IFORM) 30,30,40
30    NCURR=NCURR/2
40    CALL BITRV (DATA,NPREV,NCURR,NREM)
      CALL COOL2 (DATA,NPREV,NCURR,NREM,ISIGN)
      IF (IDIM-1+IFORM) 50,50,60
50    CALL FIXRL (DATA,N(1),NREM,ISIGN,IFORM)
      NTOT=(NTOT/N(1))*(N(1)/2+1)
60    CONTINUE
      RETURN
70    NTOT=(NTOT/N(1))*(N(1)/2+1)
      NREM=1
      DO 100 JDIM=1,NDIM
      IDIM=NDIM+1-JDIM
      NCURR=N(IDIM)
      IF (IDIM-1) 80,80,90
80    NCURR=NCURR/2
      CALL FIXRL (DATA,N(1),NREM,ISIGN,IFORM)
      NTOT=NTOT/(N(1)/2+1)*N(1)
90    NPREV=NTOT/(N(IDIM)*NREM)
      CALL BITRV (DATA,NPREV,NCURR,NREM)
      CALL COOL2 (DATA,NPREV,NCURR,NREM,ISIGN)
100   NREM=NREM*N(IDIM)
      RETURN
      END
C****************************************************
      SUBROUTINE BITRV (DATA,NPREV,N,NREM)
      IMPLICIT REAL*8(A-H,O-Z),INTEGER*4(I-N)
C     SHUFFLE THE DATA BY 'BIT REVERSAL'.
C     DIMENSION DATA(NPREV,N,NREM)
C     DATA(I1,I2REV,I3) = DATA(I1,I2,I3), FOR ALL I1 FROM 1 TO
```

```
NPREV,
C           ALL I2 FROM 1 TO N (WHICH MUST BE A POWER OF
TWO), AND ALL I3
C           FROM 1 TO NREM, WHERE I2REV-1 IS THE BITWISE
REVERSAL OF I2-1.
C           FOR EXAMPLE, N = 32, I2-1 = 10011 AND I2REV-1 = 11001.
            DIMENSION DATA(1)
            IP0=2
            IP1=IP0*NPREV
            IP4=IP1*N
            IP5=IP4*NREM
            I4REV=1
            DO 60 I4=1,IP4,IP1
            IF (I4-I4REV) 10,30,30
 10         I1MAX=I4+IP1-IP0
            DO 20 I1=I4,I1MAX,IP0
            DO 20 I5=I1,IP5,IP4
            I5REV=I4REV+I5-I4
            TEMPR=DATA(I5)
            TEMPI=DATA(I5+1)
            DATA(I5)=DATA(I5REV)
            DATA(I5+1)=DATA(I5REV+1)
            DATA(I5REV)=TEMPR
 20         DATA(I5REV+1)=TEMPI
 30         IP2=IP4/2
 40         IF (I4REV-IP2) 60,60,50
 50         I4REV=I4REV-IP2
            IP2=IP2/2
            IF (IP2-IP1) 60,40,40
 60         I4REV=I4REV+IP2
            RETURN
            END
C******************************************************
            SUBROUTINE COOL2 (DATA,NPREV,N,NREM,ISIGN)
            IMPLICIT REAL*8(A-H,O-Z),INTEGER*4(I-N)
C           OURIER TRANSFORM OF LENGTH N BY THE
COOLEY-TUKEY CA LGORITHM.
C           BIT-REVERSED TO NORMAL ORDER.
C           DIMENSION DATA(NPREV,N,NREM)
C           COMPLEX DATA
C           DATA(I1,J2,I3) =
SUM(DATA(I1,I2,I3)*EXP(ISIGN*2*PI*I*((I2-1)*
C           (J2-1)/N))), SUMMED OVER I2 = 1 TO N FOR ALL I1 FROM
1 TO NPREV,
C           J2 FROM 1 TO N AND I3 FROM 1 TO NREM.  N MUST BE A
POWER OF
C           TWO.
C           FACTORING N BY FOURS SAVES ABOUT TWENTY FIVE
PERCENT OVER C.     FACTOR-
C           ING BY TWOS.
C           NOTE--IT IS NOT NECESSARY TO REWRITE THIS
SUBROUTINE INTO
C           COMPLEX
C           NOTATION SO LONG AS THE FORTRAN COMPILER USED
STORES REAL
C           AND
C           MAGINARY PARTS IN ADJACENT STORAGE LOCATIONS.
IT MUST ALSO
C           STORE ARRAYS WITH THE FIRST SUBSCRIPT INCREASING
FASTEST.
            DIMENSION DATA(1)
            TWOPI=2.*(4.*DATAN(1.D0))*ISIGN
            IP0=2
            IP1=IP0*NPREV
            IP4=IP1*N
            IP5=IP4*NREM
            IP2=IP1
            NPART=N
 10         IF (NPART-2) 50,30,20
 20         NPART=NPART/4
            GO TO 10
C           DO A FOURIER TRANSFORM OF LENGTH TWO
 30         IP3=IP2*2
            DO 40 I1=1,IP1,IP0
            DO 40 I5=I1,IP5,IP3
            J0=I5
            J1=J0+IP2
            TEMPR=DATA(J1)
            TEMPI=DATA(J1+1)
            DATA(J1)=DATA(J0)-TEMPR
            DATA(J1+1)=DATA(J0+1)-TEMPI
            DATA(J0)=DATA(J0)+TEMPR
 40         DATA(J0+1)=DATA(J0+1)+TEMPI
            GO TO 140
C           DO A FOURIER TRANSFORM OF LENGTH FOUR (FROM BIT
REVERSED
C           ORDER)
 50         IP3=IP2*4
```

```
      THETA=TWOPI/(IP3/IP1)
      SINTH= DSIN(THETA/2.)
      WSTPR=-2.*SINTH*SINTH
C     COS(THETA)-1, FOR ACCURACY.
      WSTPI= DSIN(THETA)
      WR=1.
      WI=0.
      DO 130 I2=1,IP2,IP1
      IF (I2-1) 70,70,60
60    W2R=WR*WR-WI*WI
      W2I=2.*WR*WI
      W3R=W2R*WR-W2I*WI
      W3I=W2R*WI+W2I*WR
70    I1MAX=I2+IP1-IP0
      DO 120 I1=I2,I1MAX,IP0
      DO 120 I5=I1,IP5,IP3
      J0=I5
      J1=J0+IP2
      J2=J1+IP2
      J3=J2+IP2
      F (I2-1) 90,90,80
C     APPLY THE PHASE SHIFT FACTORS
80    TEMPR=DATA(J1)
      DATA(J1)=W2R*TEMPR-W2I*DATA(J1+1)
      DATA(J1+1)=W2R*DATA(J1+1)+W2I*TEMPR
      TEMPR=DATA(J2)
      DATA(J2)=WR*TEMPR-WI*DATA(J2+1)
      DATA(J2+1)=WR*DATA(J2+1)+WI*TEMPR
      TEMPR=DATA(J3)
      DATA(J3)=W3R*TEMPR-W3I*DATA(J3+1)
      DATA(J3+1)=W3R*DATA(J3+1)+W3I*TEMPR
90    T0R=DATA(J0)+DATA(J1)
      T0I=DATA(J0+1)+DATA(J1+1)
      T1R=DATA(J0)-DATA(J1)
      T1I=DATA(J0+1)-DATA(J1+1)
      T2R=DATA(J2)+DATA(J3)
      T2I=DATA(J2+1)+DATA(J3+1)
      T3R=DATA(J2)-DATA(J3)
      T3I=DATA(J2+1)-DATA(J3+1)
      DATA(J0)=T0R+T2R
      DATA(J0+1)=T0I+T2I
      DATA(J2)=T0R-T2R
      DATA(J2+1)=T0I-T2I
      IF (ISIGN) 100,100,110
100   T3R=-T3R
      T3I=-T3I
110   DATA(J1)=T1R-T3I
      DATA(J1+1)=T1I+T3R
      DATA(J3)=T1R+T3I
120   DATA(J3+1)=T1I-T3R
      TEMPR=WR
      WR=WSTPR*TEMPR-WSTPI*WI+TEMPR
130   WI=WSTPR*WI+WSTPI*TEMPR+WI
140   IP2=IP3
      IF (IP3-IP4) 50,150,150
150   RETURN
      END
C*********************************************************
      SUBROUTINE FIXRL (DATA,N,NREM,ISIGN,IFORM)
      IMPLICIT REAL*8(A-H,O-Z),INTEGER*4(I-N)
C     FOR IFORM = 0, CONVERT THE TRANSFORM OF A
DOUBLED-UP REAL
C     ARRAY,CONSIDERED COMPLEX, INTO ITS TRUE
TRANSFORM.  SUPPLY
C     ONLY THE FIRST HALF OF THE COMPLEX TRANSFORM,
AS THE
C     SECOND HALF HAS CONJUGATE SYMMETRY.  FOR IFORM
= -1
C     CONVERT THE FIRST HALF OF THE TRUE TRANSFORM
INTO THE
C     TRANSFORM OF A DOUBLED-UP REAL
C     ARRAY.  N MUST BE EVEN.
C     USING COMPLEX NOTATION AND SUBSCRIPTS STARTING
AT ZERO,
C     THE TRANSFORMATION IS--
C     DIMENSION DATA(N,NREM)
C     ZSTP = EXP(ISIGN*2*PI*I/N)
C     DO 10 I2=0,NREM-1
C     DATA(0,I2) = CONJ(DATA(0,I2))*(1+I)
C     DO 10 I1=1,N/4
C     Z = (1+(2*IFORM+1)*I*ZSTP**I1)/2
C     I1CNJ = N/2-I1
C     DIF = DATA(I1,I2)-CONJ(DATA(I1CNJ,I2))
C     TEMP = Z*DIF
C     DATA(I1,I2) = (DATA(I1,I2)-TEMP)*(1-IFORM)
C 10  DATA(I1CNJ,I2) =
(DATA(I1CNJ,I2)+CONJ(TEMP))*(1-IFORM)
C     IF I1=I1CNJ, THE CALCULATION FOR THAT VALUE
```

```
COLLAPSES INTO
C       A SIMPLE CONJUGATION OF DATA(I1,I2).
        DIMENSION DATA(1)
        TWOPI=2.*(4.*DATAN(1.D0))*ISIGN
        IP0=2
        IP1=IP0*(N/2)
        IP2=IP1*NREM
        IF (IFORM) 10,70,70
C       PACK THE REAL INPUT VALUES (TWO PER COLUMN)
10      J1=IP1+1
        III2=2
        DATA(III2)=DATA(J1)
        IF (NREM-1) 70,70,20
20      J1=J1+IP0
        I2MIN=IP1+1
        DO 60 I2=I2MIN,IP2,IP1
        DATA(I2)=DATA(J1)
        J1=J1+IP0
        IF (N-2) 50,50,30
30      I1MIN=I2+IP0
        I1MAX=I2+IP1-IP0
        DO 40 I1=I1MIN,I1MAX,IP0
        DATA(I1)=DATA(J1)
        DATA(I1+1)=DATA(J1+1)
40      J1=J1+IP0
50      DATA(I2+1)=DATA(J1)
60      J1=J1+IP0
70      DO 80 I2=1,IP2,IP1
        TEMPR=DATA(I2)
        DATA(I2)=DATA(I2)+DATA(I2+1)
80      DATA(I2+1)=TEMPR-DATA(I2+1)
        IF (N-2) 200,200,90
90      THETA=TWOPI/FLOAT(N)
        SINTH= DSIN(THETA/2.)
        ZSTPR=-2.*SINTH*SINTH
        ZSTPI= DSIN(THETA)
        ZR=(1.-ZSTPI)/2.
        ZI=(1.+ZSTPR)/2.
        F (IFORM) 100,110,110
100     ZR=1.-ZR
        ZI=-ZI
110     I1MIN=IP0+1
        I1MAX=IP0*(N/4)+1
        DO 190 I1=I1MIN,I1MAX,IP0
        DO 180 I2=I1,IP2,IP1
        I2CNJ=IP0*(N/2+1)-2*I1+I2
        IF (I2-I2CNJ) 150,120,120
120     IF (ISIGN*(2*IFORM+1)) 130,140,140
130     DATA(I2+1)=-DATA(I2+1)
140     IF (IFORM) 170,180,180
150     DIFR=DATA(I2)-DATA(I2CNJ)
        DIFI=DATA(I2+1)+DATA(I2CNJ+1)
        TEMPR=DIFR*ZR-DIFI*ZI
        TEMPI=DIFR*ZI+DIFI*ZR
        DATA(I2)=DATA(I2)-TEMPR
        DATA(I2+1)=DATA(I2+1)-TEMPI
        DATA(I2CNJ)=DATA(I2CNJ)+TEMPR
        DATA(I2CNJ+1)=DATA(I2CNJ+1)-TEMPI
        IF (IFORM) 160,180,180
160     DATA(I2CNJ)=DATA(I2CNJ)+DATA(I2CNJ)
        DATA(I2CNJ+1)=DATA(I2CNJ+1)+DATA(I2CNJ+1)
170     DATA(I2)=DATA(I2)+DATA(I2)
        DATA(I2+1)=DATA(I2+1)+DATA(I2+1)
180     CONTINUE
        TEMPR=ZR-.5
        ZR=ZSTPR*TEMPR-ZSTPI*ZI+ZR
190     ZI=ZSTPR*ZI+ZSTPI*TEMPR+ZI
C       RECURSION SAVES TIME, AT A SLIGHT LOSS IN
ACCURACY. IF
C       AVAIBLE USE DOUBLE PRECISION TO COMPUTE ZR AND
ZI.
200     IF (IFORM) 270,210,210
C       UNPACK THE REAL TRANSFORM VALUES (TWO PER
COLUMN)
210     I2=IP2+1
        I1=I2
        J1=IP0*(N/2+1)*NREM+1
        GO TO 250
220     DATA(J1)=DATA(I1)
        DATA(J1+1)=DATA(I1+1)
        I1=I1-IP0
        J1=J1-IP0
230     IF (I2-I1) 220,240,240
240     DATA(J1)=DATA(I1)
        DATA(J1+1)=0.
250     I2=I2-IP1
        J1=J1-IP0
        DATA(J1)=DATA(I2+1)
```

```
      DATA(J1+1)=0.
      I1=I1-IP0
      1=J1-IP0
      IF (I2-1) 260,260,230
260   III2=2
      DATA(III2)=0.
270   RETURN
      END
```

APPENDIX D
2 ALHAMBRA

D.1 Scope

This Appendix summarizes the steps used to derive the analytical model of the building, and also contains some of the analysis results related to the lateral response of the structure. In Figs. D.1 and D.2 are shown the node and bay numbering schemes used in the model.

D.2 Modelling Criteria

Material properties for the three principal construction materials used in the building are given in Table D.1 [5, 6].

Table D.1 Material properties (2 Alhambra)

MATERIAL	ν	E (ksi)	G (ksi)
WOOD	0.2	1500	625
PLASTER/STUCCO	0.17	2550	1090
MASONRY	0.4	1600	625

E = Young's Modulus of Elasticity
G = Shear Modulus
ν = Poisson's Ratio

It was assumed that at the time of the earthquake, the loads on the structure consisted of the self-weight of the building plus a 70% of the code-recommended value of live load for residential buildings [37].

D.2.1 Dead Loads: The following were estimated values for various types of dead loads on the structure [5].

<u>WALLS</u>:

Wood Studs (2- x 4-in) spaced at 12 inches	=	1.3 psf of wall area
Plaster / Stucco (0.25 in. thick)	=	10 psf of wall area
Wood Siding (1 in. thick)	=	2.5 psf of wall area
Masonry Veneer (4 in. thick)	=	38 psf of wall area

<u>BEAMS AND COLUMNS</u>:

Wood	=	35 pcf

<u>FLOOR DIAPHRAGM</u>:

Wooden Joists (2-x 12-in.) spaced at 16 inches	=	3.2 psf of floor area
0.25 in. finished floor over 1 in.sub-floor	=	4 psf of floor area

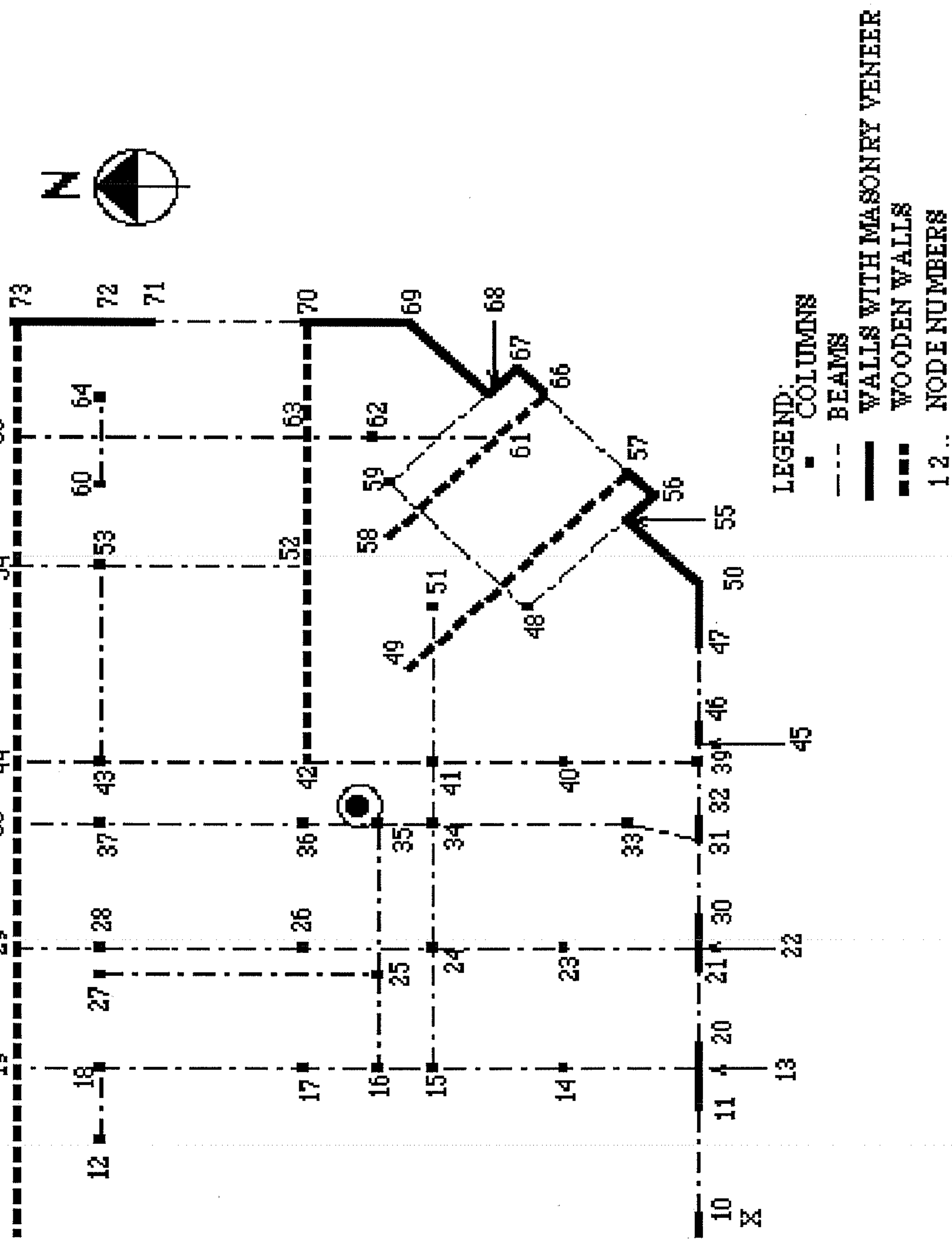

Figure D.1 Node number scheme (2 Alhambra)

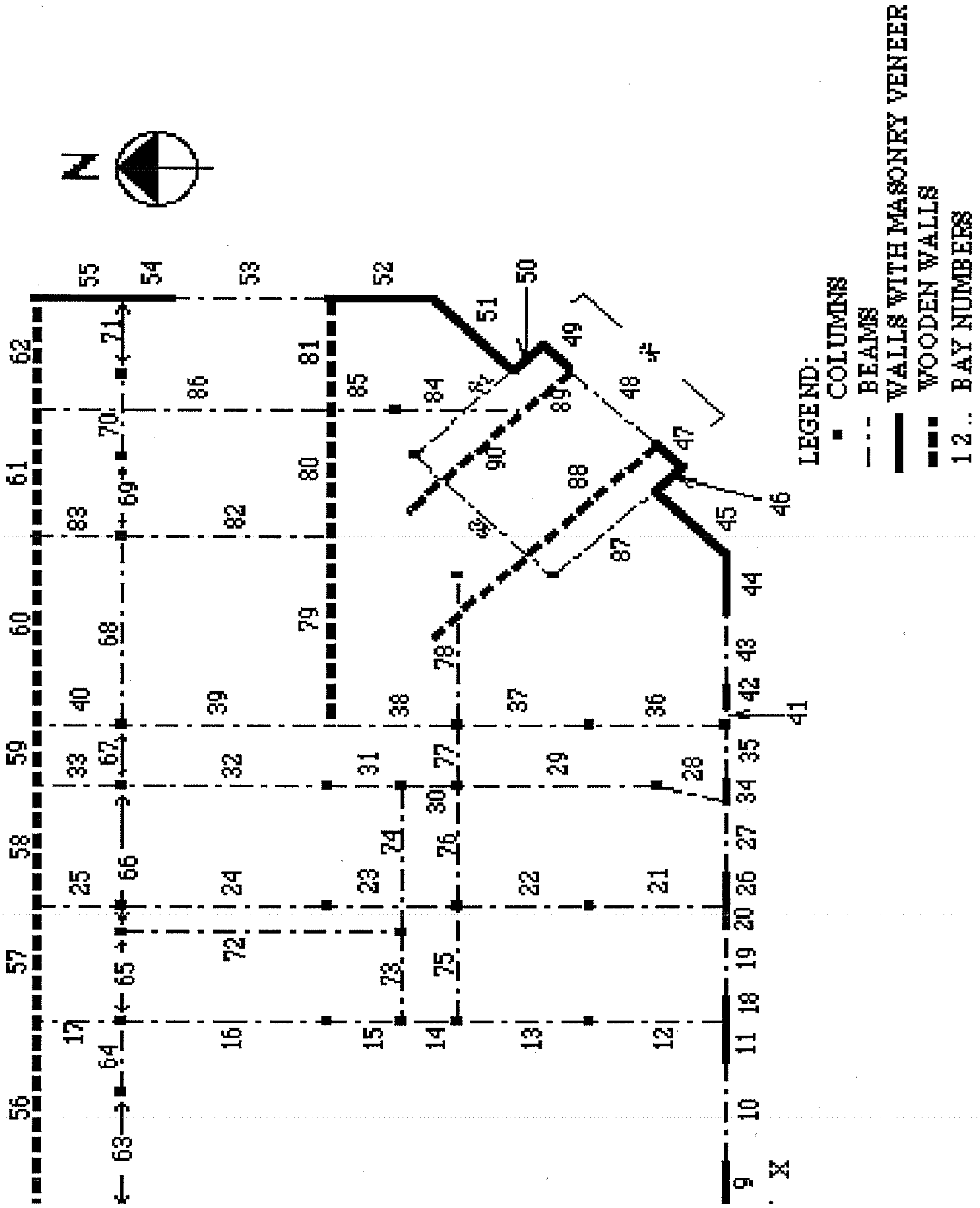

Figure D.2 Bay numbering scheme (2 Alhambra)

Suspended Ceiling	=	2 psf of floor area
Insulation (for roof only)	=	2 psf of roof area

D.2.2 Live loads: The live load on the lower three levels was assumed to be 70% of the code recommended live value [37]. This was taken as an average value of live load on the structure at the time of earthquake [16].

