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Two existing GSA buildings, one in Long Beach, California and one in Portland, Oregon, were subjected to low-level vibration tests to determine their dynamic properties and response frequencies. The measured dynamic properties of the buildings were incorporated into the computer models of the buildings and time-history analyses using these models were performed. Reasonable agreement between the measured and calculated response frequencies and deflected shapes were observed. The differences in calculated and measured response frequencies range from 3% to 32%. The larger difference is in the torsional response of the Portland building. This is probably due to the irregular geometry of this building. The models were then analyzed with past earthquake acceleration records used as source of excitations. The Portland building was subjected to three components of acceleration obtained from the November 1962 Portland earthquake. The Long Beach building was subjected to three components of acceleration obtained from the 1987-Whittier Narrows earthquake. The purpose of the analyses is to reveal building response under these realistic earthquake excitations, so that logical seismic instrumentation schemes can be developed for these buildings. The results of the analyses suggest that the response of the Portland building is influenced more by torsional and rocking motions, while the response of the Long Beach building is influenced mainly by translational modes. From the observed behavior of the buildings, a seismic instrumentation scheme is developed for each building, and a general guideline for seismic instrumentation in existing building is recommended.

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ABSTRACT

Two existing GSA buildings, one in Long Beach, California and one in Portland, Oregon, were subjected to low-level vibration tests to determine their dynamic properties and response frequencies. The measured dynamic properties of the buildings were incorporated into the computer models of the buildings and time-history analyses using these models were performed. Reasonable agreement between the measured and calculated response frequencies and deflected shapes were observed. The differences in calculated and measured response frequencies range from 3% to 32%. The larger difference is in the torsional response of the Portland building. This is probably due to the irregular geometry of this building. The models were then analyzed with past earthquake acceleration records used as source of excitations. The Portland building was subjected to three components of acceleration obtained from the November 1962 Portland earthquake. The Long Beach building was subjected to three components of acceleration obtained from the 1987-Whittier Narrows earthquake. The purpose of the analyses is to reveal building response under these realistic earthquake excitations, so that logical seismic instrumentation schemes can be developed for these buildings. The results of the analyses suggest that the response of the Portland building is influenced more by torsional and rocking motions, while the response of the Long Beach building is influenced mainly by translational modes. From the observed behavior of the buildings, a seismic instrumentation scheme is developed for each building, and a general guideline for seismic instrumentation in existing building is recommended.

Key words: Analytical model; buildings; dynamic; earthquake; frequency; ground acceleration; instrumentation; model; mode shape; seismometers, spectral density; vibration.

EXECUTIVE SUMMARY

As part of its effort to improve earthquake design of buildings in seismically active regions, the General Services Administration (GSA) has initiated a program to install strong motion instruments in existing and new buildings. As a pilot project, the Public Buildings Service (PBS)/GSA has sponsored the Center for Building Technology of the National Institute of Standards and Technology (NIST) to develop criteria for deploying strong motion instruments in new and existing Federal buildings. This includes procedures for determining an optimum number and location of instruments required for a particular building. The project also included installation of strong motion instruments in a selected building. Structural performance data obtained from strong motion instruments can give the designer useful information for verifying design assumptions. This information can be used to improve seismic design.

The Public Buildings Service selected two existing Federal buildings for this study. One is a prestressed concrete frame building, located in Portland, Oregon (Uniform Building Codes' seismic zone 2). The other is a steel frame building in Los Angeles, California (UBC seismic zone 4). NIST's technical approach included three parts, the first part included computer modeling of the two buildings by the finite element techniques, the second part included field vibration tests of the buildings, and the third part included verification of the models using the vibration test results, and time-history analyses of the models using past earthquake acceleration records. The field vibration tests, which were conducted jointly with the Branch of Geologic Risk Assessment of the U.S. Geological Survey, provided measures of the dynamic properties of the buildings (building's period and damping) which were incorporated into the computer models of the buildings for analysis.

Reasonable agreement between the analytical predictions and the test results were observed. Time-history analyses were performed using computer models of the buildings and past earthquake acceleration records as source of excitations to simulate the building's behavior under actual earthquake conditions. The model of the Portland building was analyzed using the acceleration records from the November 1962 Portland earthquake, and the model of the Long Beach building was analyzed using the acceleration records from the 1987 Whittier Narrows earthquake. The results of the analyses show that the two opposite ends in the long direction of the Portland building would undergo different motions, while the motions at the same two ends of the Long Beach building were essentially the same. This suggests that the Portland building can be considered as structure with flexible floors, i.e. more than six degrees of freedom (three translational and three rotational) are needed to fully characterize the motion of the floor in this building; and the Long Beach building can be considered as structure with rigid floors, i.e. six degrees of freedom (three translational and three rotational) are sufficient to characterize the motion of the floor in this building.

Based on this study, specific instrumentation schemes were developed for the two buildings. These instrumentation schemes are described in detail in Chapter 6 of this report. In summary, the recommended instrumentations schemes are as follows:

For the Portland building, a total of three sets of strong motion instruments are recommended to be placed at three elevations: the building foundation level (B1), floor 3, and the penthouse level. Each set of instruments consists of eight (8) uniaxial accelerometers.

For the Long Beach building, a total of three sets of strong motion instruments are recommended to be placed at the building foundation level, floor 4 and floor 8 levels. However, each set consists of only six (6) uniaxial instruments since the floors of the Long Beach building are rigid.