Actual Live Load = 0.7 x 40 = 28 psf of floor area

It was assumed that no live load was acting on the roof at the time of the earthquake.

D.3 Design Calculations

D.3.1 Mass Calculations: At each level, masses and rotary inertia were calculated using all members or portions thereof within half a story height above and below the floor diaphragm at that level. Resultants were calculated for these quantities and were assigned to a single floor master node at that level (Fig. D.1). Sample calculations for the mass and rotary inertia of roof diaphragm are given below:

As shown in Fig.D.3, the roof diaphragm was subdivided into four regions. The location of the center of mass is referenced from the lower left corner of the plan.

Load per square foot of roof area = 3.2 + 4.0 + 2.0 + 2.0 = 11.2 psf

Mass per square foot of roof area = 11.2/(32.18 x 12) = 0.029 $lb\text{-}sec^2/in$

<u>Area A.</u>

X_A = 406.5 in.

Y_A = 325 in.

Area = 3667.3 $ft.^2$

M_A = 3667.3 x 0.029 = 106.35 $lb\text{-}sec^2/in$

$$I_A = \frac{1}{12}\times 87.28\times[54.13^2 + 67.75^2] = 9597138\,lb\text{-}sec^2\text{-}in$$

Similar calculations were carried out for other regions; the results are given in Table D.2.

Table D.2 Sample mass calculations

Region	Location of Mass		M_i $lb\text{-}sec^2/in$	I_i $lb\text{-}sec^2\text{-}in$
	X_i (in)	Y_i (in)		
A	406.5	325	1063.35	9597138
B	956	483	19.23	309730
C	908	210	9.09	91590
D	980	137	1.78	6788

$\Sigma Mi \quad = \quad M_{c.g} \quad = \quad 136.44 \text{ lb - sec}^2/\text{in.}$

$\Sigma M_i X_i \quad = \quad 71607$

$\Rightarrow \quad X_{c.g} \quad = \quad 524.8 \text{ in.}$

$\Sigma M_i Y_i \quad = \quad 46003$

$\Rightarrow \quad Y_{c.g} \quad = \quad 337 \text{ in.}$

The total mass moment of inertia for the entire roof is given by

$$I_{c.g} = \sum_{A}^{D} I_i + r_i^2 M_i$$

where

$$r_i^2 = [X_i - X_{c.g}]^2 + [Y_i - Y_{c.g}]^2$$

Therefore,

$$I_{c.g} = 30654398 \text{ lb-sec}_2\text{-in.}$$

Similar mass calculations were performed for all four levels; the results are shown in Table D.3. All the distances are referenced from the lower left corner of the building (Fig. D.1).

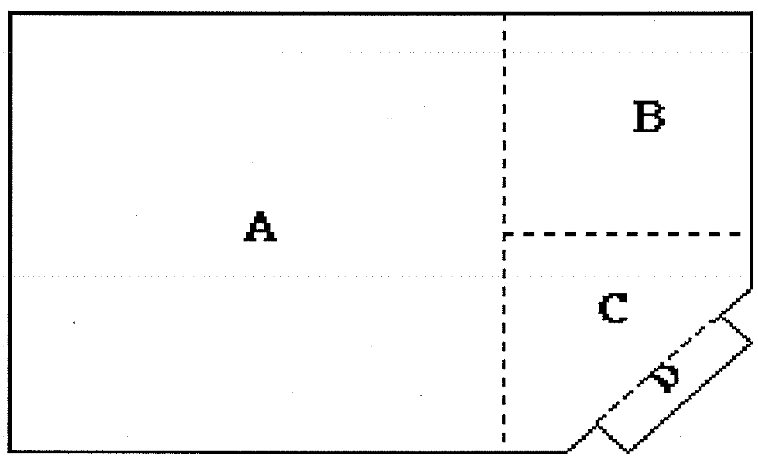

Fig. D.3 Subdivision of roof for mass calculations

Table D.3 Mass and rotary inertia for floor master node at each level (2 Alhambra)

Level	location of Center of Mass		M_i k-sec$_2$/in	I_i k-sec$_2$/in
	X_i (in.)	Y_i (in.)		
1	548	366	0.7915	120897
2	521	336	0.6996	90320
3	521	336	0.6996	90320
4	520	336	0.2439	31182

D.3.2 FOUNDATION SPRINGS: The effects of soil-foundation flexibility were incorporated into the model by attaching a set of six springs to the base of every column, and in case of wall panels, to each of the bottom two nodes at the wall corners. These springs represented translational stiffnesses (K_z, K_y and K_x) along and rotational stiffnesses ($K_{\theta x}$, $K_{\theta y}$, $K_{\theta z}$) about X, Y and Z axes.

The spread footing under each column was assumed to be 3 ft. square and the strip footing under the walls was assumed to be 1 ft. wide. The footings rest on medium dense sands; the following estimates were made for the properties of this type of soil [13].

C_s = 300 ft/sec
ρ = 3.42 lb-s/ft^2

ν = 0.3

$$G = \rho C_s^2 = 2.14 \frac{k}{in^2}$$

Using those soil properties, sample calculations are carried out below for a strip footing 93 inches long and 12 inches wide:

Following equations are used for calculation of K_z, K_y and K_x [9]:

$$K_z = 0.8\frac{2LG}{1 - \nu} \quad For \ \frac{A}{4L^2} < 0.02$$

$$K_y = 2.24\frac{2LG}{2 - \nu} \quad For \ \frac{A}{4L^2} < 0.16$$

$$K_x = K_y - \frac{0.21LG}{0.75 - \nu}\left[1 - \frac{B}{L}\right]$$

For the strip footing,

2L	=	93
2B	=	12
A	=	93 x 12 =1116

Using these values in above equations,

$$K_z = 150.73 \frac{k}{in^2}$$

$$K_x = 110.77 \frac{k}{in^2}$$

Similar calculations were done for all strip and spread footings, and stiffnesses were determined for the springs attached to various nodes at the base of the model. The results are shown in Table D.4. These values were used when the building was analyzed with all of its masonry intact. As the structure lost its masonry veneer, the contribution of the strip footings under the veneer panels also diminished, and hence some of these spring groups were revised later. The units used for the translational and rotational stiffnesses are k/in. and k-in respectively.

D.4 General Results of the Analyses

This section contains the analysis results that depict the lateral response of the structure under various combinations of static and dynamic loads.

Significant mode shapes in both the directions are shown in Figs.D.4 to D.7. Table D.5 contains the periods and mass contributions for various modes, for the case when the masonry was intact, and Table D.6 shows these quantities for the case when the masonry veneer was lost. Since the units used in the input files were kips and inches, all the quantities in the tables have the corresponding units.

Tables D.7 gives the lateral displacements in inches and rotations in radians at the floor master nodes for the case when the masonry veneer was intact. Table D.8 gives the same quantities for the case when the structure was analyzed without the masonry veneer.

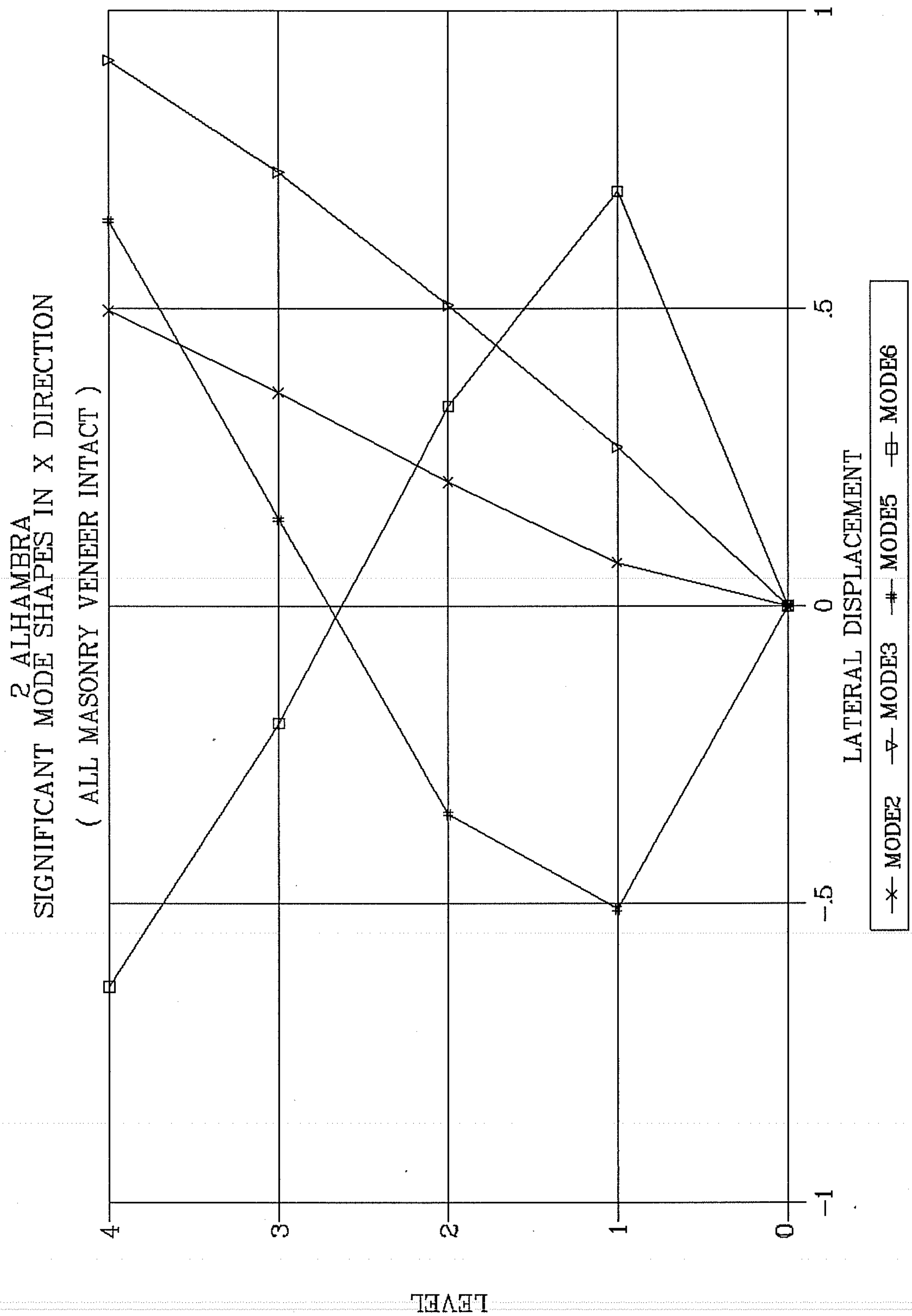

Figure D.4 Significant node shapes in X direction (masonry veneer intact)

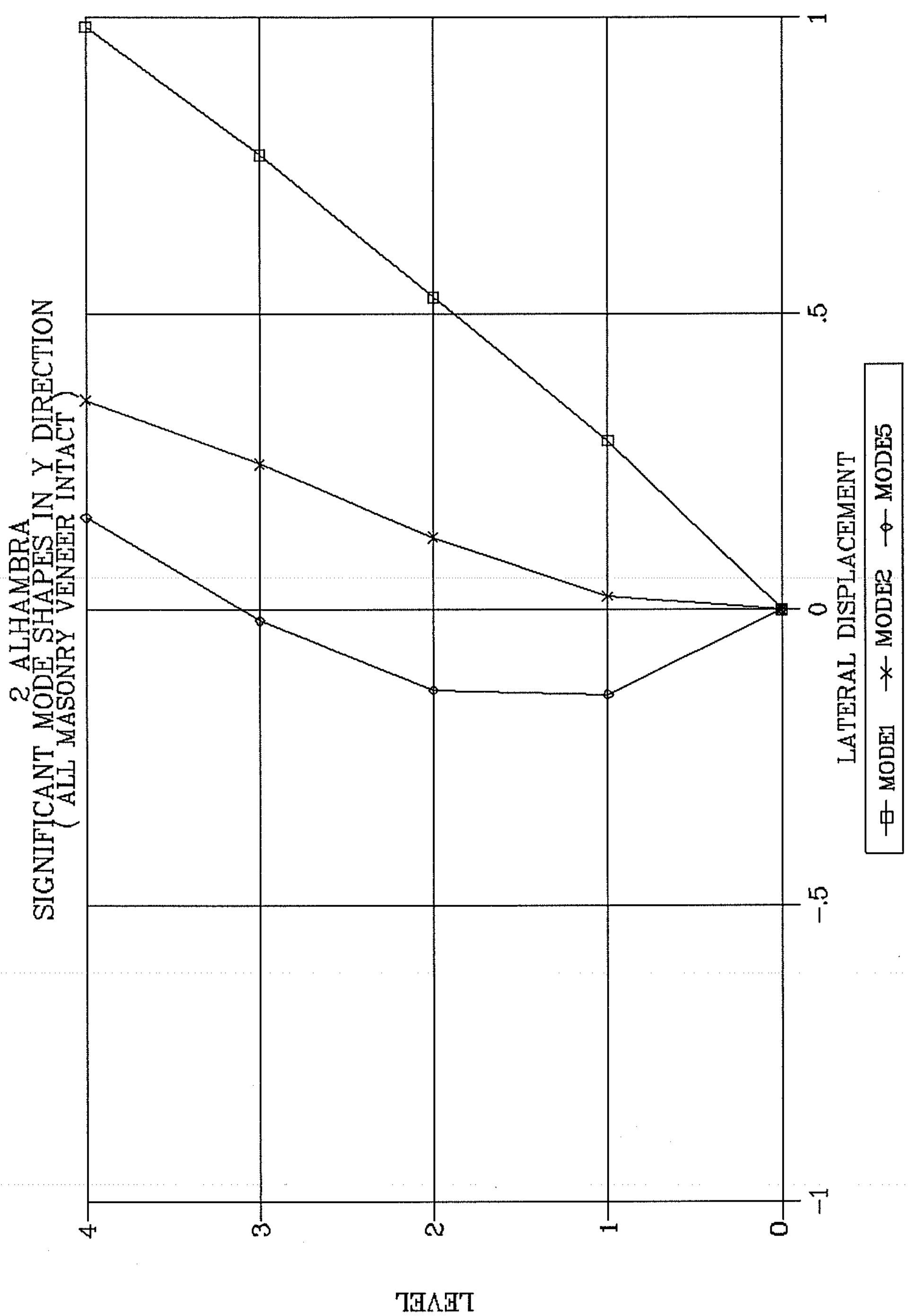

Figure D.5 Significant node shapes in Y direction (masonry veneer intact)

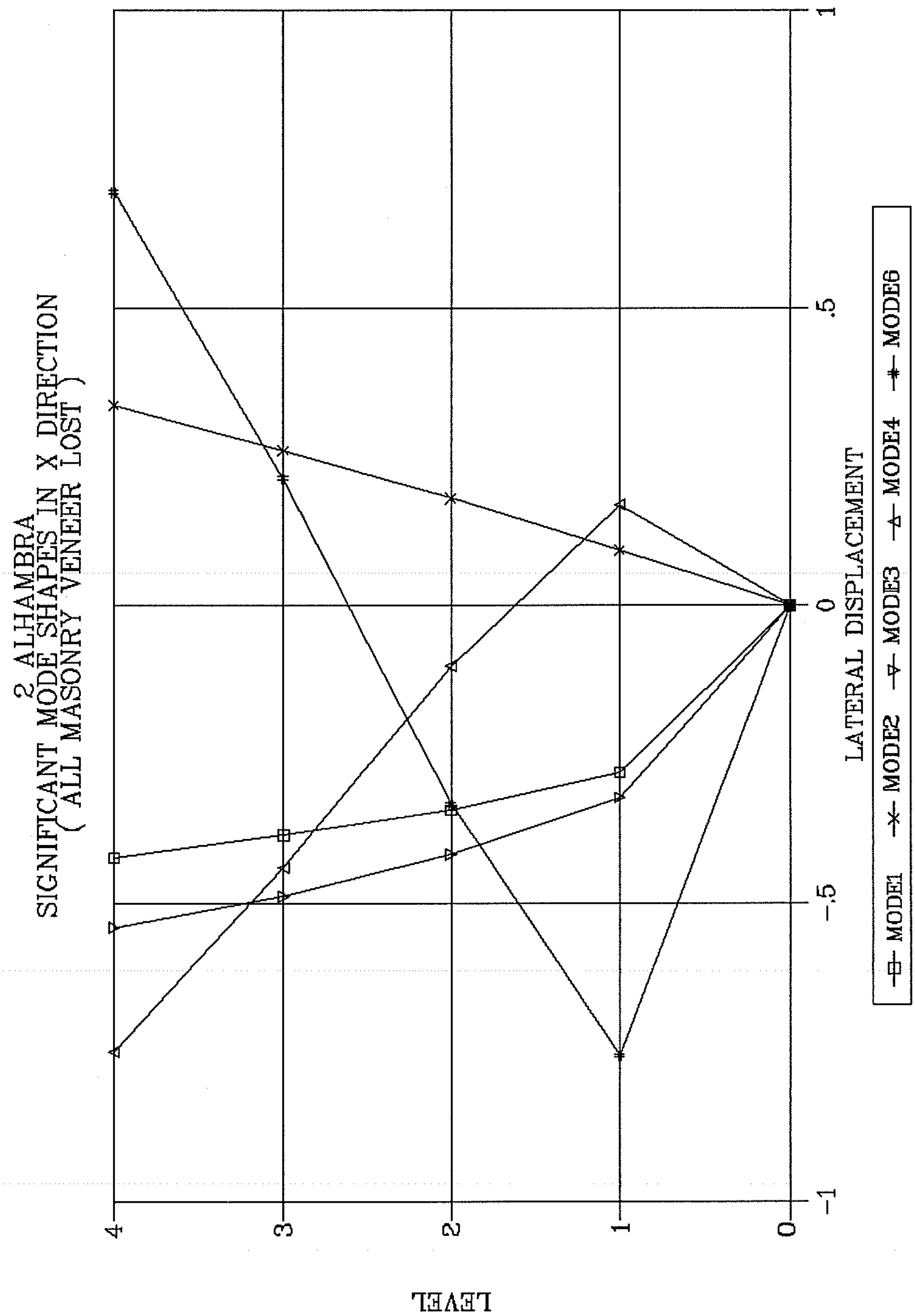

Figure D.6 Significant mode shapes in X direction (all masonry veneer lost)

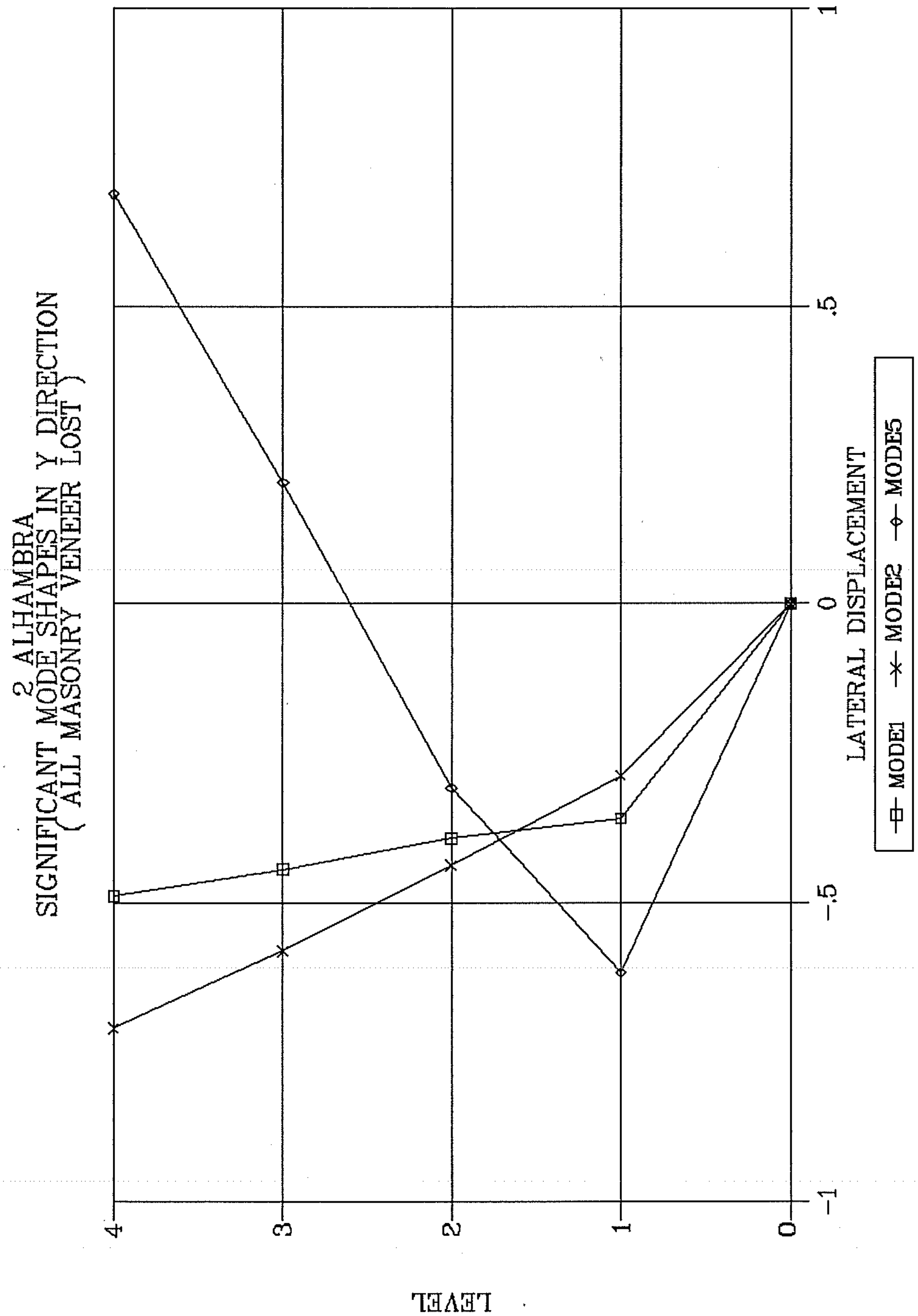

figure D.7 Significant mode shapes in Y direction (all masonry veneer lost)

Table D.4 Values for foundation springs for building with intact masonry

Node No.	K_x	K_y	K_z	$K_{\theta x}$	$K_{\theta y}$	$K_{\theta z}$
2, 12, 14, 23, 33, 34, 35, 36, 37, 39, 40, 41, 43, 48, 51, 53, 59, 60, 62, 64	1E+12	1E+12	325	0	0	1E+12
1, 10,	80	1E+12	109	1E+12	1E+12	1E+12
11	71	1E+12	93.04	1E+12	1E+12	1E+12
13	123	1E+12	158	1E+12	1E+12	1E+12
20, 21	52	1E+12	65	1E+12	1E+12	1E+12
22	110	1E+12	137	1E+12	1E+12	1E+12
30	58	1E+12	73	1E+12	1E+12	1E+12
31, 32, 45, 46	49	1E+12	60	1E+12	1E+12	1E+12
47	87	1E+12	123	1E+12	1E+12	1E+12
50	289	202	294	1E+12	1E+12	1E+12
55, 68	301	301	256	1E+12	1E+12	1E+12
56, 67	198	198	170	1E+12	1E+12	1E+12
57	720	720	530	1E+12	1E+12	1E+12
66	234	234	209	1E+12	1E+12	1E+12
69	202	335	347	1E+12	1E+12	1E+12
70	163	134	381	1E+12	1E+12	1E+12
71	1E+12	78	105	1E+12	1E+12	1E+12

(continued on the next page)

Table D.4 Values for foundation springs for building with intact masonry (continued)

Node No.	K_x	K_y	K_z	$K_{\theta x}$	$K_{\theta y}$	$K_{\theta z}$
8, 72	1E+12	187	254	1E+12	1E+12	1E+12
73	163	110	355	1E+12	1E+12	1E+12
65	348	1E+12	434	1E+12	1E+12	1E+12
54	459	1E+12	548	1E+12	1E+12	1E+12
44	363	1E+12	448	1E+12	1E+12	1E+12
38	259	1E+12	341	1E+12	1E+12	1E+12
29	343	1E+12	429	1E+12	1E+12	1E+12
19	447	1E+12	536	1E+12	1E+12	1E+12
9	274	109	469	1E+12	1E+12	1E+12
7	1E+12	324	396	1E+12	1E+12	1E+12
6	1E+12	246	292	1E+12	1E+12	1E+12
5	1E+12	85	119	1E+12	1E+12	1E+12
4	1E+12	196	270	1E+12	1E+12	1E+12
3	1E+12	110	150	1E+12	1E+12	1E+12
49	621	621	445	1E+12	1E+12	1E+12
61	458	458	377	1E+12	1E+12	1E+12
58	323	323	253	1E+12	1E+12	1E+12
42	274	1E+12	320	1E+12	1E+12	1E+12
52	459	1E+12	548	1E+12	1E+12	1E+12
63	348	1E+12	433	1E+12	1E+12	1E+12

Table D.5 Periods and participating modal masses for building without masonry

Mode No.	Per. (sec)	X Direction		Y Direction		Rotation	
		Modal mass	% of Total	Modal mass	% of Total	Modal mass	% of Total
1	0.328	4.28E-3	0.18	1.89E+0	77.43	2.46E+04	7.39
2	0.304	3.34E-01	13.71	1.31E-01	5.37	1.54E+05	46.25
3	0.233	1.69E+00	69.54	6.57E-03	0.27	3.73E+04	11.2
4	0.113	2.60E-04	0.01	3.28E-01	13.49	3.32E+03	1
5	0.089	1.48E-01	6.08	3.35E-02	1.38	6.83E+04	20.52
6	0.077	2.45E-01	10.07	3.48E-02	1.43	3.20E+04	9.62
7	0.048	2.33E-04	0.01	1.25E-02	0.52	9.99E+03	3
8	0.044	4.66E-04	0.02	1.72E-03	0.07	2.44E+03	0.73
9	0.036	8.35E-03	0.34	3.76E-04	0.02	1.41E+00	0
10	0.034	2.92E-05	0	6.19E-04	0.03	2.59E+02	0.08
Overall Mass		2.434E00	99.9	2.434E00	100	33.2E+6	99.8

Table D.6 Periods and participating modal masses for building without masonry

Mode No.	Period (sec)	X Direction		Y Direction		Rotation	
		Modal Mass	% of Total	Modal Mass	% of Total	Modal Mass	% of Total
1	0.702	7.03E-01	.702	9.78E-01	40.2	9.63E-01	28.
2	0.347	2.15E-01	8.83	1.24E+00	50.9	8.31E+04	24.
3	0.279	1.05E+00	43.04	3.34E-02	1.37	5.27E+04	15.85
4	0.178	1.82E-01	7.49	2.56E-02	1.05	4.51E+04	13.55
5	0.117	1.89E-02	0.77	1.55E-01	6.36	2.42E+04	7.29
6	0.085	2.62E-01	10.77	6.27E-05	0	2.92E+04	8.78
7	0.055	1.97E-03	0.08	8.52E-05	0	7.48E+02	0.22
8	0.045	6.64E-05	0	1.93E-03	0.08	1.19E+03	0.36
9	0.037	3.21E-03	0.13	2.48E-05	0	3.44E+01	0.01
10	0.035	5.23E-05	0	9.22E-07	0	9.25E-01	0
Mass participation		2.434E00	100.00	2.434E00	100	33.27E+6	99.97

Table D.7 Lateral displacements at floor master nodes for building with masonry

Floor Level	Direction	Load Combination				
		Static	Static + N45W	Static - N45W	Static + N45E	Static - N45E
4	X	- 0.041	-0.007	- 0.075	0.149	- 0.231
	Y	0.035	0.456	- 0.385	0.078	- 0.007
	Rot.	4.30E-05	3.14E-04	- 2.28E-04	5.06E-04	- 4.20E-04
3	X	-0.028	-0.003	-0.053	0.121	-0.177
	Y	0.024	0.350	- 0.303	0.052	- 0.004
	Rot.	3.50E-05	2.37E-04	- 1.68E-04	3.50E-04	- 2.81E-04
2	X	-.015	-.001	-.030	0.086	-0.116
	Y	0.012	0.233	- 0.209	0.025	- 0.001
	Rot.	2.30E-05	1.62E-04	- 1.17E-04	1.83E-04	- 1.37E-04
1	X	-0.003	0.003	-0.010	0.049	-0.055
	Y	0.001	0.118	- 0.117	0.007	- 0.006
	Rot.	6.00E-06	9.90E-05	- 8.70E-05	4.70E-05	- 3.50E-05

Table D.8 Lateral displacements at floor master nodes for building without masonry

Floor Level	Direction	Load Combination				
		Static	Static + N45W	Static - N45W	Static + N45E	Static - N45E
4	X	-0.039	1.114	-1.192	0.696	-0.775
	Y	0.017	1.379	-1.345	0.859	-0.825
	Rot.	1.60E-05	3.48E-03	-3.45E-03	2.23E-03	-2.20E-03
3	X	-0.028	1.018	-1.075	0.639	-0.696
	Y	0.009	1.238	-1.220	0.767	-0.749
	Rot.	1.30E-05	3.00E-03	-2.97E-03	1.90E-03	-1.87E-03
2	X	-0.017	0.915	-0.949	0.576	-0.611
	Y	0.000	1.080	-1.079	0.666	-0.666
	Rot.	6.00E-06	2.49E-03	-2.47E-03	1.56E-03	-1.55E-03
1	X	-0.007	0.753	-0.767	0.476	-0.490
	Y	-0.009	0.971	-0.988	0.600	-0.617
	Rot.	-8.00E-06	2.01E-03	-2.02E-03	1.26E-03	-1.27E-03

D.5 Typical Input File

Following is the input file that was used in the case when all the masonry veneer on the building was intact.

```
2 ALHAMBRA  STREET APARTMENT COMPLEX
Spectrum N45W along Y axis  Spectrum N45E along X axis.10 MODES
All masonry intact.
STRUCTURE
*MATERIALS
1 1500. 625.
2 2550. 1090.
3 2550. 1090.
4 1600. 640.
*GEOMETRY
**COLUMN LOCATIONS
1 2 1 0. 0. 0. 0. 66.
3 4 1 0. 147. 0. 0. 240.
5 6 1 0. 308. 0. 0. 381.
7 8 1 0. 591. 0. 0. 648.
9 10 1 0. 740. 0. 60. 0.
11 12 1 186. 0. 0. 147. 648.
13 14 1 234. 0. 0. 234. 146.
15 16 1 234. 289. 0. 234. 348.
17 18 1 234. 429. 0. 234. 648.
19 20 1 234. 742. 0. 261. 0.
21 22 1 354. 0. 0. 381. 0.
23 24 1 381. 146. 0. 381. 289.
25 26 1 345. 348. 0. 381. 429.
27 28 1 345. 648. 0. 381. 648.
29 30 1 381. 742. 0. 414. 0.
31 32 1 507. 0. 0. 531. 0.
33 34 1 525. 75. 0. 525. 288.
35 36 1 525. 348. 0. 525. 429.
37 38 1 525. 648. 0. 525. 742.
39 40 1 600. 0. 0. 600. 146.
41 42 1 600. 288. 0. 600. 429.
43 44 1 600. 648. 0. 600. 742.
45 46 1 624. 0. 0. 648. 0.
47 48 1 742. 0. 0. 786. 184.
49 50 1 709. 319. 0. 813. 0.
51 52 1 786. 288. 0. 834. 429.
53 54 1 834. 648. 0. 834. 742.
55 56 1 891. 78. 0. 921. 48.
57 58 1 950. 78. 0. 866. 344.
59 60 1 936. 334. 0. 933. 648.
61 62 1 991. 220. 0. 991. 355.
63 64 1 991. 429. 0. 1035. 648.
65 66 1 991. 742. 0. 1042. 169.
67 68 1 1071. 199. 0. 1042. 229.
69 70 1 1129. 316. 0. 1129. 429.
71 72 1 1129. 595. 0. 1129. 648.
73 73 0 1129. 742. 0. 1129. 742
**BAYS
1 8 1 1 2 1
9 9 0 1 10 0
10 10 0 10 11 0
11 11 0 11 13 0
12 17 1 13 14 1
18 18 0 13 20 0
19 22 1 20 21 1
23 24 1 24 26 2
25 25 0 28 29 0
26 26 0 22 30 0
27 27 0 30 31 0
28 28 0 31 33 0
29 33 1 33 34 1
34 34 0 31 32 0
35 35 0 32 39 0
36 40 1 39 40 1
41 41 0 39 45 0
42 43 1 45 46 1
44 44 0 47 50 0
45 45 0 50 55 0
46 47 1 55 56 1
48 48 0 57 66 0
49 55 1 66 67 1
56 57 1 9 19 10
58 58 0 29 38 0
59 59 0 38 44 0
60 60 0 44 54 0
61 61 0 54 65 0
62 62 0 65 73 0
63 63 0 8 12 0
64 64 0 12 18 0
65 65 0 18 27 0
66 66 0 27 37 0
67 67 0 37 43 0
68 68 0 43 53 0
69 69 0 53 60 0
70 70 0 60 64 0
71 71 0 64 72 0
72 72 0 25 27 0
73 73 0 16 25 0
74 74 0 25 35 0
75 75 0 15 24 0
76 76 0 24 34 0
77 77 0 34 41 0
```

78 79 1 41 51 1
80 80 0 52 63 0
81 81 0 63 70 0
82 83 1 52 53 1
84 85 1 61 62 1
86 86 0 63 65 0
87 87 0 48 55 0
88 88 0 49 57 0
89 89 0 61 66 0
90 90 0 58 61 0
91 91 0 59 68 0
92 92 0 48 59 0
93 93 0 56 67 0
94 94 0 4 6 0
**STORY HEIGHTS
1 4 120
*ELEMENTS
**COLUMNS
1 R 1 6 6
**BEAMS
1 R 1 16 8
2 R 1 .1 4
**WALLS
1 2 1
1
1
1 1 1 1 .5
2 3 1
1
1
1 1 1 1 .25
3 4 1
1
1
1 1 1 1 4
**FLOORS
1
N 1 548. 366. .7915 120896.8
S
E
2
N 1 521.2 336. .6996 90319.5
S
E
3
N 1 520. 336. .2439 31182.
S
E
**FOUNDATION SPRINGS
1 1.E12 1.E12 325. 0. 0. 1.E12
2 80. 1.E12 109. 1.E12 1.E12 1.E12
3 71. 1.E12 93.4 1.E12 1.E12 1.E12
4 123. 1.E12 158. 1.E12 1.E12 1.E12
5 52. 1.E12 65. 1.E12 1.E12 1.E12
6 110. 1.E12 137. 1.E12 1.E12 1.E12
7 58. 1.E12 73. 1.E12 1.E12 1.E12
8 49. 1.E12 60. 1.E12 1.E12 1.E12
9 87. 1.E12 123. 1.E12 1.E12 1.E12
10 289. 202. 294. 1.E12 1.E12 1.E12
11 301. 301. 256. 1.E12 1.E12 1.E12
12 198. 198. 170. 1.E12 1.E12 1.E12
13 720. 720. 530. 1.E12 1.E12 1.E12
14 234. 234. 209. 1.E12 1.E12 1.E12
15 202. 335. 347. 1.E12 1.E12 1.E12
16 163. 134. 381. 1.E12 1.E12 1.E12
17 1.E12 78. 105. 1.E12 1.E12 1.E12
18 1.E12 187. 254. 1.E12 1.E12 1.E12
19 163. 110. 355. 1.E12 1.E12 1.E12
20 348. 1.E12 434. 1.E12 1.E12 1.E12
21 459. 1.E12 548. 1.E12 1.E12 1.E12
22 363. 1.E12 448. 1.E12 1.E12 1.E12
23 259. 1.E12 341. 1.E12 1.E12 1.E12
24 343. 1.E12 429. 1.E12 1.E12 1.E12
25 447. 1.E12 536. 1.E12 1.E12 1.E12
26 274. 109. 469. 1.E12 1.E12 1.E12
27 1.E12 324. 396. 1.E12 1.E12 1.E12
28 1.E12 246. 292. 1.E12 1.E12 1.E12
29 1.E12 85. 119. 1.E12 1.E12 1.E12
30 1.E12 196. 270. 1.E12 1.E12 1.E12
31 1.E12 110. 150. 1.E12 1.E12 1.E12
32 621. 621. 445. 1.E12 1.E12 1.E12
33 458. 458. 377. 1.E12 1.E12 1.E12
34 323. 323. 253. 1.E12 1.E12 1.E12
35 274. 1.E12 320. 1.E12 1.E12 1.E12
36 459. 1.E12 548. 1.E12 1.E12 1.E12
37 348. 1.E12 433. 1.E12 1.E12 1.E12
*BUILD
**COLUMNS
12 12 0
1 1 1 2
E
2 2 0 12
15 16 1 12
14 17 3
1 4 1 2
18 18 0 12
24 25 1 12
23 26 3 12
27 28 1 12
33 36 3 14
34 35 1 12
37 37 0 14
39 39 0 12
40 40 0 14

41 43 2 12
51 53 2 12
60 64 2 12
48 59 11 12
**BEAMS
1 1 0
1 4 1 1 1
2 2 0
1 1 1 1 1
2 4 2 1 1
3 3 0
1 4 2 1 1
4 4 0
1 1 2 1 1
E
5 5 0
1 1 1 1 1
E
6 7 1 3
8 8 0
1 1 2 1 1
E
9 9 0 3
10 10 0 1
11 18 7 3
12 16 1 1
17 17 0 5
19 19 0 1
20 26 6 3
21 22 1 2
23 25 1 5
27 32 1 1
33 33 0 5
34 34 0 3
35 38 1 1
39 39 0 2
40 40 0 5
41 41 0 2
42 42 0 3
43 43 0 1
44 44 0 3
45 45 0
1 1 2 1 1
2 4 1 1 1
46 50 4 3
51 51 0 45
47 49 2 8
48 48 0 5
52 52 0 3
53 53 0 1
54 54 0 3
55 62 1 8
63 63 0
2 4 1 1 1
64 64 0
1 1 1 1 1
2 4 2 1 1
65 67 1 63
68 68 0 2
69 71 2
2 4 1 1 1
70 70 0 2
72 78 1 2
79 81 1 3
82 86 1 5
87 87 0 2
91 92 1 2
88 90 1 8
93 94 1
2 4 1 1 1
**WALLS
2 2 0
2 4 2
3 3 0
1 4 2
4 4 0
1 1 2
E
6 7 1
1 4 2
8 8 0 4
9 11 2
1 1 3
2 4 1
18 20 2 9
21 22 1
2 4 1
26 26 0 9
34 34 0 9
39 39 0 21
41 41 0
2 4 1
42 44 2 9
45 51 2
1 1 3
E
46 50 4 9
52 54 2 9
55 55 0 45
56 62 1
1 1 2
E
64 68 4
2 4 1