In addition, general guidelines which can be used to determine the appropriate number and location of strong motion instruments for existing buildings are recommended in Chapter 6. Finally, since the Long Beach building (UBC seismic zone 4) is more likely to experience strong motion than the Portland building (UBC seismic zone 2), it is recommended that this building be instrumented in accordance with the recommended instrumentation scheme for future study.

The results of this project have produced the following benefits to GSA:

- General guidelines for use in developing seismic instrumentation schemes for existing buildings. Data obtained from the instruments installed in a building can be used to monitor the building performance and to improve seismic design criteria.

- Analytical models of the Portland and the Long Beach buildings which can be used for future study.

- Detailed instrumentation schemes for the Portland and the Long Beach buildings and recommendations for implementation.

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SEISMIC INSTRUMENTATION OF EXISTING BUILDINGS

1. INTRODUCTION
1.1 Background

Because of the complexity in structural responses of modern high-rise buildings to earthquake excitations, prediction of a building's dynamic responses to an earthquake using simple analytical methods has become less reliable. Even with sophisticated mathematical models, the accuracy of prediction of the dynamic response of buildings with abrupt changes in stiffness or mass distributions is dependent upon accurate characterization of the buildings dynamic properties (mode shapes, damping) and the ground motion. Thus, beside analytical methods, long-term seismic instrumentation using accelerographs and accelerometers is increasingly desired for structural response studies. Prior to an earthquake, data obtained from a well-instrumented building permit the characterization of the building's structural dynamic properties and its elastic responses to low-level, ambient excitations. After an earthquake, these data permit the reconstruction of the ground motion and the actual building responses to the earthquake. The reconstructed responses can be used to determine the changes in structural performance, and to identify nonlinear behavior associated with high-level excitations or potential damage in the building. Thus, data obtained from seismic instrumentation are essential in (1) improving the understanding of the behavior of the instrumented structure under seismic loading, (2) assessing local damage in the building due to an earthquake and determining appropriate methodologies for repair and strengthening, and (3) evaluating the adequacy of the original earthquake-resistant design assumptions and identifying deficiencies in current design criteria.

From a structural engineering standpoint, it is desirable to have many recording instruments deployed in a building so that a comprehensive intepretation of the structural response can be performed. However, the cost of the equipment installation, maintenance, and data processing usually limits the number of instruments installed. Thus, it is more important and practical to deploy an optimum number of instruments at judiciously selected locations in the building so that sufficient and meaningful data essential for the reconstruction of the building's structural response to an earthquake can be recorded. The current code requirements pertaining to seismic instrumentation for buildings in the west coast specifies a minimum of three approved accelerographs, to be placed typically at the basement level, mid-height level, and near the top level of every building over six stories in height with an aggregate floor area of over 60,000 square feet, and every building over 10 stories in height regardless of floor area. The practicality of the current code recommendations regarding the number and locations of strong motion instruments has been discussed in many papers [3,4,5,6,7,13]. In general, most agree that the minimum number of instruments required by the codes is inadequate for capturing many possible dominant modes of vibration in a building, and thus is considered insufficient for structural response studies, especially where torsional and rocking motions, which are often dominant in medium-rise, irregularly-shaped buildings, are involved. For example, in order to determine the input ground motion to the building due to an earthquake, it would be necessary to install more than one tri-axial accelerograph at the basement level to obtain tri-directional shaking motions and rocking motions. The most desirable locations for the strong motion instruments can be determined based on anticipated or analytically predicted building responses to seismic excitation. However, certain locations such as the center of a floor slab can be ruled out immediately since it is difficult to distinguish signals from the secondary flexural vibration and the primary response motions at these locations.

This report describes the structural response studies of two GSA's buildings using low-level vibration testing and three-dimensional mathematical modeling. The mathematical models of the buildings, after being verified by field tests, are analyzed using past earthquake records as source of excitations to determine the buildings response under the actual earthquake conditions. The buildings response characteristics are used in identifying the most appropriate number and locations of seismic instruments needed for each building. The final objective and scope is explained in the next section.

1.2 Objectives and Scope

As part of its effort to improve earthquake design efficiency and life safety for building occupants in seismically active regions, the General Services Administration (GSA) has sponsored the Center for Building Technology of the National Institute of Standards and Technology (NIST) to conduct the current study with the following objectives:

1. **To develop criteria for locating strong motion instrumentation in Federal buildings, and the procedure for determining an optimum number of instruments required to determine the response of building to an actual earthquake.**

2. **To install instruments in one of the two selected buildings at recommended locations to obtain data for later use in developing techniques for improving earthquake resistance of existing buildings, in evaluating damage to Federal buildings following an earthquake and in repairing earthquake-damaged structures.**

Two federal buildings were selected by GSA for this study, one in Long Beach, California (UBC seismic zone 4), and the other in Portland, Oregon (UBC seismic zone 2). These buildings are refered to in this report as the Long Beach and the Portland buildings, respectively.

1.3 Technical Approach

NIST's technical approach involved the following initial steps. These initial steps are essential for achieving the overall objectives listed above:

1. Obtain and review the architectural and structural plans and specifications of the two selected buildings to study building layouts and structural properties. Make site visits to document non-structural information such as partitions that might effect the dynamic response of the buildings and any deviations from the building plans.

2. Develop three-dimensional finite element models of the two buildings.

3. Conduct low-level vibration tests on the buildings to determine in-situ dynamic properties such as damping values, frequencies and mode shapes. These measured dynamic properties are to be incorporated into the finite element models. Mode shapes, response frequencies obtained from vibration tests are compared with analytically predicted values to verify the validity of the computer models. The dynamic testings of the buildings are jointly conducted with the Branch of Geologic Risk Assessment (BGRA) of the U.S. Geological Survey (USGS).