70 70 0 64
72 78 1 64
79 81 1
1 4 1
87 87 0 64
88 90 1
1 1 1
E
91 92 1 64
**FLOORS
1 1 1 0
2 2 3 1
3 4 4 0
**FOUNDATIONS
1 1 0 1 2
2 2 0 1 1
3 3 0 1 31
4 4 0 1 30
5 5 0 1 29
6 6 0 1 28
7 7 0 1 27
8 8 0 1 18
9 9 0 1 26
10 10 0 1 2
11 11 0 1 3
13 13 0 1 4
12 12 0 1 1
14 18 1 1 1
19 19 0 1 25
20 21 1 1 5
22 22 0 1 6
23 28 1 1 1
29 29 0 1 24
30 30 0 1 7
31 32 1 1 8
33 37 1 1 1
38 38 0 1 23
39 41 1 1 1
42 42 0 1 35
43 43 0 1 1
44 44 0 1 22
45 46 1 1 8
47 47 0 1 9
48 48 0 1 1
49 49 0 1 32
50 50 0 1 10
51 51 0 1 1
52 52 0 1 36
53 53 0 1 1
54 54 0 1 21
55 55 0 1 11
56 56 0 1 12
57 57 0 1 13
58 58 0 1 34
59 60 1 1 1
61 61 0 1 33
62 62 0 1 1
63 63 0 1 37
64 64 0 1 1
65 65 0 1 20
66 66 0 1 14
67 67 0 1 12
68 68 0 1 11
69 69 0 1 15
70 70 0 1 16
71 71 0 1 17
72 72 0 1 18
73 73 0 1 19
LOADS
*VERTICAL
**PATTERN
1 U R 0. 0.
**GROUPS
1 1 .003 .003
2 1 .008 .008
3 1 .011 .011
4 1 .013 .013
5 1 .017 .017
6 1 .021 .021
7 1 .024 .024
8 1 .027 .027
9 1 .032 .032
10 1 .034 .034
11 1 .036 .036
12 1 .041 .041
13 1 .043 .043
14 1 .049 .049
15 1 .006 .006
16 1 .019 .019
17 1 .056 .056
**BAY LOADING
1 1 0
1 1 8
2 3 7
4 4 3
E
2 3 1
1 1 11
2 3 9
4 4 2
E
4 4 0
1 1 7
E

5 5 0
1 1 8
E
6 6 0
1 1 11
2 3 9
4 4 2
E
7 7 0
1 1 7
2 3 9
4 4 2
E
8 8 0
1 1 7
E
9 9 0
1 3 9
4 4 1
E
10 10 0
1 3 5
4 4 2
E
11 11 0
1 3 9
4 4 1
E
12 16 1
1 1 13
2 3 11
4 4 3
E
17 17 0
1 1 13
E
18 20 1
1 3 3
4 4 15
E
21 22 1
1 1 14
2 3 13
4 4 2
E
23 25 1
1 1 10
E
26 27 1
1 3 3
4 4 15
E
28 32 1
1 1 8
2 3 7
4 4 2
E
33 33 0
1 1 8
E
34 34 0
1 1 16
2 3 8
4 4 1
E
35 35 0
1 1 3
2 3 2
4 4 1
E
36 38 1
1 1 9
2 3 7
4 4 2
E
39 39 0
1 1 9
2 3 12
4 4 2
E
40 40 0
1 1 9
E
41 41 0
1 1 11
2 3 9
4 4 1
E
42 42 0
1 3 5
4 4 2
E
43 43 0
1 1 6
2 3 5
4 4 2
E
44 44 0
1 1 5
2 3 9
4 4 1
E
45 45 0
1 3 2

4 4 15
E
46 46 0
1 1 1
2 3 17
4 4 3
E
47 49 2
1 1 1
E
48 48 0
1 1 3
E
50 50 0
1 1 1
2 3 11
4 4 15
E
51 51 0
1 1 2
2 3 4
4 4 2
E
52 52 0
1 3 9
4 4 1
E
53 53 0
1 3 5
4 4 15
E
54 54 0
1 3 9
4 4 1
E
55 55 0
1 1 4
E
56 62 1
1 1 1
E
63 63 0
2 3 16
4 4 15
E
64 68 4
1 1 16
2 3 8
4 4 15
E
65 67 1
2 3 16
4 4 2
E
69 71 2
2 3 16
4 4 2
E
70 70 0
1 1 16
2 3 8
4 4 15
E
72 72 0
1 1 16
2 3 13
4 4 2
E
73 74 1
1 1 16
2 3 9
4 4 1
E
75 78 1
1 1 16
2 3 11
4 4 15
E
79 81 1
1 1 12
2 3 17
4 4 3
E
82 83 1
1 1 10
E
84 86 1
1 1 7
E
87 87 0
2 3 17
4 4 3
E
88 90 1
1 1 10
E
91 91 0
2 3 11
4 4 15
E
92 92 0
2 3 14
4 4 3
E

```
93 93 0
2 3 4
4 4 2
E
94 94 0
2 3 7
4 4 3
E
*MODES
10 325.
*DAMP
1 10 0.05
*SPECTRUM
132 1. 0.0
**USER
PRESIDIO..PRN45W. Damping  STRUC. 5% SOIL 7%
.02,    77.31
.04,    77.57
.06,    77.98
.08,    78.65
.10,    78.51
.12,    80.04
.14,    86.08
.16,    87.21
.18,    98.11
.20,    107.94
.22,    110.77
.24,    101.07
.26,    97.22
.28,    100.16
.30,    103.57
.32,    100.22
.34,    122.52
.36,    151.11
.38,    173.07
.40,    195.83
.42,    206.58
.44,    202.80
.46,    216.12
.48,    233.41
.50,    225.92
.52,    212.99
.54,    200.47
.56,    194.73
.58,    195.01
.60,    196.97
.62,    199.79
.64,    206.03
.66,    212.41
.68,    215.26
.70,    218.22
.72,    221.67
.74,    217.95
.76,    205.81
.78,    189.63
.80,    176.41
.82,    162.18
.84,    148.12
.86,    134.48
.88,    121.77
.90,    109.77
.92,    99.31
.94,    90.64
.96,    83.52
.98,    78.52
1.00,   76.56
1.02,   77.39
1.04,   80.17
1.06,   84.53
1.08,   89.82
1.10,   95.17
1.12,   102.40
1.14,   108.40
1.16,   111.67
1.18,   111.97
1.20,   109.69
1.22,   105.57
1.24,   100.44
1.26,   94.98
1.28,   90.26
1.30,   87.42
1.32,   84.97
1.34,   82.82
1.36,   80.52
1.38,   78.17
1.40,   75.55
1.42,   72.63
1.44,   69.79
1.46,   67.02
1.48,   64.40
1.50,   61.99
1.52,   59.90
1.54,   57.91
1.56,   55.94
1.58,   53.95
1.60,   52.01
1.62,   50.11
1.64,   48.24
1.66,   46.44
1.68,   44.75
1.70,   43.22
1.72,   41.82
1.74,   40.48
1.76,   39.19
```

1.78, 37.90
1.80, 36.66
1.82, 35.41
1.84, 34.13
1.86, 32.82
1.88, 31.55
1.90, 30.68
1.92, 30.61
1.94, 30.54
1.96, 30.45
1.98, 30.33
2.00, 30.17
2.25, 21.10
2.50, 22.24
2.75, 24.59
3.00, 15.08
3.25, 13.41
3.50, 9.45
3.75, 9.55
4.00, 9.73
4.25, 8.33
4.50, 6.89
4.75, 5.71
5.00, 4.30
5.25, 3.30
5.50, 2.56
5.75, 2.47
6.00, 2.53
6.25, 2.65
6.50, 2.66
6.75, 2.95
7.00, 3.18
7.25, 3.08
7.50, 2.80
7.75, 2.41
8.00, 2.20
8.25, 2.11
8.50, 1.96
8.75, 1.78
9.00, 1.58
9.25, 1.38
9.50, 1.18
9.75, 1.03
10.00, .98
132 1. 90.
**USER
PRESIDIO..PRN45E. Damping STRUC. 5% SOIL 7%
.02, 72.73
.04, 72.90
.06, 73.08
.08, 73.00
.10, 73.85
.12, 76.00
.14, 80.57
.16, 82.32
.18, 88.06
.20, 94.38
.22, 99.03
.24, 104.94
.26, 117.31
.28, 119.29
.30, 97.37
.32, 110.33
.34, 142.23
.36, 154.21
.38, 152.63
.40, 127.98
.42, 115.57
.44, 135.72
.46, 146.50
.48, 137.18
.50, 115.11
.52, 118.37
.54, 125.45
.56, 126.37
.58, 119.15
.60, 129.20
.62, 142.51
.64, 148.23
.66, 157.11
.68, 162.43
.70, 160.59
.72, 166.23
.74, 170.88
.76, 174.48
.78, 177.35
.80, 180.29
.82, 183.48
.84, 185.05
.86, 184.38
.88, 181.53
.90, 196.52
.92, 212.88
.94, 221.81
.96, 222.82
.98, 217.39
1.00, 206.75
1.02, 194.55
1.04, 184.89
1.06, 179.26
1.08, 177.63
1.10, 179.37
1.12, 182.47
1.14, 185.46

```
1.16,  192.33
1.18,  196.64
1.20,  197.77
1.22,  195.79
1.24,  191.33
1.26,  185.19
1.28,  178.05
1.30,  170.39
1.32,  162.60
1.34,  154.55
1.36,  146.10
1.38,  139.09
1.40,  135.17
1.42,  130.75
1.44,  125.85
1.46,  122.91
1.48,  119.93
1.50,  116.70
1.52,  113.63
1.54,  110.49
1.56,  107.66
1.58,  104.93
1.60,  102.36
1.62,  100.13
1.64,  98.03
1.66,  96.09
1.68,  94.48
1.70,  93.02
1.72,  91.68
1.74,  90.54
1.76,  89.54
1.78,  88.59
1.80,  87.71
1.82,  86.90
1.84,  86.02
1.86,  85.16
1.88,  84.24
1.90,  83.21
1.92,  82.20
1.94,  81.05
1.96,  79.88
1.98,  78.66
2.00,  77.35
2.25,  61.12
2.50,  51.35
2.75,  49.28
3.00,  37.38
3.25,  33.12
3.50,  36.40
3.75,  38.91
4.00,  38.93
4.25,  36.71
4.50,  32.22
4.75,  26.39
5.00,  20.90
5.25,  18.03
5.50,  15.68
5.75,  13.77
6.00,  12.24
6.25,  10.95
6.50,  9.78
6.75,  8.68
7.00,  7.64
7.25,  6.69
7.50,  5.98
7.75,  5.43
8.00,  4.92
8.25,  4.45
8.50,  4.03
8.75,  3.65
9.00,  3.32
9.25,  3.04
9.50,  2.79
9.75,  2.58
10.00,  2.40
*COMBINATIONS
1.
1. 0. 0. 0. 0. 0. 0. 0. 0. 0. 0. 1.
1. 0. 0. 0. 0. 0. 0. 0. 0. 0. 0. 0. 1.
SOLVE
OUTPUT
*FORCES
**COLUMNS
1 2
**BEAMS
1 2
**WALLS
1 2
*PROCESS
**WALLS
H 1 9 1 1
H 1 11 1 1
H 1 18 1 1
H 1 20 1 1
H 1 26 1 1
H 1 34 1 1
H 1 42 1 1
H 1 44 1 1
H 1 45 1 1
H 1 46 1 1
H 1 47 1 1
H 1 49 1 1
H 1 50 1 1
H 1 51 1 1
```

H 1 52 1 1
H 1 54 1 1
H 1 55 1 1
H 1 60 1 1
H 1 6 1 1
STOP

APPENDIX E
HOTEL WOODROW

E.1 Scope

This appendix summarizes the steps used to derive the analytical model of the Hotel Woodrow, and also contains some of the analysis results related to the lateral response of the structure. In Figs. E.1 and E.2 are shown the numbering schemes used for floor beams and floor nodes at different levels. Figs. E.3 and E.4 contain the node and bay numbering schemes used in the model.

E.2 Modelling Criteria

Material properties for principal construction materials used in the building are given in Table E.1.

Table E.1 Material properties

MATERIAL	ν	E (ksi)	G (ksi)
STEEL	0.30	29000	11200
CONCRETE	0.25	3120	1230
WOOD	0.20	1500	625
PLASTER/STUCCO	0.17	2550	1090
MASONRY	0.40	1600	640

Where

E	=	Modulus of Elasticity
G	=	Shear Modulus
ν	=	Poisson's Ratio

E.2.1 Dead Loads: The following were estimated values for various types of dead loads on the structure [5].

<u>WALLS:</u>

Wooden Studs (2-x 4-in.) spaced at 12 inches	=	1.30 psf of wall area
Plaster on Wood Lath	=	10 psf pf wall area
Brick Masonry (8 in. thick)	=	76 psf of wall area
Hollow Terra Cotta (8 in. thick)	=	35 psf of wall area

<u>BEAMS AND COLUMNS:</u>

Steel	=	490 pcf

FLOOR DIAPHRAGM:

Wooden Joists (2-x12-in.) spaced at 16 inches	=	3.2 psf of floor area
Lumber Sheathing (1.5 in. thick)	=	4 psf of floor area
Ceiling (1 in. thick plaster on metal lath)	=	8.5 psf of floor area
Loose Insulation (6 in. thick)	=	3 psf of floor area
Poured Insulation on Roof	=	2.0 psf of roof area

E.2.2 Live Loads: The live load was assumed to be 70% of the code recommended value [37]. This represented an average value of live load on the structure at the time of the earthquake [16]. Because of difference in usage, live loads were assumed to be different for various levels and their values are given below:

Live load on Level 1	=	0.7 x 50 = 35 psf
Live load on Levels 2-8	=	0.7 x 40 =28 psf
Live load on Levels 9 and 10	=	0 psf

E.3 Design Calculations

E.3.1 Mass Calculations: At each level, masses and rotary inertia were calculated using all members or portions thereof within half a story height above and below the floor diaphragm at that level. Resultants were calculated for these quantities by using the procedure described in Appendix B, Section B.3.1.

At each floor level, these resultants were distributed among five floor nodes at that level in order to better represent distribution of masses. Sample calculations for the distribution of masses and rotary inertia at Level 1 are given below:

$$M_{c.g} = 0.9641 \text{ k-sec}^2/\text{in.}$$

$$I_{c.g} = 126281 \text{ k-sec}^2\text{-in.}$$

The location of the center of gravity with respect to the southwest corner of the building is

$$X_{c.g} = 460 \text{ in.}$$

$$Y_{c.g} = 238 \text{ in.}$$

Using this information, the locations of the floor nodes were decided. Nodes 1 and 5 were placed at the east and west extremes of the building respectively, and Node 3 was placed at the plan center of gravity. Nodes 2 and 4 were placed midway between Node 3 and each of the extremes, as shown in Fig. E.1. The location of each node is shown in Table E.2.

Table E.2 Locations of floor nodes at Level 1

Node	X (in.)	Y (in.)
1	0	238
2	230	238
3	460	238
4	731	238
5	1002	238

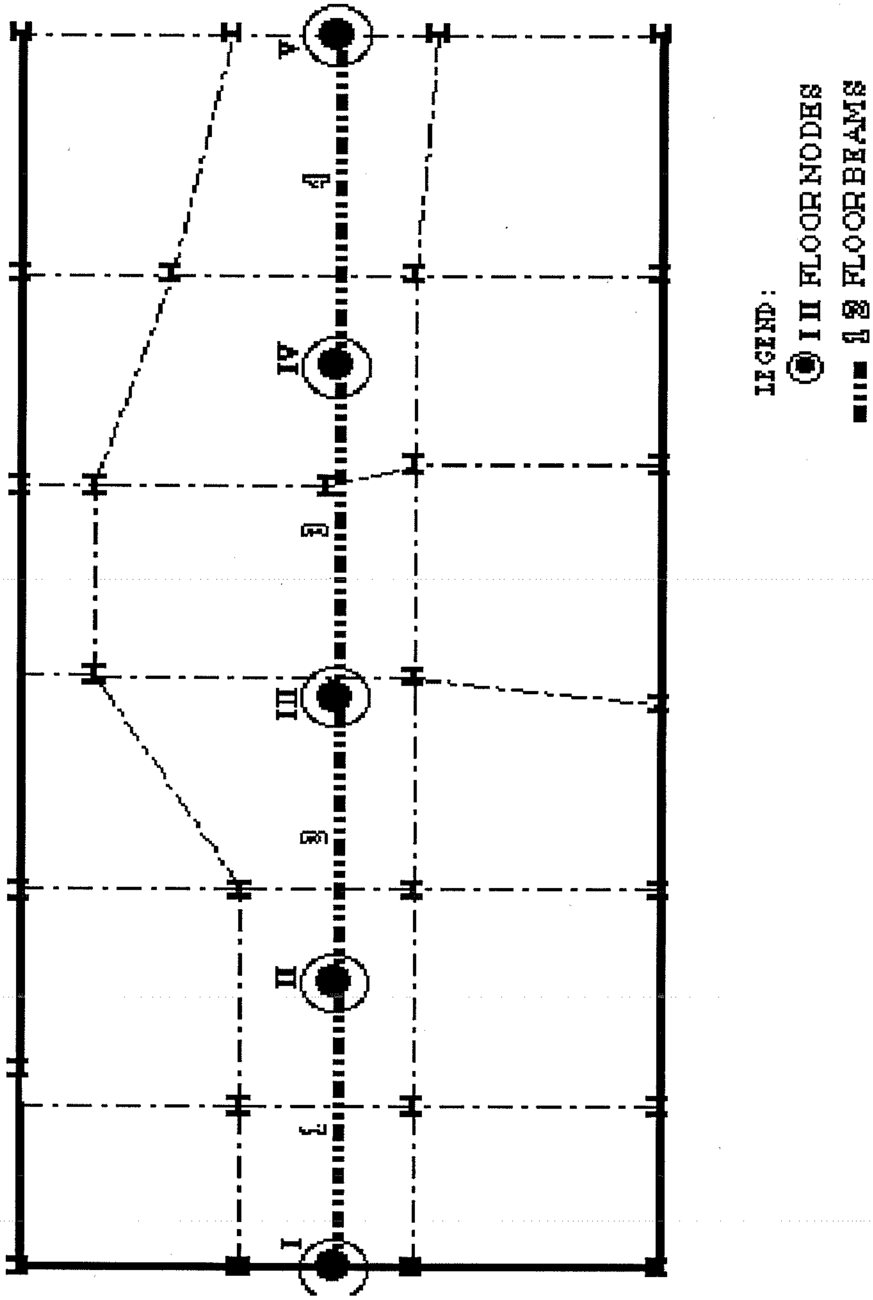

Figure E.1 Numbering schemes for floor beams and floor nodes except at Level 2 (Hotel Woodrow)

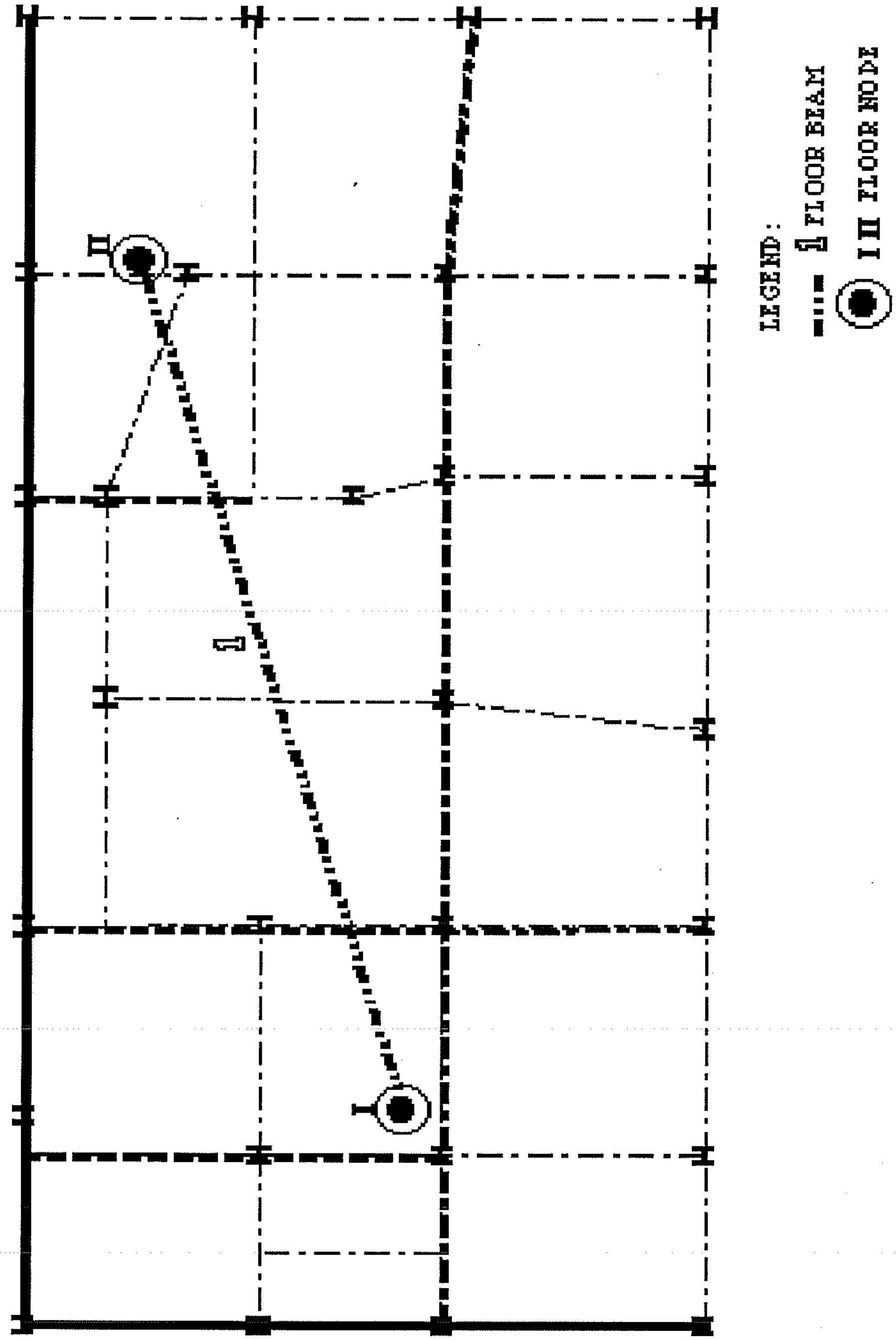

Figure E.2 Numbering scheme for floor beam and floor nodes at Level 2 (Hotel Woodrow)

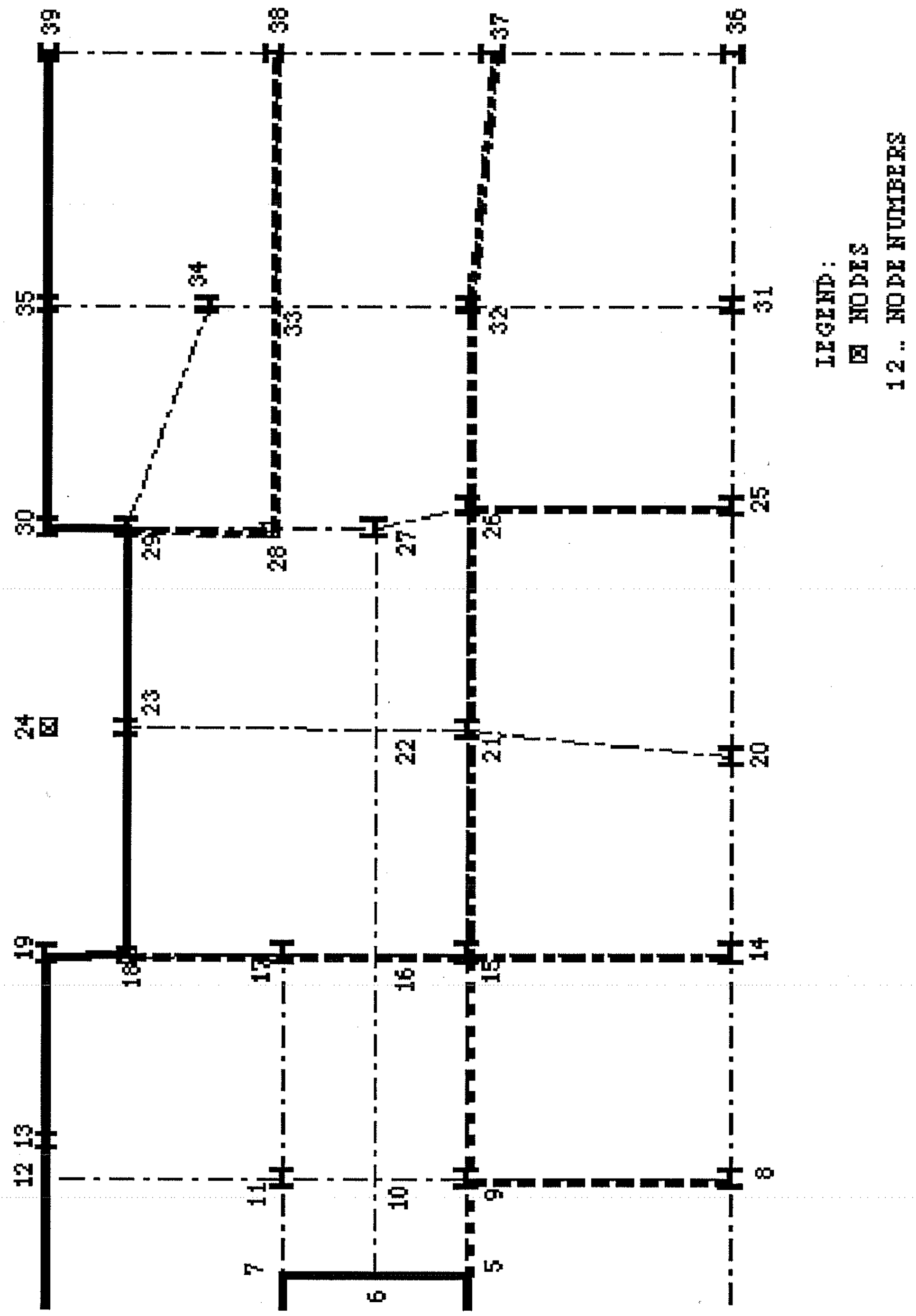

Figure E.3 Node number scheme (Hotel Woodrow)

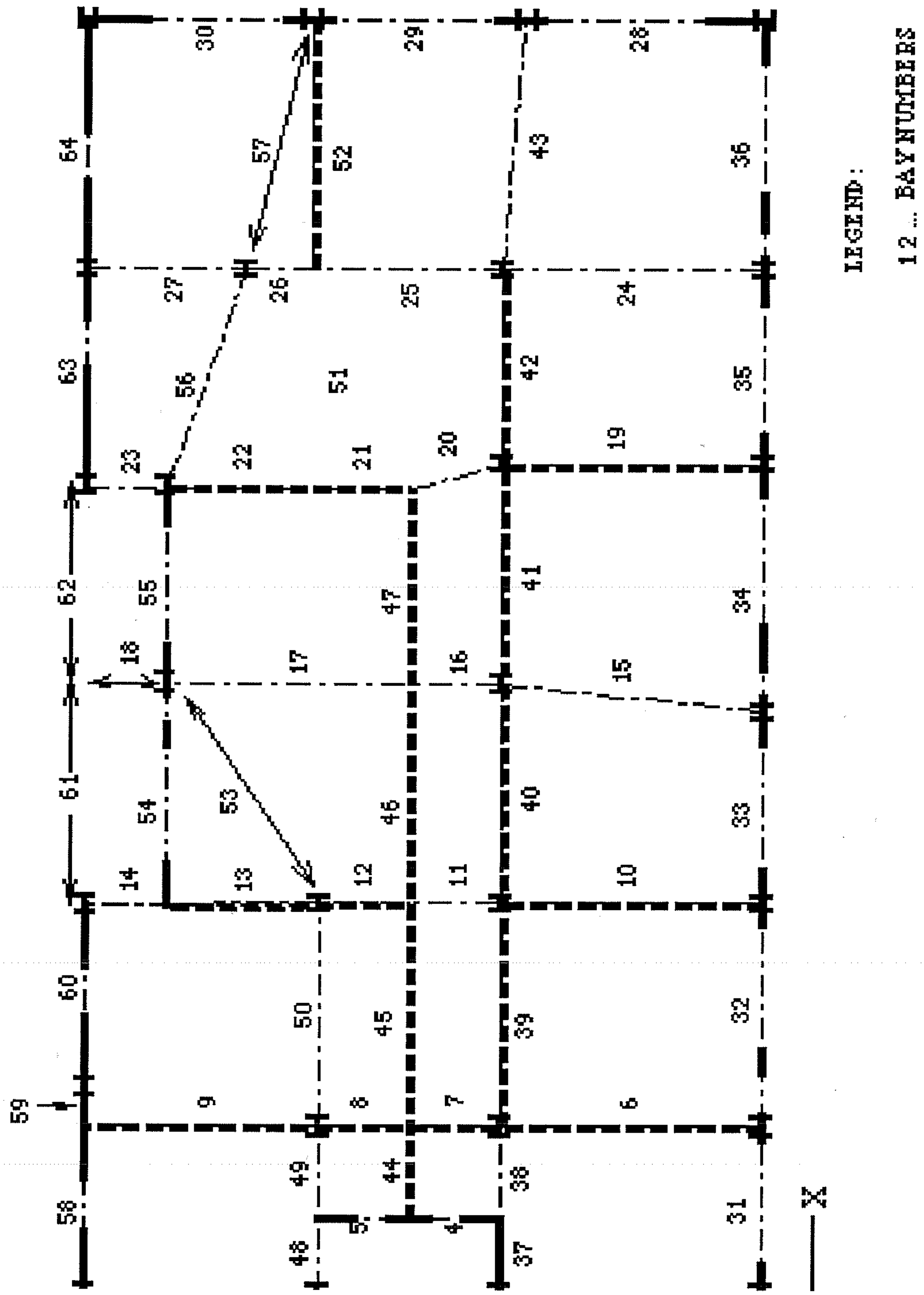

Figure E.4 Bay number scheme (Hotel Woodrow)

The centroidal mass was at first tentatively distributed between Nodes 2 and 4 on the basis of their proximity to Node 3, as shown below:

$$M_1^* + M_2^* = 0.9641$$

$$230M_1^* - 271M_2^* = 0$$

Solution of those two equation gives

$$M_1^* = 0.5215$$

$$M_2^* = 0.4426$$

These two masses were distributed among the five floor nodes as follows.

$$M_1 = \frac{M_1^*}{4} = 0.1304$$

$$M_2 = \frac{M_1^*}{2} = 0.2607$$

$$M_3 = \frac{M_1^*}{4} + \frac{M_2^*}{4} = 0.2410$$

$$M_4 = \frac{M_2^*}{2} = 0.2213$$

$$M_5 = \frac{M_2^*}{4} = 0.1107$$

In order to conserve the rotary inertia of the floor diaphragm, the following equation was satisfied:

$$\Sigma M_i d_i + \Sigma I_i = I_{c.g}$$

where

M_i = Mass associated with each floor node

d_i = Distance between the floor node and the ... center of gravity

I_i = Rotary inertia associated with each ... floor node

Solving the above equation gives

$$\Sigma I_o = 36125$$

This rotary inertia was distributed among the floor nodes in proportion to their masses.

However, this approach was not used in the case of Level 2, whose floor diaphragm was in two separate segments. In that case, centroidal mass and rotary inertia were calculated for each segment separately, considering only the mass of the floor diaphragm itself. Once the locations of those centroids were determined, masses and rotary inertia of other members on that level were assigned to these centroids employing the principles demonstrated above. Hence, the total mass at that level and the rotary inertia associated with it were conserved.

Masses, rotary inertia and locations with respect to the southwest corner of the building are given in Table E.3 for floor nodes at all levels. All distances are in inch units while the masses and rotary inertia have the units of k-sec^2/in and k-sec^2-in. respectively.

E.3.2 Foundation Springs: The effects of soil-foundation flexibility were incorporated into the model by attaching a set of six springs to the base of every column, and in case of wall panels, to each of the bottom two nodes at the wall corners. These springs represented translational stiffnesses (K_x, K_y, K_z) along and rotational stiffnesses ($K_{\theta x}$, $K_{\theta y}$ and $K_{\theta z}$) about the X, Y and Z axes. These stiffnesses were calculated using the technique described in Appendix B.

The following soil properties were used in the calculations [30].

C_s = 1200 ft/sec

ρ = 4.04 lb-sec/ft^2

ν = 0.24

$G = \rho C_s^2 = 40.4$ k/in^2

From structural drawings it was determined that the spread footings were 26 in. square and the strip footings were 24 in. wide. The final results are shown in Table E.4. The units used for the translational and rotational stiffnesses are k/in. and k-in. respectively.

E.4 General Results of the Analyses

This section contains analysis results depicting the lateral response of the structure under various combinations of static and dynamic loads. In the tables presented at the end of this section, N115W denotes the spectrum applied in the global Y direction, and N25W represents the spectrum applied in the global X direction (Fig. E.4).

Significant mode shapes in both the directions are shown in Figs. E.5-E.8 for Cases A and D respectively. These two cases represent the initial and the final phases of the sequential linear analysis that was used to analyze this structure, and are fully explained in Chapter 4. For Case A, up to 25 modes were considered during the analysis and the total mass participation in X and Y directions and in rotation about Z axis was 99.86%, 99.95% and 38.2% respectively. Table E.5 contains the periods and mass contributions for the first 15 modes. For Case D, since the structure had become more flexible, 35 modes were considered during the analysis and the total mass participation in X and Y directions and in rotation about Z axis was 94.88%, 99.12% and 59.75% respectively. Table E.6 shows periods and mass contributions for the first 15 modes for Case D. In both the cases, higher modes were less important because of their high frequencies and minimal mass contributions, and hence have not been included in the tables. As the units used in the input files were kips and inches, all the quantities in the tables have the corresponding units.

Tables E.7 and E.8 give the lateral displacements in inches and rotations in radians at the floor nodes for cases A and D respectively.

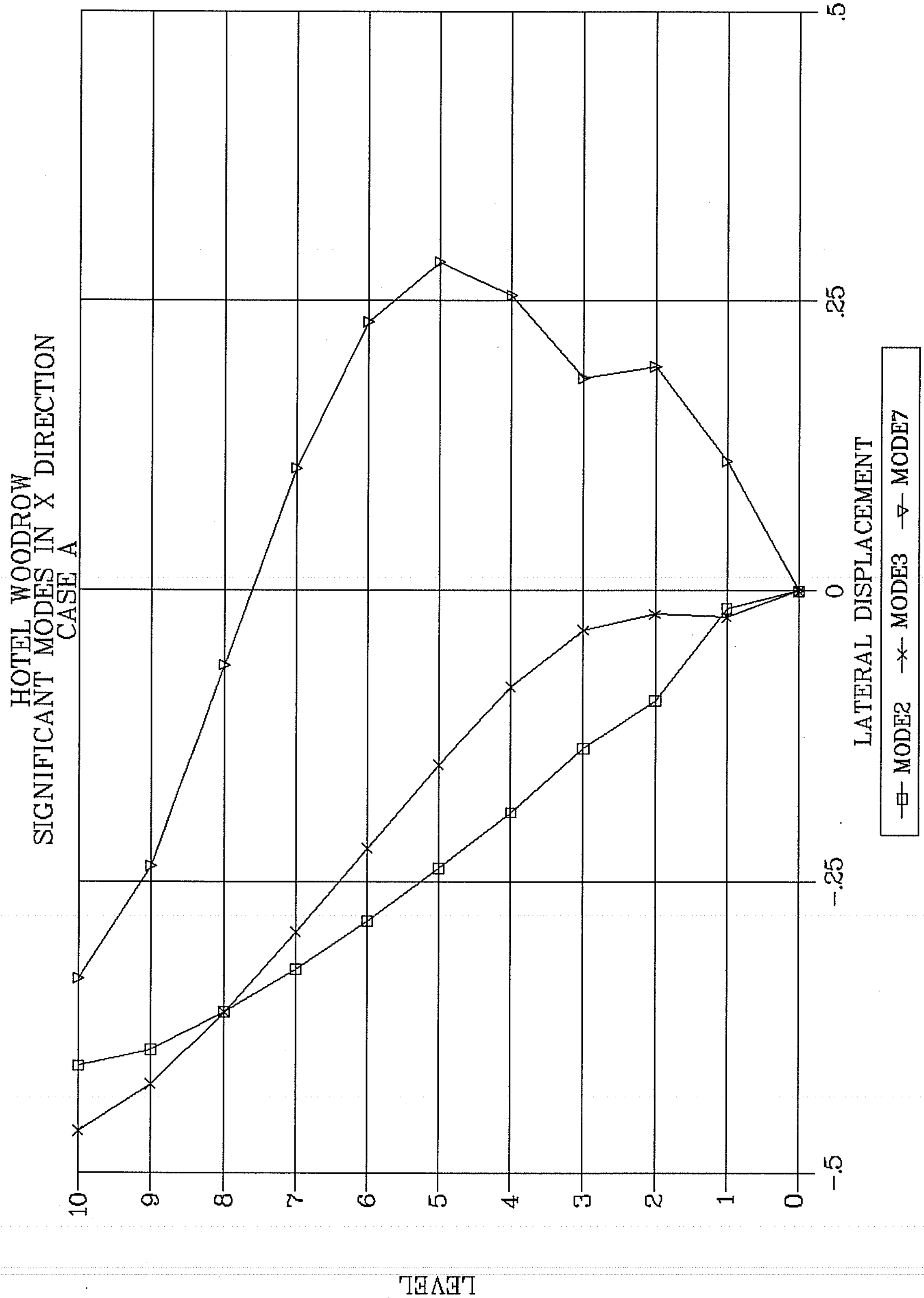

Figure E.5 Significant nodes in X direction (Case A)

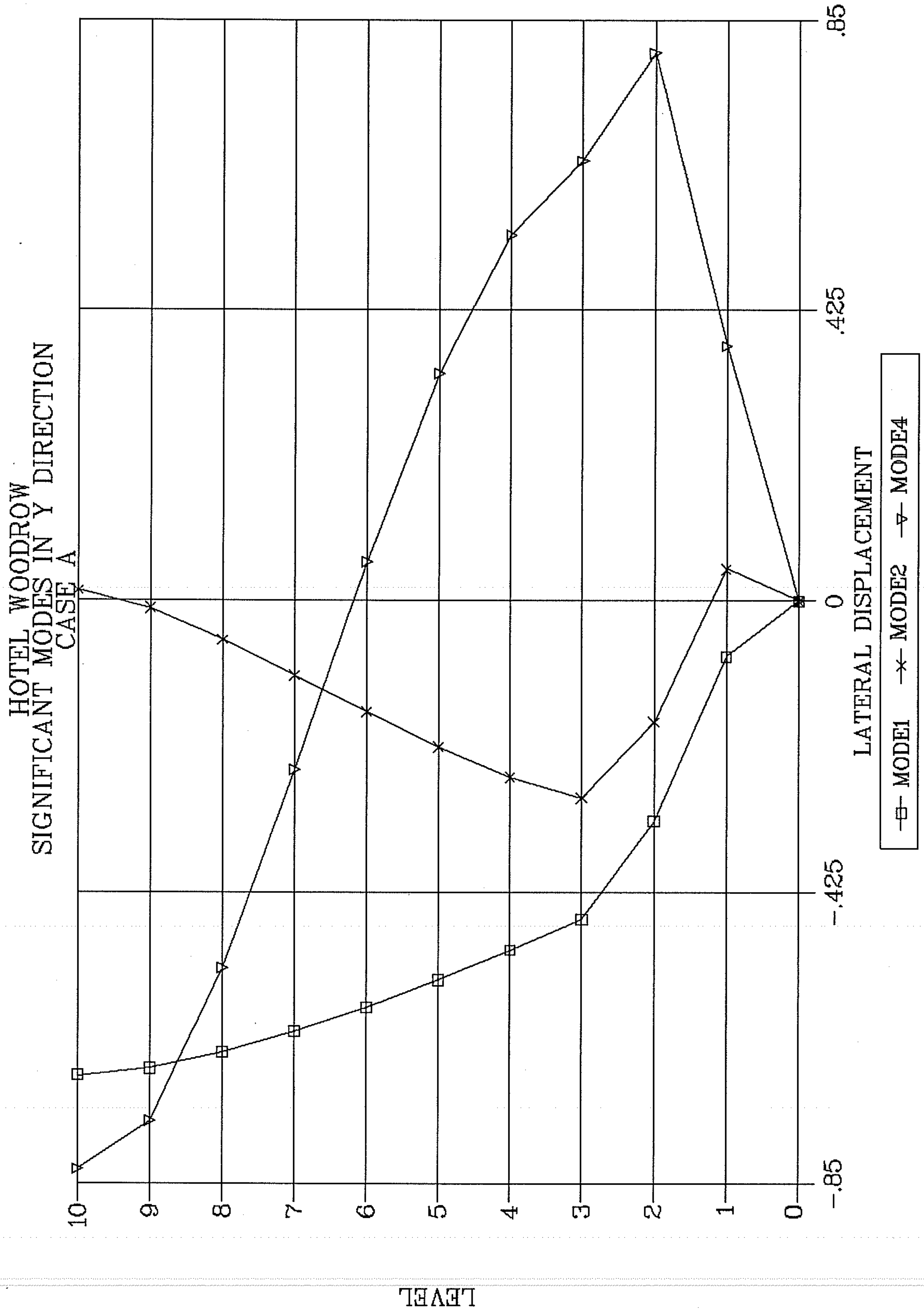

Figure E.6 Significant nodes in Y direction (Case A)

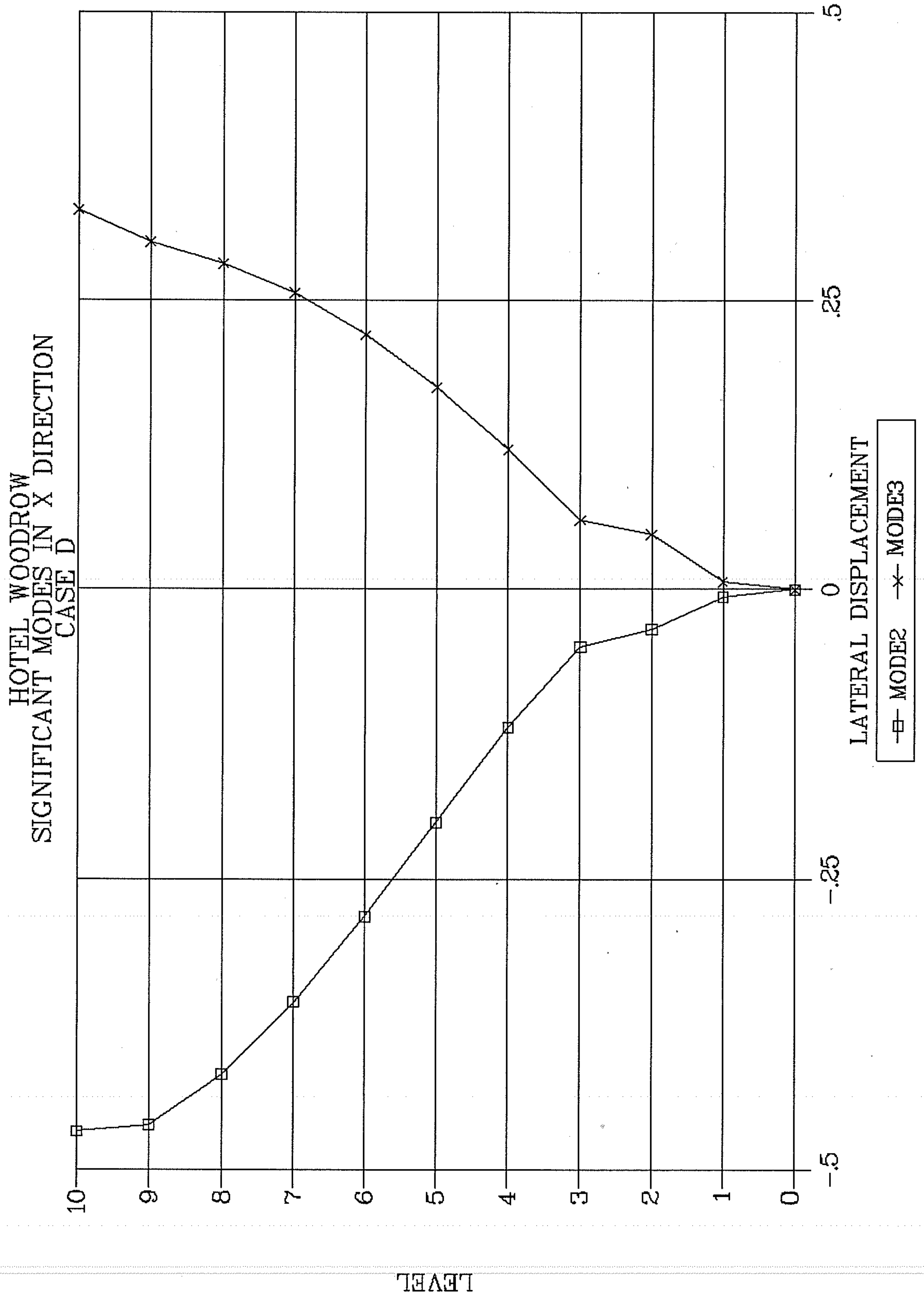

Figure E.7 Significant nodes in X direction (Case D)

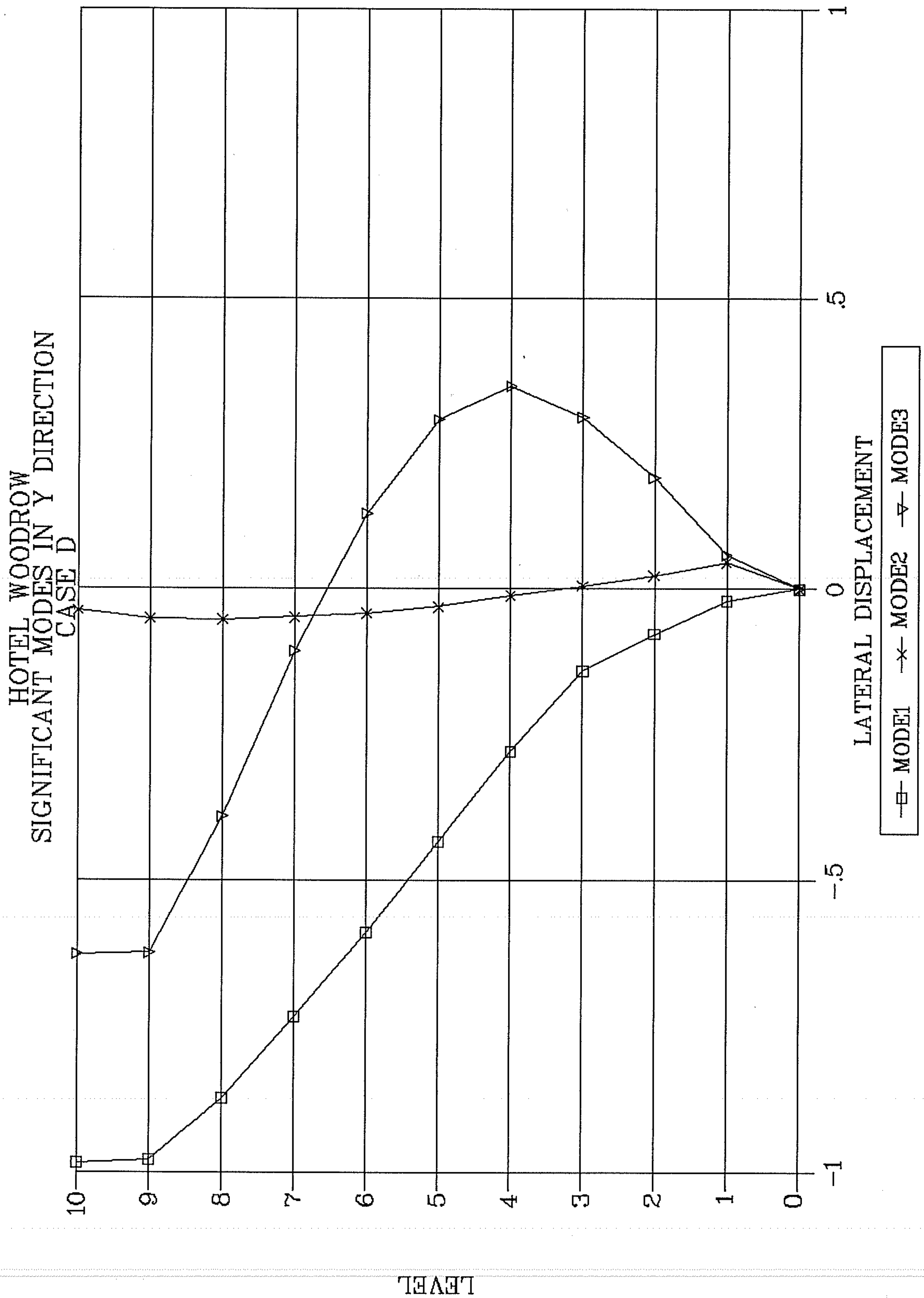

Figure E.8 Significant nodes in Y direction (Case D)

Table E.3 Locations, masses and rotary inertias of floor nodes

Level	Floor Node	X	Y	MASS	ROTARY INERTIA
1	1	0	238	0.1304	4886
	2	230	238	0.2607	9768
	3	460	238	0.241	9030
	4	731	238	0.2213	8292
	5	1002	238	0.1107	4148
2	1	167.5	205	0.30835	3926
	2	818	386	0.2139	1963
3	1	0	253	0.0969	3580
	2	239	253	0.1939	7164
	3	478	253	0.1854	6850
	4	740	253	0.1768	6532
	5	1002	253	0.0885	3270
4 - 8	1	57	233	0.1128	6184
	2	271.5	233	0.2255	12363
	3	486	233	0.2065	11321
	4	744	233	0.1875	10280
	5	1002	233	0.0937	5137
9	1	57	230	0.1039	5899
	2	275.5	230	0.2077	11792
	3	494	230	0.1933	10975
	4	748	230	0.1787	10146
	5	1002	230	0.0894	5076
10	1	57	230	0.0377	2549
	2	275.5	230	0.0754	5098
	3	494	230	0.0702	4746
	4	748	230	0.0649	4388
	5	1002	230	0.0324	2190

Table E.4 Values for foundation springs

Node No.	K_x	K_y	K_z	$K_{\theta x}$	$K_{\theta y}$	$K_{\theta z}$
1, 4	7670	7468	9095	1E+12	1E+12	1E+12
2, 3	1E+12	9600	8863	1E+12	1E+12	1E+12
8, 14, 19, 30	9900	1E+12	9363	1E+12	1E+12	1E+12
20, 25, 31, 35	10400	1E+12	9963	1E+12	1E+12	1E+12
36, 39	7010	5039	5463	1E+12	1E+12	1E+12
13	7400	1E+12	5963	1E+12	1E+12	1E+12
12	4500	1E+12	5650	1E+12	1E+12	1E+12
24	7250	1E+12	9500	1E+12	1E+12	1E+12
9, 11, 15, 17, 21, 23, 26, 27, 28, 29, 32, 33, 34, 37, 38	1E+12	1E+12	3137	0	0	1E+12

Table E.5 Periods and participating modal masses (Case A)

Mode No.	Period (sec)	X Direction		Y Direction		Rotation about Z	
		Modal Mass	% of Total	Modal Mass	% of Total	Modal Mass	% of Total
1	0.562	2.04E-01	2.750	5.02E+00	67.760	2.21E+04	6.170
2	0.295	3.03E+00	40.820	7.02E-01	9.470	3.20E+04	8.920
3	0.235	2.07E+00	27.960	1.26E-01	1.700	5.70E+04	15.900
4	0.138	4.99E-02	0.670	8.84E-01	11.930	5.18E+03	1.450
5	0.085	3.69E-01	4.980	1.65E-01	2.230	8.29E+02	0.230
6	0.082	1.51E-01	2.040	1.91E-02	0.260	1.75E+00	0.000
7	0.066	5.54E-01	7.470	2.19E-01	2.950	7.48E+03	2.090
8	0.063	2.52E-02	0.340	1.26E-01	1.700	8.01E+02	0.220
9	0.054	4.13E-02	0.560	2.87E-02	0.390	3.80E+03	1.060
10	0.043	1.27E-01	1.720	3.13E-03	0.040	7.38E+01	0.020
11	0.040	1.85E-01	2.490	6.89E-04	0.010	9.75E+02	0.270
12	0.037	1.07E-01	1.450	3.52E-02	0.480	5.87E+02	0.160
13	0.032	1.32E-02	0.180	2.83E-05	0.000	2.08E+00	0.000
14	0.030	8.58E-02	1.160	8.19E-03	0.110	1.01E+03	0.280
15	0.027	1.47E-02	0.200	4.84E-03	0.070	1.87E+02	0.050

Table E.6 Periods and participating modal masses (Case D)

Mode No.	Period (sec)	X Direction		Y Direction		Rotation about Z	
		Modal Mass	% of Total	Modal Mass	% of Total	Modal Mass	% of Total
1	1.055	1.19E-01	1.610	3.01E+00	40.570	4.05E+04	11.290
2	0.458	2.83E+00	38.240	1.20E+00	16.140	1.85E+04	5.160
3	0.377	1.60E+00	21.650	1.48E+00	19.950	2.26E+04	6.290
4	0.321	2.25E-01	3.040	7.73E-02	1.040	2.79E+04	7.770
5	0.186	4.66E-03	0.060	2.82E-01	3.810	1.20E+03	0.340
6	0.138	3.18E-01	4.290	5.26E-01	7.100	1.74E+03	0.480
7	0.131	3.92E-01	5.290	1.73E-03	0.020	6.45E+02	0.180
8	0.107	3.57E-02	0.480	1.03E-01	1.380	1.27E+03	0.350
9	0.099	2.63E-01	3.550	9.94E-02	1.340	1.59E+04	4.440
10	0.091	4.53E-03	0.060	1.64E-02	0.220	1.39E+02	0.040
11	0.082	1.04E-01	1.400	1.54E-01	2.080	1.49E+02	0.040
12	0.080	9.91E-02	1.340	8.61E-02	1.160	5.20E+02	0.150
13	0.070	6.46E-03	0.090	7.89E-04	0.010	3.16E+02	0.090
14	0.068	1.21E-02	0.160	7.22E-02	0.970	4.34E+02	0.120
15	0.065	1.01E-02	0.140	1.59E-03	0.020	1.16E+04	3.240

Table E.7 Lateral displacements at floor nodes under various loads (Case A)

Floor Level	Floor Node	Direction	Load Combination				
			Static	Static + N115W	Static - N115W	Static + N25W	Static - N25W
10	1	X	0.047	0.278	-0.183	0.401	-0.307
		Y	-0.052	0.589	-0.693	0.405	-0.510