4. After satisfactory model verification, the buildings are analyzed by subjecting them actual acceleration records of past earthquakes. The analytical dynamic responses of the buildings will help identifying the most desirable locations for strong-motion instrumentations in the buildings.

2. EXISTING GUIDELINES FOR SEISMIC INSTRUMENTATION

2.1 The 1988 Uniform Building Code

The 1988 Uniform Building Code (UBC) [1, section 2312(i)] recommends the followings:

1. In seismic zones 3 and 4 every building over six stories in height with an aggregate floor area of 60,000 square feet or more, and every building over 10 stories in height regardless of floor area, shall be provided with not less than three approved recording accelerographs.

2. The instruments shall be located in the basement, midportion, and near the top of the building. Each instrument shall be located so that access is maintained at all times and is unobstructed by room contents. A sign stating "MAINTAIN CLEAR ACCESS TO THIS INSTRUMENT" shall be posted in a conspicuous location.

3. Maintenance and service of the instruments shall be provided by the owner of the building, subject to the approval of the building official. Data produced by the instruments shall be made available to the building official upon his request.

2.2 The 1981 Los Angeles County Building Code

The 1981 Los Angeles County Building Code [2, chapter 23-General Design Requirement, section 2300] adopted earlier UBC earthquake recording instrumentation provisions and expanded it to include not only those buildings which are required by the UBC to have three triaxial accelerographs, but also all unusually shaped buildings in the Los Angeles county into the types of buildings which require at least three approved recording accelerographs.

It should be noted that neither of the selected buildings is required to be instrumented by the codes. Since the Portland building, despite having an irregular shape, is located in UBC seismic zone 2 and outside Los Angeles County; and the Long Beach building, despite being in UBC seismic zone 4, is symmetric with a largest floor area of 48,390 square feet, less than the 60,000 square feet specified by the codes.

3. BUILDING DESCRIPTIONS

3.1 The Portland Building

Figure 3.1 shows the shape and dimensions of the Portland building. Figure 3.2 shows plan view of a typical floor. Views of the North and South facings of the building are shown in Figures 3.3 (a) and (b).

The Portland building is a prestressed concrete office building which was constructed in 1984. The building is unsymmetrical in plan with 8 above-ground floors (level B-1 to level 7), 2 basement floors (levels B-2 and B-3), and a penthouse. The total height from the ground floor (level B-1) to the roof of the penthouse is 132.17 feet. The story height of the intermediate floors is typically 13.5 feet, except for the first story and the penthouse level which have story heights of 14.83 feet and 20.83 feet, respectively. The foundation is trapezoidal in shape, with an approximate foundation plan area of 78,640 square feet. The building also includes two shear walls, located toward the center, to provide shear resistance. One shear wall extends from the foundation all the way to the roof of the penthouse level. The other terminates at the penthouse level. The ratio of building height to base width (aspect ratio) is approximately 0.8. The building's exterior consists of marble stone plates and glass. At the time of testing, the building was fully occupied and was furnished with office furnitures.

The floor systems of the basement floors consist of reinforced concrete floors supported by a system of concrete joists and beams. The floor systems of the above-ground floors are prestressed in both orthogonal directions and supported by reinforced concrete perimeter beams. The floor systems are in turn supported by round and rectangular reinforced concrete columns, which are evenly spaced at 32 feet in the east-west direction and 30 feet in the north-south direction. Many perimeter columns in the north and west faces are terminated with increased elevation, resulting in a stepback look from the outside and smaller plan areas for floors above the termination elevations. This discontinuity also results in abrupt change of structural stiffness in both orthogonal directions of the building. The foundation system consists of 10 inches thick reinforced concrete perimeter walls and individual square or rectangular spread footings which are not tied together. The elevation difference between the grade at the east and west sides is approximately 17.1 feet. Typical bay length in the east-west direction is 32 feet and in the north-south direction is 30 feet. The north and west facings of the building are square, the south and east facings are continuously curved.

3.2 The Long Beach Building

Figure 3.4 shows the shape and dimensions of the Long Beach building. Plan view of a typical floor of this building is shown in Figure 3.5. Figure 3.6 shows view of the South facing of the building. Figure 3.7 shows view of the interior of the top floor.

The Long Beach building is symmetrical in plan with respect to the north-south direction, with a total of 8 floors above ground (floors 1 to 8), one basement floor, and a small mechanical level. The total building height, from ground floor (floor 1) to top of the mechanical room is 147.5 feet. Typical story height of the above-ground floors is 15 feet, except for the first story (between floor 1 and 2) which is 20 feet. The height between the basement floor and the first floor is 13.5 feet. The basement is surrounded by a 12-inch reinforced concrete perimeter wall, supported by reinforced concrete wall footing. The foundation is rectangular, 315.92 ft by 153.17 ft, with an approximate base area of 48,390 square feet. The long dimension of the building is in the east-west direction, with typical floor length of 305 feet. The short dimension is in the north-south direction, with typical floor width of 105 feet. Typical bay length in the east-west direction is 30 ft. The bay length in the north-south direction varies from 22.5 ft to 33.25 ft. The elevation difference between the grade in the east and west sides is approximately 13.5 feet. The building aspect ratio (height-to-base width ratio) is 0.82. The building is in UBC seismic region 4. At the time of testing, all structural aspects of the building had been completed. However, it was neither ready for occupancy nor furnished with any non-structural partitions or mechanical equipment. The elevators, to be installed in the elevator shafts located toward the center of the building, were not yet in place. This provided ideal conditions for low-level vibration testing and analytical modeling.