		ROT	-3.50E-05	1.14E-03	-1.21E-03	5.57E-04	-6.26E-04
	2	X	0.047	0.278	-0.183	0.401	-0.307
		Y	-0.060	0.812	-0.932	0.309	-0.429
		ROT	-3.50E-05	1.14E-03	-1.21E-03	5.57E-04	-6.26E-04
	3	X	0.047	0.278	-0.183	0.401	-0.307
		Y	-0.067	1.047	-1.182	0.244	-0.378
		ROT	-3.50E-05	1.14E-03	-1.21E-03	5.57E-04	-6.26E-04
	4	X	0.047	0.278	-0.183	0.401	-0.307
		Y	-0.076	1.327	-1.479	0.228	-0.381
		ROT	-3.50E-05	1.14E-03	-1.21E-03	5.57E-04	-6.26E-04
	5	X	0.047	0.278	-0.183	0.401	-0.307
		Y	-0.085	1.611	-1.781	0.281	-0.451
		ROT	-3.50E-05	1.14E-03	-1.21E-03	5.57E-04	-6.26E-04
9	1	X	0.042	0.269	-0.185	0.377	-0.294
		Y	-0.047	0.563	-0.658	0.389	-0.484
		ROT	-3.20E-05	1.15E-03	-1.21E-03	5.50E-04	-6.14E-04
	2	X	0.042	0.269	-0.185	0.377	-0.294
		Y	-0.054	0.788	-0.897	0.296	-0.404
		ROT	-3.20E-05	1.15E-03	-1.21E-03	5.50E-04	-6.14E-04
	3	X	0.042	0.269	-0.185	0.377	-0.294
		Y	-0.061	1.025	-1.147	0.234	-0.356
		ROT	-3.20E-05	1.15E-03	-1.21E-03	5.50E-04	-6.14E-04
	4	X	0.042	0.269	-0.185	0.377	-0.294
		Y	-0.069	1.307	-1.446	0.225	-0.363
		ROT	-3.20E-05	1.15E-03	-1.21E-03	5.51E-04	-6.14E-04
	5	X	0.042	0.269	-0.185	0.377	-0.294
		Y	-0.077	1.592	-1.747	0.282	-0.437
		ROT	-3.20E-05	1.15E-03	-1.21E-03	5.50E-04	-6.14E-04

(continued)

Table E.7 Lateral displacements at floor nodes under various loads (Case A)(continued)

Floor Level	Floor Node	Direction	Load Combination				
			Static	Static + N115W	Static - N115W	Static + N25W	Static - N25W
8	1	X	0.035	0.252	-0.182	0.337	-0.267
		Y	-0.040	0.513	-0.593	0.353	-0.433
		ROT	-2.70E-05	1.15E-03	-1.20E-03	5.32E-04	-5.85E-04
	2	X	0.035	0.252	-0.182	0.337	-0.267
		Y	-0.046	0.735	-0.827	0.267	-0.359
		ROT	-2.70E-05	1.15E-03	-1.20E-03	5.32E-04	-5.85E-04
	3	X	0.035	0.252	-0.182	0.337	-0.267
		Y	-0.051	0.969	-1.072	0.214	-0.317
		ROT	-2.70E-05	1.15E-03	-1.20E-03	5.32E-04	-5.85E-04
	4	X	0.035	0.252	-0.182	0.337	-0.267
		Y	-0.058	1.256	-1.373	0.216	-0.333
		ROT	-2.70E-05	1.15E-03	-1.20E-03	5.33E-04	-5.86E-04
	5	X	0.035	0.252	-0.182	0.337	-0.267
		Y	-0.065	1.546	-1.677	0.284	-0.414
		ROT	-2.70E-05	1.15E-03	-1.20E-03	5.32E-04	-5.85E-04
7	1	X	0.028	0.237	-0.181	0.292	-0.236
		Y	-0.033	0.453	-0.518	0.305	-0.370
		ROT	-2.20E-05	1.14E-03	-1.18E-03	5.07E-04	-5.50E-04
	2	X	0.028	0.237	-0.181	0.292	-0.236
		Y	-0.037	0.676	-0.750	0.228	-0.303
		ROT	-2.20E-05	1.14E-03	-1.18E-03	5.07E-04	-5.50E-04
	3	X	0.028	0.237	-0.181	0.292	-0.236
		Y	-0.042	0.909	-0.993	0.188	-0.271
		ROT	-2.20E-05	1.14E-03	-1.18E-03	5.07E-04	-5.50E-04
	4	X	0.028	0.237	-0.181	0.292	-0.236
		Y	-0.047	1.196	-1.290	0.208	-0.303
		ROT	-2.20E-05	1.14E-03	-1.18E-03	5.07E-04	-5.50E-04
	5	X	0.028	0.237	-0.181	0.292	-0.236
		Y	-0.053	1.485	-1.591	0.286	-0.392
		ROT	-2.20E-05	1.14E-03	-1.18E-03	5.07E-04	-5.50E-04

(continued)

Table E.7 Lateral displacements at floor nodes under various loads (Case A)(continued)

Floor Level	Floor Node	Direction	Load Combination				
			Static	Static + N115W	Static - N115W	Static + N25W	Static - N25W
6	1	X	0.022	0.222	-0.179	0.245	-0.201
		Y	-0.025	0.386	-0.436	0.250	-0.300
		ROT	-1.70E-05	1.12E-03	-1.16E-03	4.75E-04	-5.09E-04
	2	X	0.022	0.222	-0.179	0.245	-0.201
		Y	-0.029	0.607	-0.665	0.184	-0.242
		ROT	-1.70E-05	1.12E-03	-1.16E-03	4.75E-04	-5.09E-04
	3	X	0.022	0.222	-0.179	0.245	-0.201
		Y	-0.032	0.840	-0.905	0.161	-0.226
		ROT	-1.70E-05	1.12E-03	-1.16E-03	4.75E-04	-5.09E-04
	4	X	0.022	0.222	-0.179	0.245	-0.201
		Y	-0.037	1.124	-1.197	0.202	-0.276
		ROT	-1.70E-05	1.12E-03	-1.16E-03	4.75E-04	-5.09E-04
	5	X	0.022	0.222	-0.179	0.245	-0.201
		Y	-0.041	1.410	-1.493	0.289	-0.372
		ROT	-1.70E-05	1.12E-03	-1.16E-03	4.75E-04	-5.09E-04
5	1	X	0.016	0.207	-0.176	0.197	-0.165
		Y	-0.018	0.313	-0.349	0.191	-0.227
		ROT	-1.30E-05	1.10E-03	-1.13E-03	4.38E-04	-4.64E-04
	2	X	0.016	0.207	-0.176	0.197	-0.165
		Y	-0.021	0.534	-0.575	0.140	-0.181
		ROT	-1.30E-05	1.10E-03	-1.13E-03	4.38E-04	-4.64E-04
	3	X	0.016	0.207	-0.176	0.197	-0.165
		Y	-0.023	0.763	-0.810	0.139	-0.185
		ROT	-1.30E-05	1.10E-03	-1.13E-03	4.38E-04	-4.64E-04
	4	X	0.016	0.207	-0.176	0.197	-0.165
		Y	-0.027	1.043	-1.096	0.201	-0.254
		ROT	-1.30E-05	1.10E-03	-1.13E-03	4.39E-04	-4.64E-04
	5	X	0.016	0.207	-0.176	0.197	-0.165
		Y	-0.030	1.324	-1.384	0.294	-0.353
		ROT	-1.30E-05	1.10E-03	-1.13E-03	4.38E-04	-4.64E-04

(continued)

Table E.7 Lateral displacements at floor nodes under various loads (Case A) (continued)

Floor Level	Floor Node	Direction	Load Combination				
			Static	Static + N115W	Static - N115W	Static + N25W	Static - N25W
4	1	X	0.010	0.194	-0.173	0.150	-0.130
		Y	-0.011	0.240	-0.262	0.132	-0.155
		ROT	-9.00E-06	1.07E-03	-1.08E-03	3.99E-04	-4.16E-04
	2	X	0.010	0.194	-0.173	0.150	-0.130
		Y	-0.013	0.458	-0.484	0.100	-0.126
		ROT	-9.00E-06	1.07E-03	-1.08E-03	3.99E-04	-4.16E-04
	3	X	0.010	0.194	-0.173	0.150	-0.130
		Y	-0.015	0.682	-0.712	0.127	-0.156
		ROT	-9.00E-06	1.07E-03	-1.08E-03	3.99E-04	-4.16E-04
	4	X	0.010	0.194	-0.173	0.150	-0.130
		Y	-0.017	0.955	-0.990	0.204	-0.239
		ROT	-9.00E-06	1.07E-03	-1.08E-03	3.99E-04	-4.17E-04
	5	X	0.010	0.194	-0.173	0.150	-0.130
		Y	-0.019	1.229	-1.268	0.297	-0.336
		ROT	-9.00E-06	1.07E-03	-1.08E-03	3.99E-04	-4.16E-04
3	1	X	0.005	0.160	-0.149	0.103	-0.093
		Y	-0.005	0.120	-0.130	0.093	-0.103
		ROT	-6.00E-06	1.02E-03	-1.03E-03	3.58E-04	-3.70E-04
	2	X	0.005	0.160	-0.149	0.103	-0.093
		Y	-0.006	0.353	-0.365	0.070	-0.082
		ROT	-6.00E-06	1.02E-03	-1.03E-03	3.58E-04	-3.70E-04
	3	X	0.005	0.160	-0.149	0.103	-0.093
		Y	-0.008	0.595	-0.610	0.123	-0.138
		ROT	-6.00E-06	1.02E-03	-1.03E-03	3.58E-04	-3.70E-04
	4	X	0.005	0.160	-0.149	0.103	-0.093
		Y	-0.009	0.862	-0.880	0.208	-0.226
		ROT	-6.00E-06	1.02E-03	-1.03E-03	3.59E-04	-3.70E-04
	5	X	0.005	0.160	-0.149	0.103	-0.093
		Y	-0.010	1.129	-1.150	0.298	-0.319
		ROT	-6.00E-06	1.02E-03	-1.04E-03	3.58E-04	-3.70E-04

(continued)

Table E.7 Lateral displacements at floor nodes under various loads (Case A) (continued)

Floor Level	Floor Node	Direction	Load Combination				
			Static	Static + N115W	Static - N115W	Static + N25W	Static - N25W
2	1	X	0.003	0.144	-0.138	0.079	-0.073
		Y	-0.003	0.209	-0.215	0.049	-0.054
		ROT	-4.00E-06	7.16E-04	-7.23E-04	2.38E-04	-2.45E-04
	2	X	0.003	0.023	-0.016	0.049	-0.042
		Y	-0.005	0.653	-0.662	0.158	-0.168
		ROT	-4.00E-06	7.17E-04	-7.24E-04	2.39E-04	-2.46E-04
1	1	X	0.001	0.006	-0.004	0.016	-0.014
		Y	-0.001	0.062	-0.063	0.028	-0.029
		ROT	-2.00E-06	1.39E-04	-1.43E-04	4.30E-05	-4.70E-05
	2	X	0.001	0.006	-0.004	0.016	-0.014
		Y	-0.001	0.093	-0.095	0.031	-0.033
		ROT	-2.00E-06	1.39E-04	-1.43E-04	4.30E-05	-4.70E-05
	3	X	0.001	0.006	-0.004	0.016	-0.014
		Y	-0.001	0.125	-0.128	0.037	-0.040
		ROT	-2.00E-06	1.39E-04	-1.42E-04	4.30E-05	-4.70E-05
	4	X	0.001	0.006	-0.004	0.016	-0.014
		Y	-0.002	0.162	-0.166	0.046	-0.050
		ROT	-2.00E-06	1.37E-04	-1.41E-04	4.30E-05	-4.70E-05
	5	X	0.001	0.006	-0.004	0.016	-0.014
		Y	-0.002	0.200	-0.205	0.056	-0.061
		ROT	-2.00E-06	1.39E-04	-1.42E-04	4.30E-05	-4.70E-05

Table E.8 Lateral displacements at floor nodes under various loads (Case D)

10	1	X	0.025	0.711	-0.661	0.761	-0.710
		Y	-0.046	0.730	-0.822	0.818	-0.910
		ROT	-3.36E-04	6.05E-03	-6.72E-03	1.36E-03	-2.03E-03
	2	X	0.025	0.711	-0.661	0.761	-0.710
		Y	-0.119	1.576	-1.815	0.588	-0.827
		ROT	-3.36E-04	6.05E-03	-6.72E-03	1.36E-03	-2.04E-03
	3	X	0.025	0.711	-0.661	0.761	-0.710
		Y	-0.193	2.814	-3.200	0.536	-0.921
		ROT	-3.36E-04	6.05E-03	-6.72E-03	1.36E-03	-2.04E-03
	4	X	0.025	0.711	-0.661	0.761	-0.710
		Y	-0.278	4.317	-4.872	0.677	-1.233
		ROT	-3.35E-04	6.06E-03	-6.73E-03	1.36E-03	-2.03E-03
	5	X	0.025	0.711	-0.661	0.761	-0.710
		Y	-0.363	5.837	-6.563	0.927	-1.654
		ROT	-3.36E-04	6.05E-03	-6.72E-03	1.36E-03	-2.04E-03
9	1	X	0.014	0.711	-0.683	0.733	-0.706
		Y	-0.037	0.706	-0.781	0.795	-0.869
		ROT	-3.25E-04	6.04E-03	-6.69E-03	1.36E-03	-2.01E-03
	2	X	0.014	0.711	-0.683	0.733	-0.706
		Y	-0.108	1.561	-1.778	0.573	-0.790
		ROT	-3.25E-04	6.04E-03	-6.69E-03	1.36E-03	-2.01E-03
	3	X	0.014	0.711	-0.683	0.733	-0.706
		Y	-0.179	2.801	-3.160	0.532	-0.891
		ROT	-3.25E-04	6.04E-03	-6.68E-03	1.36E-03	-2.01E-03
	4	X	0.014	0.711	-0.683	0.733	-0.706
		Y	-0.262	4.302	-4.825	0.684	-1.207
		ROT	-3.25E-04	6.03E-03	-6.69E-03	1.36E-03	-2.01E-03
	5	X	0.014	0.711	-0.683	0.733	-0.706
		Y	-0.344	5.819	-6.507	0.939	-1.627
		ROT	-3.25E-04	6.04E-03	-6.68E-03	1.36E-03	-2.01E-03

(continued)

Table E.8 Lateral displacements at floor nodes under various loads (Case D) (continued)

Floor Level	Floor Node	Direction	Load Combination				
			Static	Static + N115W	Static - N115W	Static + N25W	Static - N25W
8	1	X	0.012	0.632	-0.609	0.664	-0.640
		Y	-0.028	0.637	-0.693	0.719	-0.775
		ROT	-2.50E-04	5.42E-03	-5.92E-03	1.23E-03	-1.73E-03
	2	X	0.012	0.632	-0.609	0.664	-0.640
		Y	-0.082	1.389	-1.552	0.530	-0.694
		ROT	-2.50E-04	5.43E-03	-5.93E-03	1.24E-03	-1.74E-03
	3	X	0.012	0.632	-0.609	0.664	-0.640
		Y	-0.135	2.478	-2.748	0.493	-0.763
		ROT	-2.49E-04	5.42E-03	-5.92E-03	1.23E-03	-1.73E-03
	4	X	0.012	0.632	-0.609	0.664	-0.640
		Y	-0.199	3.846	-4.244	0.624	-1.023
		ROT	-2.49E-04	5.44E-03	-5.94E-03	1.25E-03	-1.74E-03
	5	X	0.012	0.632	-0.609	0.664	-0.640
		Y	-0.264	5.227	-5.754	0.856	-1.383
		ROT	-2.49E-04	5.42E-03	-5.91E-03	1.23E-03	-1.73E-03
7	1	X	0.011	0.546	-0.524	0.569	-0.548
		Y	-0.021	0.541	-0.584	0.607	-0.650
		ROT	-1.77E-04	4.57E-03	-4.92E-03	1.04E-03	-1.39E-03