The typical floor system of this building consists of a composite reinforced concrete slab on cold-formed steel decking, which is supported by wide-flange steel joists and beams. Lateral load is resisted by a moment resisting steel frame, consisting of steel columns. The columns, made of either commercially available wide-flange or built-up sections, are connected at every other story by welded connections and have smaller section with increased elevation. Some of the perimeter columns are supported by the foundation footing, others including all interior columns are supported by individual spread footings which are not tied together. At the 5th floor, there is an abrupt change in floor plan resulting in stiffness discontinuity at this elevation. Since the basement floor is used for parking, all steel columns in the basement level are encased in thick concrete to prevent possible damage by accidental car impact.

4. ANALYTICAL MODELING AND FIELD TESTING OF THE BUILDINGS

4.1 3-D Finite Element Discretization of the Buildings

To ensure accurate prediction of buildings translational, torsional, and vertical responses, the analytical models must be capable of accurately simulating all structural aspects of the buildings in the plan directions as well as in the vertical direction. This raises the need for 3-dimensional analytical modeling. This is particularly necessary for the Portland building because of its unsymmetrical floor plan and set backs a several story levels. Both buildings were modeled by discretizing all structural components into finite elements, interconnected at corner or end nodes to form the three-dimensional geometries of the models. Time-history analyses of the models were performed using Nastran, which is a large-scale general purpose digital computer program. The process used in discretizing the buildings for finite element analyses is discussed below.

The 3-D models of the buildings consisted of a large number of finite elements representing structural components such as beams, columns, slabs and shear walls. Structural connections, such as welded and bolted joints or monolithic cast-in-place connections between beams, columns, and slabs were modeled as rigid connections. The number of elements used in the discretization process depended on the capabilities of the software for modeling and hardware for analysis. Because of changes in geometry and in structural properties of the buildings, each story of the buildings had to be modeled individually.

4.1.1 Modeling Beams and Columns

Structural beams and columns were modeled using 2-noded beam elements. Each end node of a beam element has 6 degrees of freedom, or 12 degrees of freedom per element. The required input for each element includes such physical and material properties as the cross sectional area, moments of inertia with respect to the strong and weak axes of the element, shear and elastic moduli, Poisson's ratio, and mass density. For steel beams or columns, except f^r built-up members where physical properties were hand-computed before input, the physical properties of commercially available sections were conveniently incorporated using special library which contains the properties of these sections. For concrete beams, joists, and columns, the physical properties were computed using the dimension obtained from the structural drawing. The Portland building is assumed to be new with no serious cracks in its structural components, thus only the masses of the reinforcements were considered. Contribution of the reinforcement to the structural stiffness of the entire building was ignored.

Within a story, each column was modeled using three line elements equal in length. Within a bay, each beam was modeled by at least two line elements. The geometry of each beam element was characterized by its cross sectional properties at two ends. As mentioned above, all beam-column connections were modeled as rigid connections.

4.1.2 Modeling Slabs and Shear Walls

Slabs, shear walls and foundation walls were modeled using three-noded triangular and four-noded quadrilateral elements. Rigid beam elements were used at the boundaries of the quadrilateral elements to simulate built-in foundation columns. The floor slabs, shear walls, and foundation walls were subdivided such that the connections between them and the beam and column elements were at their corner nodes.

4.1.3 Modeling Foundation

Since only low-level excitation was applied to the building on the top floor, soil-structure interaction was considered insignificant for the purpose of model verification. The surrounding soil around the foundation wall was therefore not modeled. All column, foundation wall and shear wall elements were fixed at the foundation level.

4.2 Field Testing

Vibration tests were conducted on both buildings in cooperation with USGS. The response data were recorded and analyzed by USGS to determine the in-situ dynamic properties of the buildings. Dynamic properties that were measured included response frequency and damping. The response data were obtained by deploying portable digital seismometers at judiciously selected locations in the buildings. The buildings were excited by human-induced motion. The advantage of using controlled human-induced motion, rather than mechanically induced motion is that the buildings can be tested conveniently without disrupting building occupants or damage to the facility. The disadvantage is that the forcing function of this kind of dynamic excitations can not be accurately defined. Although the motions induced by this technique were small, they were large enough to be clearly distinguished from motions induced by ambient conditions such as wind or moving traffic.

4.2.1 Test Setup and Excitation Technique

Three-component short period seismometers were used to measure velocity at various locations in the building. A total of 7 seismometers were used in testing the Portland building, and 10 seismometers were used in the Long Beach building. The difference in number of seismometers used was due to the availability of extra seismometers at the time the Long Beach building was tested. These seismometers were connected to individual Portable Data Acquisition Systems. Several tests were performed in each building. In each test, the seismometers were deployed at predetermined locations, usually near the slab-column joints. To excite the building, two persons apply synchronous impulses to columns located at the far ends of the building to induce the building into motion in the desired direction. For example, to induce torsional motion, one person would push the column in one direction while the other would push in the opposite direction. While this technique is simple and quite effective in determining the fundamental frequencies of building response, it poses difficulty in computer modeling since the dynamic forcing function needed for dynamic transient analyses cannot be accurately defined. The difficulty involves determining accurate duration of each impulse and the time interval between impulses.

4.2.2 Mode Shape and Frequency Calculation

Frequencies of vibration of the buildings in two orthogonal directions and in torsion were computed by performing spectral analyses of the velocity records. The velocity records were integrated to give displacement histories of instrumented nodes. The displacement histories were used to plot the mode shapes at different time intervals. For the Portland building, data obtained at different elevations (from basement level to floor 7) of column Q7 were used in calculating mode shapes. For the Long Beach building, mode shapes were plotted using data at different elevations of column D9 (from basement level to floor 8).

Typical three-component velocity records, obtained at column L23, on floor 7 of the Portland building are shown in Figures 4.1 (a), (b), and (c). The dashed curves that overlay the data identify "windows", i.e. the portions of the records that were used in spectral analyses. Three components of the displacement histories of the same node, integrated from the velocity records are shown in Figures 4.2 (a), (b), and (c). Figures 4.3 (a) and (b) show spectra for the N-S and E-W direction, respectively, of the Portland building. The building mode shapes are shown in section 4.3.2.

4.3 Verification of Mathematical Models

As discussed in section 4.2, the buildings were excited by human-induced impulses applied at pre-determined columns on the top floor of each building. For dynamic analyses, the forcing function was modeled as impulse loads (see Figure 4.4). The duration and the time interval of impulses were estimated since they were not precisely measured.

To verify the mathematical models, dynamic analyses were performed on models of both buildings using impulse forcing function which emulates human-induced impulse loading. The calculated responses were then compared with the measured responses. In the analyses, the damping values obtained from the actual tests were used. For the Portland building, masses due to exterior and interior non-structural elements were added to the structural mass in the analysis. Since the Long Beach building was not furnished at the time of testing, no add d mass was used in the analysis of this building.

The analytical model was verified by comparing calculated and measured values of mode shapes and corresponding frequencies. Frequencies of the buildings in the North-South, East-West, and in torsional directions are obtained by performing fast fourier transform of the nodal displacement histories. The calculated response frequencies and the test response frequencies of both buildings are listed in Tables 4.1 and 4.2.

For the Portland building, the calculated and measured deflected shapes, plotted using normalized displacements obtained at different elevations of column Q7, are shown in Figures 4.5 and 4.6. Spectral densities of four nodes of the Portland building, corresponding to locations of columns S4, S7, L20, and L23 on the seventh floor, are shown in Figures 4.7 (a) to (l). For this building, a structural damping ratio of 0.95% was used in all directions for the analyses, since the measured damping ratios are very close together, being 0.93%, 0.95%, and 0.95% in the North-South, East-West, and torsional directions. The calculated translational response frequencies in the North-South direction of the four indicated nodes, obtained from fast fourier transform of the time-histories of their translational displacements are 1.09 Hertz, 1.10 Hertz, 1.01 Hertz, and 1.01 Hertz, respectively. The calculated translational response frequencies in the East-West direction are 1.18 Hertz, 1.17 Hertz, 1.30 Hertz, and 1.33 Hertz, respectively. The calculated torsional response frequencies are 2.11 Hertz for column S4, 1.74 Hertz for column S7, 1.50 Hertz for column L20, and 1.46 Hertz for column L23.

For the Long Beach building, the calculated and measured deflected shapes, plotted using normalized displacements obtained at different elevations of column D9, are shown in Figures 4.8 and 4.9. The measured damping ratios for the North-South and the East-West directions are 1.27% and 0.98%, respectively. The spectral densities, obtained from fast fourier transform of the translational and torsional displacement histories of four nodes at grids C2, C5, C8, and C11 on the eigth floor are shown in Figures 4.10 (a) to (l). The translational responses of this building appear to be very uniform in both horizontal directions. In the North-South direction, the response frequencies due to model impulses applied in that direction range from 0.72 Hertz to 0.73 Hertz. In the East-West direction, the response frequencies range from 0.845 Hertz to 0.860 Hertz. The torsional response frequencies, obtained from the fast fourier transform of the rotational displacement histories with respect to the vertical axis, are slightly different for the two ends of the building. In the west end where grids C2 and C5 are located, the computed torsional response frequencies are 1.090 Hertz and 0.92 Hertz, respectively. In the east end where grids C8 and C11 are located, the computed torsional response frequencies are 0.92 Hertz and 0.86 Hertz, respectively.

The differences between the measured and calculated response frequencies of the Portland building are approximately 3.8% to 4.8% in the N-S direction, 6.4% in the E-W direction, and 6.2% to 32% in torsion. For the Long Beach building, they are 3.3% to 4.3% in the N-S direction, 3.0% to 4.9% in the E-W direction, and 3.1% to 14.7% in torsion. Better agreement between the measured and calculated frequencies in the Long Beach building is thought to be due to the symmetry of the building.

5. BUILDINGS RESPONSES TO KNOWN EARTHQUAKE RECORDS

5.1 Introduction

Dynamic analyses were performed on the finite-element models of both buildings with acceleration records obtained from past earthquakes. The purpose was to examine building response to actual recorded earthquake ground motion. Since general dynamic response characteristics of the buildings are only of interest, influence of such factors as site amplification, distance from the buildings to the free-field stations where the acceleration records were taken, and soil-structure interaction were not considered. Even though analytical response was imprecise due to the reasons stated above, the buildings response characteristics obtained from the analyses provided helpful insight for the development of logical seismic instrumentation schemes for the buildings being studied. The model of the Portland building was excited by three components of the acceleration records from the 1962 Portland earthquake, and the model of the Long Beach building was subjected to acceleration records from the 1987 Whittier Narrows earthquake.