	2	X	0.011	0.545	-0.524	0.569	-0.548
		Y	-0.059	1.193	-1.312	0.460	-0.579
		ROT	-1.78E-04	4.58E-03	-4.94E-03	1.05E-03	-1.41E-03
	3	X	0.011	0.546	-0.524	0.569	-0.548
		Y	-0.097	2.112	-2.306	0.434	-0.628
		ROT	-1.77E-04	4.56E-03	-4.92E-03	1.04E-03	-1.40E-03
	4	X	0.011	0.545	-0.524	0.569	-0.548
		Y	-0.143	3.264	-3.549	0.546	-0.831
		ROT	-1.77E-04	4.58E-03	-4.94E-03	1.05E-03	-1.41E-03
	5	X	0.011	0.545	-0.524	0.569	-0.548
		Y	-0.188	4.427	-4.804	0.740	-1.117
		ROT	-1.77E-04	4.56E-03	-4.91E-03	1.04E-03	-1.39E-03

(continued)

Table E.8 Lateral displacements at floor nodes under various loads (Case D) (continued)

Floor Level	Floor Node	Direction	Load Combinatin				
			Static	Static + N115W	Static - N115W	Static + N25W	Static - N25W
6	1	X	0.010	0.453	-0.434	0.457	-0.438
		Y	-0.016	0.429	-0.461	0.473	-0.505
		ROT	-1.18E-04	3.66E-03	-3.90E-03	8.39E-04	-1.07E-03
	2	X	0.010	0.453	-0.434	0.457	-0.438
		Y	-0.041	0.980	-1.062	0.373	-0.455
		ROT	-1.18E-04	3.67E-03	-3.90E-03	8.52E-04	-1.09E-03
	3	X	0.010	0.453	-0.434	0.457	-0.438
		Y	-0.066	1.723	-1.855	0.369	-0.501
		ROT	-1.18E-04	3.66E-03	-3.90E-03	8.44E-04	-1.08E-03
	4	X	0.010	0.453	-0.434	0.457	-0.438
		Y	-0.097	2.648	-2.842	0.470	-0.663
		ROT	-1.18E-04	3.67E-03	-3.91E-03	8.53E-04	-1.09E-03
	5	X	0.010	0.453	-0.434	0.457	-0.438
		Y	-0.127	3.582	-3.837	0.631	-0.885
		ROT	-1.18E-04	3.66E-03	-3.89E-03	8.43E-04	-1.08E-03
5	1	X	0.008	0.350	-0.333	0.333	-0.316
		Y	-0.011	0.313	-0.336	0.330	-0.353
		ROT	-6.80E-05	2.69E-03	-2.82E-03	6.34E-04	-7.69E-04
	2	X	0.008	0.350	-0.333	0.333	-0.316
		Y	-0.026	0.748	-0.800	0.278	-0.330
		ROT	-6.80E-05	2.69E-03	-2.82E-03	6.44E-04	-7.80E-04
	3	X	0.008	0.350	-0.333	0.333	-0.316
		Y	-0.040	1.299	-1.379	0.298	-0.379
		ROT	-6.80E-05	2.69E-03	-2.82E-03	6.39E-04	-7.75E-04
	4	X	0.008	0.350	-0.333	0.333	-0.316
		Y	-0.058	1.980	-2.096	0.389	-0.505
		ROT	-6.90E-05	2.70E-03	-2.83E-03	6.48E-04	-7.85E-04
	5	X	0.008	0.350	-0.333	0.333	-0.316
		Y	-0.076	2.667	-2.818	0.518	-0.669
		ROT	-6.80E-05	2.69E-03	-2.82E-03	6.39E-04	-7.75E-04

(continued)

Table E.8 Lateral displacements at floor nodes under various loads (Case D) (continued)

Floor Level	Floor Node	Direction	Load Combination				
			Static	Static + N115W	Static - N115W	Static + N25W	Static - N25w
4	1	X	0.007	0.242	-0.228	0.207	-0.193
		Y	-0.008	0.207	-0.222	0.195	-0.211
		ROT	-3.00E-05	1.71E-03	-1.77E-03	4.27E-04	-4.86E-04
	2	X	0.007	0.242	-0.228	0.207	-0.193
		Y	-0.014	0.510	-0.538	0.185	-0.214
		ROT	-3.00E-05	1.70E-03	-1.76E-03	4.34E-04	-4.95E-04
	3	X	0.007	0.242	-0.228	0.207	-0.193
		Y	-0.020	0.866	-0.906	0.221	-0.261
		ROT	-3.00E-05	1.71E-03	-1.77E-03	4.33E-04	-4.93E-04
	4	X	0.007	0.242	-0.228	0.207	-0.192
		Y	-0.028	1.301	-1.357	0.297	-0.353
		ROT	-3.10E-05	1.71E-03	-1.78E-03	4.40E-04	-5.03E-04
	5	X	0.007	0.242	-0.228	0.207	-0.192
		Y	-0.036	1.739	-1.811	0.390	-0.461
		ROT	-3.00E-05	1.71E-03	-1.77E-03	4.33E-04	-4.94E-04
3	1	X	0.005	0.128	-0.118	0.092	-0.082
		Y	-0.005	0.098	-0.107	0.098	-0.107
		ROT	-6.00E-06	8.24E-04	-8.35E-04	2.46E-04	-2.58E-04
	2	X	0.005	0.128	-0.118	0.092	-0.082
		Y	-0.006	0.264	-0.276	0.105	-0.117
		ROT	-6.00E-06	8.25E-04	-8.36E-04	2.46E-04	-2.58E-04
	3	X	0.005	0.127	-0.117	0.092	-0.082
		Y	-0.007	0.456	-0.471	0.139	-0.153
		ROT	-7.00E-06	8.49E-04	-8.62E-04	2.45E-04	-2.58E-04
	4	X	0.005	0.127	-0.117	0.092	-0.082
		Y	-0.009	0.676	-0.694	0.190	-0.208
		ROT	-7.00E-06	8.48E-04	-8.61E-04	2.49E-04	-2.62E-04
	5	X	0.005	0.127	-0.117	0.092	-0.082
		Y	-0.011	0.897	-0.919	0.247	-0.269
		ROT	-6.00E-06	8.49E-04	-8.62E-04	2.45E-04	-2.58E-04

(continued)

Table E.8 Lateral displacements at floor nodes under various loads (Case D) (continued)

Floor Level	Floor Node	Direction	Load Combination				
			Static	Static + N115W	Static - N115W	Static + N25W	Static - N25W
2	1	X	0.002	0.111	-0.106	0.064	-0.059
		Y	-0.003	0.155	-0.162	0.071	-0.077
		ROT	-6.00E-06	5.50E-04	-5.61E-04	1.64E-04	-1.75E-04
	2	X	0.005	0.034	-0.024	0.047	-0.037
		Y	-0.003	0.502	-0.509	0.151	-0.157
		ROT	1.40E-05	7.27E-04	-6.98E-04	2.33E-04	-2.04E-04
1	1	X	0.001	0.008	-0.006	0.012	-0.010
		Y	-0.001	0.046	-0.048	0.033	-0.034
		ROT	-2.00E-06	1.01E-04	-1.06E-04	3.80E-05	-4.30E-05
	2	X	0.001	0.008	-0.006	0.012	-0.010
		Y	-0.001	0.068	-0.071	0.041	-0.043
		ROT	-2.00E-06	1.01E-04	-1.06E-04	3.80E-05	-4.30E-05
	3	X	0.001	0.008	-0.006	0.012	-0.010
		Y	-0.002	0.091	-0.095	0.049	-0.053
		ROT	-2.00E-06	1.01E-04	-1.06E-04	3.80E-05	-4.30E-05
	4	X	0.001	0.008	-0.006	0.012	-0.010
		Y	-0.002	0.118	-0.123	0.059	-0.064
		ROT	-2.00E-06	1.00E-04	-1.05E-04	3.80E-05	-4.30E-05
	5	X	0.001	0.008	-0.006	0.012	-0.010
		Y	-0.003	0.145	-0.151	0.069	-0.075
		ROT	-2.00E-06	1.01E-04	-1.06E-04	3.80E-05	-4.30E-05

E.5 Typical Input File

```
WOOD14.INP CASE A
INPUT FILE
STRUCTURE
*MATERIAL
1 S 36.
2 1600. 640.
3 2550. 1090.
4 1500. 625.
5 C 3000.
*GEOMETRY
**COLUMN LOCATIONS
1 2 1 0. 0. 90. 0. 178.
3 4 1 0. 303. 90. 0. 462.
5 6 1 57. 178. 90. 57. 240.
7 8 1 57. 303. 90. 130. 0.
9 10 1 130. 178. 90. 130. 240.
11 12 1 130. 303. 90. 130. 462.
13 14 1 160. 462. 90. 305. 0.
15 16 1 305. 178. 90. 305. 240.
17 18 1 305. 303. 90. 305. 408.
19 20 1 305. 462. 90. 456. 0.
21 22 1 478. 178. 90. 480. 240.
23 24 1 480. 408. 90. 480. 462.
25 26 1 651. 0. 90. 651. 178.
27 28 1 534. 240. 90. 634. 310.
29 30 1 634. 408. 90. 634. 462.
31 32 1 806. 0. 90. 806. 178.
33 34 1 806. 310. 90. 806. 354.
35 35 0 806. 462. 90. 806. 462.
36 37 1 1002. 0. 0. 1002. 162.
38 39 1 1002. 310. 0. 1002. 462.
**BAYS
1 3 1 1 2 1
4 5 1 5 6 1
6 9 1 8 9 1
10 14 1 14 15 1
15 18 1 20 21 1
19 23 1 25 26 1
24 27 1 31 32 1
28 30 1 36 37 1
31 31 0 1 8 0
32 33 1 8 14 6
34 34 0 20 25 0
35 35 0 25 31 0
36 36 0 31 36 0
37 37 0 2 5 0
38 38 0 5 9 0
39 40 1 9 15 6
41 41 0 21 26 0
42 42 0 26 32 0
43 43 0 32 37 0
44 44 0 6 10 0
45 46 1 10 16 6
47 47 0 22 27 0
48 49 1 3 7 4
50 50 0 11 17 0
51 52 1 28 33 5
53 55 2 17 23 6
54 54 0 18 23 0
56 56 0 29 34 0
57 57 0 34 38 0
58 58 0 4 12 0
59 59 0 12 13 0
60 60 0 13 19 0
61 61 0 19 24 0
62 62 0 24 30 0
63 63 0 30 35 0
64 64 0 35 39 0
**STORY HEIGHTS
1 2 123.
3 3 93.
4 9 123.
10 10 84.
*ELEMENTS
**COLUMNS
1 1 56.9 3.8 .25 5.34
2 1 110.39 31.17 .425 11.36
3 1 126.61 37.44 .63 12.94
4 1 156.56 49.98 1.31 13.
5 1 109.7 37. .5 9.12
**BEAMS
1 1 21.8 1.8 .13 3.61
2 1 215.8 9.5 .71 9.26
3 1 56.9 3.8 .26 5.34
4 1 268.9 13.8 1.39 11.84
5 1 441.8 14.6 1.23 12.49
6 1 122.1 6.9 .47 7.38
7 1 145.8 8.5 1.09 10.22
8 1 1. .12 .1 .1
9 1 32.6 1.32 0.13 3.38
**FBEAMS
1 4 12326. 693. 577.
2 4 7.05 2.67 2.22
**WALLS
1 2 2
3 .18 .75 .07
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0.
2 2 3
5 .05 .19 .1 .56 .1
3 .21 .55 .24
1 5 1 3 8.
2 2 2 2 0.
4 4 2 2 0.
3 2 2
3 .16 .69 .15
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0.
4 2 3
5 .06 .11 .17 .54 .12
3 .21 .55 .24
1 5 1 3 8.
2 2 2 2 0.
4 4 2 2 0.
5 2 2
3 .16 .67 .17
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0
6 2 1
1
1
1 1 1 1 8.
7 2 2
3 .24 .7 .06
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0.
8 2 2
3 .14 .77 .09
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0.
9 2 2
3 .11 .63 .26
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0.
10 2 2
3 .39 .22 .39
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0.
11 2 2
3 .41 .38 .21
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0.
```

12 2 2
3 .21 .38 .41
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0.
13 2 2
3 .58 .25 .17
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0.
14 2 2
3 .37 .31 .32
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0.
15 2 2
3 .38 .28 .34
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0.
16 2 3
5 .17 .54 .1 .14 .05
3 .21 .55 .24
1 5 1 3 8.
2 2 2 2 0.
4 4 2 2 0.
17 2 2
3 .19 .62 .19
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0.
18 2 2
3 .53 .23 .24
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0.
19 2 2
3 .34 .2 .46
3 .21 .55 .24
1 3 1 3 8.
2 2 2 2 0.
20 3 1
1
1
1 1 1 1 .25
21 2 1
1
1
1 1 1 1 8.
22 2 1
3 .39 .22 .39
1
1 3 1 1 8.
23 2 1
3 .58 .25 .17
1
1 3 1 1 8.
24 2 1
3 .21 .38 .41
1
1 3 1 1 8.
25 2 1
3 .24 .70 .06
1
1 3 1 1 8.
26 2 1
3 .14 .77 .09
1
1 3 1 1 8.
27 2 1
3 .11 .63 .26
1
1 3 1 1 8.
28 2 1
3 .18 .75 .07
1
1 3 1 1 8.
29 2 1
5 .05 .19 .1 .56 .1
1
1 5 1 1 8.
30 2 1
3 .16 .69 .15
1
1 3 1 1 8.
31 2 1
5 .06 .11 .17 .54 .12
1
1 5 1 1 8.
32 2 1
3 .16 .67 .17
1
1 3 1 1 8.
33 2 1
5 .17 .54 .1 .14 .05
1
1 5 1 1 8.
34 2 1
3 .19 .62 .19
1
1 3 1 1 8.
35 2 1
3 .37 .31 .32
1
1 3 1 1 8.
36 2 1
3 .38 .28 .34
1
1 3 1 1 8.
37 2 1
3 .53 .23 .24
1
1 3 1 1 8.
38 2 1
3 .34 .20 .46
1
1 3 1 1 8.
39 2 1
1
3 .21 .55 .24
1 1 1 3 8.
40 3 1
1
3 .21 .55 .24
1 1 1 3 .25
41 2 1
3 .41 .38 .21
1
1 3 1 1 8.
42 2 1
1
3 .21 .55 .24
1 1 1 3 8.
43 2 1
1
1
1 1 1 1 4.
44 5 1
3 .39 .22 .39
1
1 3 1 1 8.
45 5 1
1
1
1 1 1 1 8.
46 5 1
3 .58 .25 .17
1
1 3 1 1 8.
47 5 1
3 .37 .31 .32
1
1 3 1 1 8.
48 5 1
3 .38 .28 .34
1

```
1 3 1 1 8.
49 5 1
1
1
1 1 1 1 8.
50 5 1
3 .53 .23 .24
1
1 3 1 1 8.
51 5 1
3 .34 .20 .46
1
1 3 1 1 8.
52 2 1
3 .37 .31 .32
1
1 3 1 1 12.
53 2 1
1
1
1 1 1 1 12.
54 2 1
3 .38 .28 .34
1
1 3 1 1 12.
55 2 1
1
1
1 1 1 1 12.
56 2 1
3 .53 .23 .24
1
1 3 1 1 12.
57 2 1
3 .34 .20 .46
1
1 3 1 1 12.
58 2 1
1
1
1 1 1 1 1.
**FLOORS
1
N 1 0. 238. .1304 4886.
N 2 230. 238. .2607 9768.
N 3 460. 238. .2410 9030.
N 4 731. 238. .2213 8292.
N 5 1002. 238. .1107 4148.