5.2 Response of the Portland Building to the 1962-Portland Earthquake

The three components of ground acceleration which were used as input excitation for the Portland building are shown in Figures 5.1a, b, and c. These acceleration records were obtained during the November 1962 Portland earthquake at a station located at 45.52° latitude and 122.68° longitude (station 2110P). The maximum peak ground acceleration recorded was approximately 0.12g, in the north-south direction. The building response to this earthquake is shown by the particle motions of two nodes located at columns Q4 and L23, see Figure 5.2). These particle motions were superimposed to show the relative motion of the opposite ends of floor 7. Figures 5.3, 5.4, and 5.5 show the particle motions in three planes, horizontal (x-y plane), vertical and building long direction (x-z plane), and vertical and building short direction (y-z plane), respectively. From the particle motion plots, it can be seen that the east and west ends of the building responded very differently. The motion (see Figure 5.3) of the east end, where column L23 is located, is very directional. In the horizontal plane, this motion is predominantly in the north, north-west direction. While the motion of the west end, where column Q4 is located, is strongly influenced by circular motion. This difference in responses of the east and west ends clearly suggests that the response of this building is dominated by torsional and rocking motions. This probably may be attributed to the irregular shape of the building and the locations of the shear walls relative to the two nodes. These observations also indicate that the floors of the Portland building are flexible and the motion of any one floor in this building cannot be sufficiently characterized by only 6 degrees of freedom.

5.3 Response of the Long Beach Building to the 1987-Whittier Earthquake

The north-south, east-west, and vertical components of ground acceleration for the 1987 Whittier Narrows earthquake are shown in Figures 5.6a, b, and c. The station at which these records were obtained is located at 12400 Imperial Highway in Norwalk, California (station NORWA, 33.92° latitude, 118.07° longitude). Maximum peak acceleration was approximately 0.24g, in the N-S direction. Similar to the Portland building, the building response to this earthquake excitation was illustrated by superimposing the particle motions of two nodes located at columns C2 and C11 on floor 8. These two nodes are symmetrical with respect to the short direction (North-South) of the building (see Figure 5.7). Figures 5.8, 5.9, and 5.10, respectively, show the particle motions of the two selected nodes in the horizontal plane (x-y plane), in the vertical and building long direction plane (x-z plane), and in the vertical and building short direction plane (y-z plane). From these particle motion plots, it can be seen that the two ends of the Long Beach building experienced similar motions in all three planes. This suggests that this building is predominantly controlled by translational displacements. Since the selected nodes are located at opposite ends along the building length, it also indicates that the floors of this building behaves like a rigid body, i.e. the motions of an entire floor of this building can be characterized by the 6 degrees of freedom of any node on that floor.

6. GUIDELINES FOR SEISMIC INSTRUMENTATION

6.1 General Recommendations

Since buildings with different aspect ratios, geometries, and/or floor rigidity behave differently under seismic excitations, it is difficult to develop a detailed, rigid guideline for seismic instrumentation that is suitable for all the particular designs of existing buildings. General instrumentation schemes for monitoring in-plane and 3-dimensional motions of buildings, with some variation to accomodate structures with different floor rigidity, has been recommended in [1]. These recommendations are adopted in this section and developed into general guidelines for seismic instrumentation of existing buildings. These guidelines will be used along with the observed building behavior under earthquake excitations, described in previous section, to develop seismic instrumentation schemes for the two buildings investigated here.

Two sets of guidelines are recommended in the following sections. One set for structures with rigid floors, where each floor can be considered as a rigid body and thus its motion in the 3-D space can be characterized by 3 translational and 3 rotational degrees of freedom (x, y, z, theta-x, theta-y, and theta-z). And one set for structures with flexible floors, where different segments of a floor might behave differently and thus the motion of a floor in the 3-D space cannot be sufficiently defined using only 6 degrees of freedom. It is beyond the scope of this study to suggest which types of design will result in either rigid or flexible floor. However, as discussed in previous section, the responses of the two buildings being studied indicate that the Long Beach building may be treated as a building with rigid floors, and the Portland building may be treated as a building with flexible floors.

6.1.1 Structures with Rigid Floors

As discussed above, the horizontal motion of each rigid floor can be characterized by 2 in-plane translational degrees of freedom (x and y) and 1 rotational degree of freedom with respect to the vertical axis (theta-z). For vertical motion, 3 additional degrees of freedom are needed to define the position of the floor plane in the 3-D space (z, theta-x, and theta-y). Thus at least 3 uniaxial instruments are needed to characterize horizontal motion of a rigid floor, and at least 6 uniaxial instruments are needed to characterize both vertical and horizontal motions of a rigid floor. Based on that observation, the following instrumentation deployment schemes are recommended for buildings with rigid floors:

6.1.1.A Guidelines for Vertical Deployment

Seismic monitoring instruments are recommended as follows (schematic describing these recommendations is shown in Figure 6.1):

1. At the building foundation level, a minimum of one set of 6 uniaxial accelerometers or equivalents (e.g. 1 triaxial and 3 uniaxial accelerometers) should be used to fully characterize building base excitations. Within the foundation level, these instruments should be placed at locations recommended in section 6.1.1.b.

2. On the top floor or at highest permissible elevation where the instruments can be properly maintained, a minimum of one set of 6 uniaxial accelerometers or equivalents should be deployed since there will always be non-zero displacements at this elevation, regardless of what mode the building is responding. Within floor arrangement should also follow recommendations in section 6.1.1.b.

3. At elevation where there are significant or abrupt changes in structural stiffness, structural mass, or structural geometry, such as termination of perimeter columns, changes in floor plan geometry , etc., since these discontinuities represent structural nonlinearity and may significantly alter the structural responses of the building. A complete set of 6 uniaxial instruments is desirable at these locations, but all 6 are not required since two complete sets are required at the foundation and the top floor. These instruments should be placed at locations and in directions which, along with data otained from the foundation and the top floor (recommendations 1 and 2), would facilitate mode shape calculation with at least 3 data points.

4. At elevations where maximum displacements associated with individual mode shape, are anticipated. Similar to recommendation 3, a complete set of 6 uniaxial instruments is desirable but not required at these elevations.

5. Where possible, free-field instrumentation should be deployed to allow an assessment of the complex soil-structure interaction. Actual building base excitation can be assessed by differentiating the free-field motion and the motion recorded at the foundation level of the building.

6.1.1.B Guidelines for Horizontal (within Floor) Deployment

1. Within one floor, the instruments should be placed close to the perimeters of the building. Since displacements at locations closer to the building perimeter are usually larger than those close to the building center.

2. Out of one set of six (6) uniaxial instruments recommended for each rigid floor, three (3) should be placed horizontally to monitor horizontal motion, and three (3) vertically to monitor vertical and rocking motion, of the floor. Two (2) of the three horizontal instruments should be placed in the same horizontal direction which may have larger translational displacement or smaller stiffness (generally in the direction perpendicular to the length of the building), but the straight line between these two instruments should not be parallel to that horizontal direction in order to allow the calculation of rotational displacement in the horizontal plane. The remaining horizontal instrument should be placed at the same location with one of the first two, but in the orthogonal direction to allow the calculation of the horizontal translational displacement in that orthogonal direction and another calculation of rotational displacement in the horizontal plane.

3. The remaining three instruments of the set should be placed in the vertical direction to monitor vertical and rocking motion. Two (2) of these vertical instruments should be placed at the same locations where the horizontal instruments are placed, and the remaining vertical instrument at a location which forms a right angle with the other two locations. This allows the calculation of vertical displacement and rotational displacements with respect to both horizontal axes, thus defines the rocking motion of the building. A schematic explaining these recommendations is shown in Figure 6.2.

6.1.2 Structures with Flexible Floors

Buildings which have large floor length-to-width ratios may be considered as having flexible floors. For this type of buildings, more instruments are needed on each floor to characterize the horizontal and vertical motions of the floors.

6.1.2.A Guidelines for Vertical Deployment

Recommendations for vertical deployment of instruments for buildings with flexible floor are the same as for buildings with rigid floors (section 6.1.1.A), since the vertical deployment of instruments depends on other factors such as the building height-to-base width ratio, the continuity of stiffness and mass distributions, etc., rather than the floor rigidity.

6.1.2.B Guidelines for Horizontal Deployment

A flexible floor may be subdivided into smaller finite segments that are rigid. Within each segment, a complete set of 6 uniaxial instruments, placed following recommendations for a rigid floor, is needed. However, the number of instruments used may be reduced by placing some instruments on common boundary of two rigid segments. There is no specific rules pertaining to the division of flexible floor into finite segments. The practitioner will make his or her own judgement regarding this division.

6.2 Recommended Instrumentation Schemes for the Selected Buildings

Based on the recommended guidelines and the observed behavior in the two buildings, the following instrumentation schemes are suggested for future seismic monitoring of the Portland and the Long Beach buildings. It should be noted that these schemes are developed from the structural performance viewpoint only. The probability of the buildings experiencing future strong ground motion is not considered in developing these seismic instrumentation schemes.

6.2.1 The Portland Building

The instrumentation scheme for the Portland building is shown in Figures 6.3 and 6.4. Since the floor of this building is considered flexible, each floor is arbitrarily divided into two segments, with line 16 in the North-South direction as common boundary between the two segments. Three sets of instruments are recommended for the foundation level (B1), floor 3, and the Penthouse level. Within each floor, a total of 8 uniaxial accelerometers are recommended. Of these, 4 accelerometers are placed on the common boundary line, 3 at grid Q16 and 1 at grid H16. The remaining 4 accelerometers on each floor are placed in the East and West sides of the common boundary, 2 at grid Q4 and 2 at grid H23. This arrangement allows all 6 degrees of freedom needed to define the motion of each of the two rigid segments to be measured.

6.2.2 The Long Beach Building

A total of three sets of instruments, each consisting of 6 uniaxial accelerometers or equivalents, and one triaxial accelerometer for free-field instrumentation are recommended for the Long Beach building (21 uniaxial accelerometers in all). Of the three sets, one set is to be placed at the building foundation level. The second set is to be placed on floor 4, where the floor configuration changes. And the third set is to be placed on floor 8. The locations and direction of these instruments are identical within the floors. Figures 6.5 and 6.6 show the locations of the instruments. This instrumentation scheme allows two mode shape calculations in the North-South direction, one for the East end of the building at column D10, and one for the West end at column D3. One mode shape calculation in the East-West direction can also be obtained at column D3. Torsional and rocking motion of the foundation, floor 4, and floor 8 can also be measured.