M 1 1 2 1
M 2 2 3 1
M 3 3 4 1
M 4 4 5 1
S 1 4 1 1
S 5 13 1 0 1
S 14 20 1 0 2
S 21 30 1 0 3
S 31 35 1 0 4
S 36 39 1 5
E
2
N 1 167.5 205. .30835 3926.
N 2 818. 386. .2139 1963.
M 1 1 2 2
S 1 19 1 1
S 28 30 1 2
S 33 35 1 2
S 38 39 1 2
E
3
N 1 0. 253. .0969 3580.
N 2 239. 253. .1939 7164.
N 3 478. 253. .1854 6850.
N 4 740. 253. .1768 6532.
N 5 1002. 253. .0885 3270.
M 1 1 2 1
M 2 2 3 1
M 3 3 4 1
M 4 4 5 1
S 1 4 1 1
S 5 13 1 0 1
S 14 19 1 0 2
S 20 20 0 3
S 21 30 1 0 3
S 31 35 1 0 4
S 36 39 1 5
E
4
N 1 57. 233. .1128 6184.
N 2 271.5 233. .2255 12363.
N 3 486. 233. .2065 11321.
N 4 744. 233. .1875 10280.
N 5 1002. 233. .0937 5137.
M 1 1 2 1
M 2 2 3 1
M 3 3 4 1
M 4 4 5 1
S 1 7 1 1
S 8 13 1 0 1
S 14 24 1 0 2
S 25 30 1 0 3
S 31 35 1 0 4
S 36 39 1 5
E
5
N 1 57. 230. .1039 5899.
N 2 275.5 230. .2077 11792.
N 3 494. 230. .1933 10975.
N 4 748. 230. .1787 10146.
N 5 1002. 230. .0894 5076.
M 1 1 2 1
M 2 2 3 1
M 3 3 4 1
M 4 4 5 1
S 1 7 1 1
S 8 13 1 0 1
S 14 24 1 0 2
S 25 30 1 0 3
S 31 35 1 0 4
S 36 39 1 5
E
6
N 1 57. 230. .0377 2549.
N 2 275.5 230. .0754 5098.
N 3 494. 230 .0702 4746.
N 4 748. 230. .0649 4388.
N 5 1002. 230. .0324 2190.
M 1 1 2 1
M 2 2 3 1
M 3 3 4 1
M 4 4 5 1
S 1 7 1 1
S 8 13 1 0 1
S 14 24 1 0 2
S 25 30 1 0 3
S 31 35 1 0 4
S 36 39 1 5
E
**FOUNDATION SPRINGS
1 7670. 7468. 9095. 1.E12 1.E12 1.E12
2 1.E12 9600. 8863. 1.E12 1.E12 1.E12
3 9900. 1.E12 9363. 1.E12 1.E12 1.E12
4 10400. 1.E12 9963. 1.E12 1.E12 1.E12
5 7010. 5039. 5463. 1.E12 1.E12 1.E12
6 7400. 1.E12 5963. 1.E12 1.E12 1.E12
7 4500. 1.E12 5650. 1.E12 1.E12 1.E12
8 7250. 1.E12 9500. 1.E12 1.E12 1.E12
9 1.E12 1.E12 3137. 0. 0. 1.E12
*BUILD
**COLUMNS
1 2 1
1 1 2 2
2 2 2 1
3 3 2
4 7 5
8 10 1
3 4 1
```

1 1 3 2
2 2 3 1
3 3 3
4 10 5
8 8 0
1 1 3 2
2 2 3 1
3 3 3
4 7 5
8 10 1
9 11 2
1 3 2
4 7 5
8 10 1
13 13 0
1 1 3 2
2 2 3 1
3 3 3
4 10 5
14 14 0 8
15 15 0 9
17 17 0
1 3 2
4 7 5
8 10 1
19 19 0
1 1 3 2
2 2 3 1
3 3 3
4 7 5
8 10 1
20 20 0 8
21 21 0 9
23 23 0
1 3 4
4 7 5
8 10 1
25 25 0 8
26 26 0 9
27 27 0
1 3 1 2
10 10 1 2
29 29 0 9
30 30 0 1
31 31 0 8
32 32 0
1 3 4
4 7 5
8 10 1
34 34 0 9
35 35 0
1 1 4 2
2 2 4 1
3 3 4
4 7 5
8 10 1
36 36 0 8
37 38 1
1 3 3
4 7 5
8 10 1
39 39 0 19
**BEAMS
1 1 0
1 1 2 1 1
2 10 6 1 1
2 2 0
1 2 3 1 1
5 5 3 1 1
7 7 3 1 1
9 9 3 1 1
E
3 3 0 1
4 4 0
2 10 6 1 0
5 5 0
2 10 6 0 1
6 9 3
1 10 1 1 1
7 7 0
1 10 1 1 0
8 8 0
1 10 1 0 1
10 10 0 6
11 11 0 7
12 12 0 8
13 13 0 7
14 14 0 8
15 15 0
1 1 1 1 1
2 2 3 1 1
3 10 1 1 1
16 16 0 7
17 17 0 8
18 18 0
1 1 1 1 1
E
19 19 0 15
20 20 0 6
21 21 0 7
22 22 0 8
23 23 0
1 1 1 1 1
2 10 6 1 1
24 24 0 1
25 25 0
1 1 1 1 0
2 10 6 1 0
26 26 0
1 1 1 0 1
2 10 6 0 1
27 27 0 23
28 30 1
1 2 6 1 1
3 10 5 1 1
31 33 1
1 1 2 1 1
2 2 6 1 1
3 3 9
4 10 6 1 1
34 34 0
1 1 4 1 1
2 2 7 1 1
3 3 9
4 10 7 1 1
35 35 0 31
36 36 0 34
37 37 0
1 1 2 1 0
2 10 6 1 0
38 38 0
1 1 2 0 1
2 10 6 0 1
39 39 0 1
40 41 1
1 1 4 1 1
2 10 7 1 1
42 42 0
1 2 6 1 1
3 10 3 1 1
43 43 0
1 1 4 1 1
2 10 1 1 1
44 47 1
3 3 2 1 1
E
48 48 0 37
49 49 0 38
50 50 0 1
51 51 0
2 2 1 1 1
E
52 52 0
2 10 1 1 1
53 53 0
1 1 4 1 1

E
54 54 0
2 10 4 1 1
55 56 1 1
57 57 0
1 1 4 1 1
E
58 58 0
1 1 4 1 0
2 10 7 1 0
59 59 0
1 1 4 0 1
2 10 7 0 1
60 60 0 1
61 62 1
1 2 8 1 1
E
63 64 1 1
**WALLS
1 1 0
1 1 44
2 3 22
4 9 10
10 10 22
2 2 0
1 1 45
2 2 6
E
3 3 0
1 1 46
2 3 23
4 9 13
10 10 23
4 4 0
3 3 41
4 9 11
10 10 41
5 5 0
3 3 24
4 9 12
10 10 24
6 6 0
3 3 20
4 9 40
E
7 9 1
2 2 20
4 9 40
E
10 10 0
2 3 58
4 9 40
E
12 13 1
2 3 20
4 9 40
E
11 11 0
2 3 20
E
14 14 0
2 2 20
3 3 21
E
19 19 0
2 3 20
4 9 40
E
21 21 0 19
22 23 1
2 3 20
E
28 28 0
4 9 7
10 10 25
29 29 0
4 9 8
10 10 26
30 30 0
4 9 9
10 10 27
31 31 0
1 1 45
4 9 1
10 10 28
32 32 0
1 1 45
4 9 2
10 10 29
33 33 0
1 1 45
4 9 3
10 10 30
34 34 0
1 1 45
4 9 4
10 10 31
35 35 0
1 1 45
4 9 5
10 10 32
36 36 0 34
37 37 0
2 2 20
3 3 21
4 9 39
10 10 21
38 38 0
2 3 20
E
39 39 0
2 3 20
4 9 40
E
40 42 1
2 3 58
4 9 40
E
43 43 0
2 3 58
E
44 47 1 19
48 48 0
3 3 21
10 10 21
51 51 0
3 3 20
E
52 52 0
3 3 20
4 9 40
E
54 54 0
3 3 33
4 9 16
10 10 33
55 55 0
3 3 34
4 9 17
10 10 34
58 58 0
1 1 47
2 2 52
3 3 35
4 9 14
10 10 35
59 59 0
1 1 45
2 2 53
3 3 6
4 9 42
10 10 6
60 60 0
1 1 48
2 2 54
3 3 36

```
4 9 15
10 10 36
61 62 1
1 1 49
2 2 55
E
63 63 0
1 1 50
2 2 56
3 3 37
4 9 18
10 10 37
64 64 0
1 1 51
2 2 57
3 3 38
4 9 19
10 10 38
**FLOORS
1 1 1 0
2 2 2 0
3 3 3 0
4 4 8 1
5 9 9 0
6 10 10 0
**FOUNDATIONS
1 4 3 1 1
2 3 1 1 2
8 8 0 1 3
9 11 2 1 9
12 12 0 1 7
13 13 0 1 6
14 19 5 1 3
15 17 2 1 9
20 20 0 1 4
21 23 2 1 9
24 24 0 1 8
25 25 0 1 4
26 27 1 1 9
28 28 0 1 9
29 29 0 1 9
30 30 0 1 3
31 35 4 1 4
33 33 0 1 9
32 34 2 1 9
36 36 0 1 5
37 38 1 1 9
39 39 0 1 5
LOADS
*VERTICAL
**PATTERN
1 F R 0. 1.
2 U R 0.0 0.0
**GROUPS
1 1 .225 .932
2 1 .932 .932
3 1 .932 .225
4 1 .225 1.368
5 1 .368 1.164
6 1 1.164 1.421
7 1 1.421 1.421
8 1 1.4 .7
9 1 .225 .95
10 1 .95 .45
11 1 .451 1.07
12 1 1.07 .14
13 1 .14 .34
14 1 .34 .7
15 1 .7 1.5
16 1 .17 .9
17 1 .9 .9
18 1 .9 .17
19 1 .69 .0
20 1 .0 .18
21 1 .18 .19
22 1 .19 .61
23 1 .16 1.22
24 1 1.22 .16
25 1 .16 2.98
26 1 2.98 3.11
27 1 3.11 .16
28 1 .16 1.35
29 1 1.4 .16
30 1 0. 1.25
31 1 1.25 5.25
32 1 .16 2.0
33 1 .51 .8
34 1 .11 .7
35 1 .7 .11
36 1 .11 1.21
37 1 1.21 1.26
38 1 1.26 .38
39 1 .65 .65
40 1 .0 .61
41 1 .61 2.4
42 1 .11 .55
43 1 .11 1.06
44 1 .34 .44
45 1 .44 .38
46 1 0. .32
47 1 .32 0.
48 1 0. .28
49 1 .28 .28
50 1 .27 1.13
51 1 .08 .18
52 1 .18 0.
53 2 .05 .05
54 2 .06 .06
55 2 .04 .04
56 2 .07 .07
57 2 .08 .08
58 2 .01 .01
59 2 .005 .005
60 2 .02 .02
61 1 .26 0.
**BAY LOADING
1 1 0
1 1 57 1
2 2 54 16
3 8 54 23
9 9 53 34
10 10 58 46
E
2 2 0
1 1 57 2
2 2 58 17
E
3 3 0
1 1 57 3
2 2 54 18
3 8 54 24
9 9 53 35
10 10 58 47
E
4 4 0
3 8 54
9 9 53
10 10 59
E
5 5 0
3 8 54
9 9 53
10 10 59
E
28 28 0
3 8 53 25
9 9 53 36
10 10 59 40
E
29 29 0
3 8 53 26
9 9 53 37
10 10 59 39
E
30 30 0
3 8 53 27
```

```
9 9 53 38
10 10 59 19
E
31 31 0
1 1 60 4
3 8 53 28
9 9 53 34
10 10 59 48
E
32 32 0
1 1 58 5
3 8 53 7
9 9 53 39
10 10 59 49
E
33 33 0
1 1 58 6
3 8 53 7
9 9 53 39
10 10 59 49
E
34 34 0
1 1 58 7
3 8 54 7
9 9 53 39
10 10 59 49
E
35 35 0
1 1 58 7
3 8 53 7
9 9 53 39
10 10 59 49
E
36 36 0
1 1 60 8
3 8 54 29
9 9 53 35
10 10 59 61
E
54 54 0
3 8 55 30
9 9 53 40
10 10 59 48
E
55 55 0
3 8 55 31
9 9 53 41
10 10 59 50
E
58 58 0
1 1 57 9
2 2 54 16
3 8 56 23
9 9 53 42
10 10 59 48
E
59 59 0
1 1 57 10
2 2 54 17
3 8 57 12
9 9 53 35
10 10 59 61
E
60 60 0
1 1 57 11
2 2 54 18
3 8 56 32
9 9 53 43
10 10 59 40
E
61 61 0
1 1 56 12
2 2 58 19
E
62 62 0
1 1 56 13
2 2 58 20
E
63 63 0
1 1 56 14
2 2 54 21
3 8 56 33
9 9 53 44
10 10 59 51
E
64 64 0
1 1 57 15
2 2 54 22
3 8 56 18
9 9 53 45
10 10 59 52
E
*MODES
25 1000.
*DAMP
1 25 0.05
*SPECTRUM
132 1. 0.0
**USER
N115W RESPONSE SPECTRUM ALONG Y-AXIS
 .02,    67.10
 .04,    69.65
 .06,    76.99
 .08,    84.41
 .10,    79.14
 .12,    91.86
 .14,   114.87
 .16,   122.07
 .18,   118.41
 .20,   176.60
 .22,   213.93
 .24,   197.64
 .26,   201.88
 .28,   198.12
 .30,   161.85
 .32,   177.43
 .34,   171.49
 .36,   157.87
 .38,   139.63
 .40,   119.04
 .42,   147.88
 .44,   151.50
 .46,   153.03
 .48,   171.34
 .50,   181.45
 .52,   160.45
 .54,   146.68
 .56,   137.06
 .58,   130.13
 .60,   117.19
 .62,   105.35
 .64,   101.29
 .66,   100.95
 .68,    98.35
 .70,    92.56
 .72,    98.34
 .74,   101.97
 .76,   104.47
 .78,   103.96
 .80,    99.35
 .82,    91.18
 .84,    90.96
 .86,    92.49
 .88,    94.07
 .90,    96.63
 .92,   100.03
 .94,   104.68
 .96,   109.55
 .98,   113.92
1.00,   117.22
1.02,   119.67
1.04,   121.51
1.06,   131.17
1.08,   144.77
1.10,   156.42
1.12,   165.21
```

1.14, 170.59
1.16, 173.17
1.18, 178.77
1.20, 183.70
1.22, 186.42
1.24, 188.00
1.26, 189.18
1.28, 188.34
1.30, 185.60
1.32, 181.62
1.34, 176.20
1.36, 169.84
1.38, 163.22
1.40, 158.16
1.42, 152.91
1.44, 147.69
1.46, 142.15
1.48, 136.53
1.50, 130.70
1.52, 124.68
1.54, 118.50
1.56, 112.19
1.58, 105.75
1.60, 99.18
1.62, 95.08
1.64, 90.95
1.66, 86.82
1.68, 82.77
1.70, 78.67
1.72, 74.82
1.74, 72.11
1.76, 69.32
1.78, 66.58
1.80, 63.88
1.82, 61.13
1.84, 58.38
1.86, 55.75
1.88, 53.12
1.90, 50.50
1.92, 47.94
1.94, 45.47
1.96, 43.33
1.98, 41.75
2.00, 40.23
2.25, 24.61
2.50, 19.79
2.75, 19.34
3.00, 13.76
3.25, 9.85
3.50, 8.72
3.75, 8.51
4.00, 8.44
4.25, 7.72
4.50, 6.31
4.75, 4.77
5.00, 3.61
5.25, 3.98
5.50, 3.99
5.75, 3.86
6.00, 3.84
6.25, 4.04
6.50, 4.24
6.75, 4.21
7.00, 3.88
7.25, 3.57
7.50, 3.19
7.75, 2.81
8.00, 2.60
8.25, 2.34
8.50, 2.05
8.75, 1.81
9.00, 1.61
9.25, 1.50
9.50, 1.41
9.75, 1.32
10.00, 1.23
132 1. 90.0
**USER

N25W RESPONSE SPECTRUM ALONG X-AXIS
.02, 67.10
.04, 69.65
.06, 76.99
.08, 84.41
.10, 79.14
.12, 91.86
.14, 114.87
.16, 122.07
.18, 118.41
.20, 176.60
.22, 213.93
.24, 197.64
.26, 201.88
.28, 198.12
.30, 161.85
.32, 177.43
.34, 171.49
.36, 157.87
.38, 139.63
.40, 119.04
.42, 147.88
.44, 151.50
.46, 153.03
.48, 171.34
.50, 181.45
.52, 160.45
.54, 146.68
.56, 137.06
.58, 130.13
.60, 117.19
.62, 105.35
.64, 101.29
.66, 100.95
.68, 98.35
.70, 92.56
.72, 98.34
.74, 101.97
.76, 104.47
.78, 103.96
.80, 99.35
.82, 91.18
.84, 90.96
.86, 92.49
.88, 94.07
.90, 96.63
.92, 100.03
.94, 104.68
.96, 109.55
.98, 113.92
1.00, 117.22
1.02, 119.67
1.04, 121.51
1.06, 131.17
1.08, 144.77
1.10, 156.42
1.12, 165.21
1.14, 170.59
1.16, 173.17
1.18, 178.77
1.20, 183.70
1.22, 186.42
1.24, 188.00
1.26, 189.18
1.28, 188.34
1.30, 185.60
1.32, 181.62
1.34, 176.20
1.36, 169.84
1.38, 163.22
1.40, 158.16
1.42, 152.91
1.44, 147.69
1.46, 142.15
1.48, 136.53
1.50, 130.70
1.52, 124.68
1.54, 118.50

```
1.56,  112.19
1.58,  105.75
1.60,  99.18
1.62,  95.08
1.64,  90.95
1.66,  86.82
1.68,  82.77
1.70,  78.67
1.72,  74.82
1.74,  72.11
1.76,  69.32
1.78,  66.58
1.80,  63.88
1.82,  61.13
1.84,  58.38
1.86,  55.75
1.88,  53.12
1.90,  50.50
1.92,  47.94
1.94,  45.47
1.96,  43.33
1.98,  41.75
2.00,  40.23
2.25,  24.61
2.50,  19.79
2.75,  19.34
3.00,  13.76
3.25,  9.85
3.50,  8.72
3.75,  8.51
4.00,  8.44
4.25,  7.72
4.50,  6.31
4.75,  4.77
5.00,  3.61
5.25,  3.98
5.50,  3.99
5.75,  3.86
6.00,  3.84
6.25,  4.04
6.50,  4.24
6.75,  4.21
7.00,  3.88
7.25,  3.57
7.50,  3.19
7.75,  2.81
8.00,  2.60
8.25,  2.34
8.50,  2.05
8.75,  1.81
9.00,  1.61
9.25,  1.50
9.50,  1.41
9.75,  1.32
10.00,  1.23
*combinations
1. 1.
1. 1. 0. 0. 0. 0. 0. 0. 0. 0. 0. 0. 1.
1. 1. 0. 0. 0. 0. 0. 0. 0. 0. 0. 0. 0. 1.
SOLVE
OUTPUT
*FORCES
**NONE
*PROCESS
**WALLS
H 0 1 0 0 1 2 2 3
H 0 1 0 0 3 4 0 0
H 0 2
H 0 3 0 0 1 2 2 3
H 0 3 0 0 3 4 0 0
H 0 4 0 0 1 2 2 3
H 0 4 0 0 3 4 0 0
H 0 5 0 0 1 2 2 3
H 0 5 0 0 3 4 0 0
H 0 28 0 0 1 2 2 3
H 0 28 0 0 3 4 0 0
H 0 29 0 0 1 2 2 3
H 0 29 0 0 3 4 0 0
H 0 30 0 0 1 2 2 3
H 0 30 0 0 3 4 0 0
H 0 31 0 0 1 2 2 3
H 0 31 0 0 3 4 0 0
H 0 32 0 0 1 2 2 3
H 0 32 0 0 3 4 4 5
H 0 32 0 0 5 6 0 0
H 0 33 0 0 1 2 2 3
H 0 33 0 0 3 4 0 0
H 0 34 0 0 1 2 2 3
H 0 34 0 0 3 4 4 5
H 0 34 0 0 5 6 0 0
H 0 35 0 0 1 2 2 3
H 0 35 0 0 3 4 0 0
H 0 36 0 0 1 2 2 3
H 0 36 0 0 3 4 4 5
H 0 36 0 0 5 6 0 0
H 0 58 0 0 1 2 2 3
H 0 58 0 0 3 4 0 0
H 0 59
H 0 60 0 0 1 2 2 3
H 0 60 0 0 3 4 0 0
H 0 54 0 0 1 2 2 3
H 0 54 0 0 3 4 4 5
H 0 54 0 0 5 6 0 0
H 0 55 0 0 1 2 2 3
H 0 55 0 0 3 4 0 0
H 0 63 0 0 1 2 2 3
H 0 63 0 0 3 4 0 0
H 0 64 0 0 1 2 2 3
H 0 64 0 0 3 4 0 0
STOP
```

BULLETIN

The Occupational Safety & Health Administration (OSHA) has determined that the maximum safe load capacity on my butt is two (2) persons at one time (unless I install handrails or safety straps). As you have arrived sixth in line to ride my ass today, please take a number and wait your turn. Thank you.