7. SUMMARY

Two buildings, the Long Beach Federal Building and the Portland Federal Building East, were identified by GSA for study with the aim to develop guidelines for seismic instrumentation in existing buildings. Low-level vibration tests were performed for both buildings to determined their dynamic properties. The building dynamic properties were then incorporated into 3-D finite element models of the buildings and time-history analyses were performed with the controlled human-induced excitations as impulse excitations (step function). The measured and calculated response frequencies and mode shapes were compared. For the Portland building, the largest difference between the measured and calculated frequencies was 32%, in torsion. The smallest difference was 3.8%, in the N-S direction. For the Long Beach building, the largest difference was 14.7% in torsion, and the smallest was 3.0% in the E-W direction. The models were then subjected to the acceleration records obtained from past earthquakes to reveal buildings response under these realistic earthquake excitations. The Portland building was analyzed using the acceleration records from the November 1962-Portland earthquake. The results show that the east and west ends of the building behave very differently under this kind of excitation, and that the building response is influenced by torsional and rocking motions. The Long Beach building was analyzed with acceleration records obtained during the 1987-Whittier Narrows earthquake. Similar motions were observed for the east and west ends of the building which suggests that the response of this building was mainly translational. Based on the analytical results, the Portland building was considered as having flexible floors, while the long Beach building was considered as having rigid floors. From these observations, two seismic instrumentation schemes were recommended as follows:

1. For the Portland building, a total of 24 uniaxial accelerometers are recommended to be deployed within the building at three elevations, the foundation floor, floor 3, and the penthouse floor. Within each instrumented floor, 8 uniaxial accelerometers are to be arranged as follows: 4 accelerometers on line 16 (3 at grid Q16 and 1 at grid H16, see Figure 6.4), 2 accelerometers in the west end at grid Q4, and 2 accelerometers in the east end at grid H23. In addition, three free-field accelerometers are recommended if field condition permits.

2. For the Long Beach building, a total of 18 uniaxial accelerometers are recommended at three elevations, the foundation floor, floor 4, and floor 8 (6 uniaxial accelerometers per floor). Within each floor, 3 accelerometers are to be placed at grid D3, 2 accelerometers at grid D10, and 1 at grid A3 (see Figure 6.6). Three optional free-field accelerometers are also recommended if field condition permits.

This study provided GSA with guidelines for use in developing appropriate instrumentation scheme, including the procedure to determine the required number and locations of the instruments, for existing buildings. Specific instrumentation schemes were also recommended for the two buildings analyzed in this study. Since the Long Beach building is more likely to experience strong motion, it is recommended that this building be instrumented for future study.

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PORTLAND BUILDING

DIRECTION	MEASURED DAMPING RATIO (%)	MEASURED RESPONSE FREQUENCIES (HZ)	CALCULATED RESPONSE FREQUENCIES (HZ)
N - S	0.93	1.05	1.09 (7S4) 1.10 (7S7) 1.01 (7L20) 1.01 (7L23)
E - W	0.95	1.25	1.18 (7S4) 1.17 (7S7) 1.30 (7L20) 1.33 (7L23)
TORSION	0.95	1.60	2.11 (7S4) 1.74 (7S7) 1.50 (7L20) 1.46 (7L23)

Table 4.1 Measured and Calculated Frequencies of the Portland Building

LONG BEACH BUILDING

DIRECTION	MEASURED DAMPING RATIO (%)	MEASURED RESPONSE FREQUENCIES (HZ)	CALCULATED RESPONSE FREQUENCIES (HZ)
N - S	1.27	0.70	0.728 (8C2) 0.730 (8C5) 0.726 (8C8) 0.723 (8C11)
E - W	0.98	0.82	0.845 (8C2) 0.860 (8C5) 0.859 (8C8) 0.859 (8C11)
TORSION	———	0.96	1.090 (8C2) 0.918 (8C5) 0.920 (8C8) 0.889 (8C11)

Table 4.2 Measured and Calculated Frequencies of the Long Beach Building

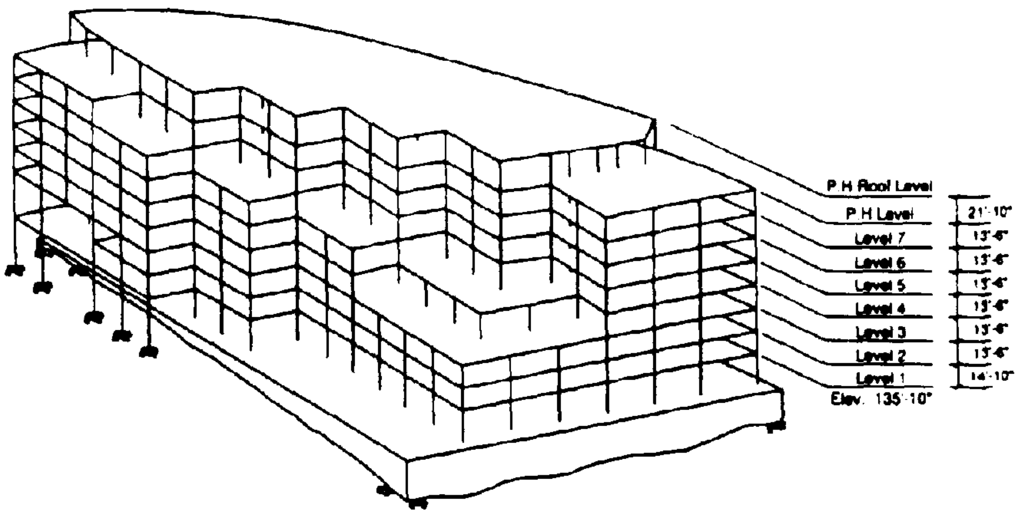


Figure 3.1 Structural View of the Federal Building East (Portland Building)

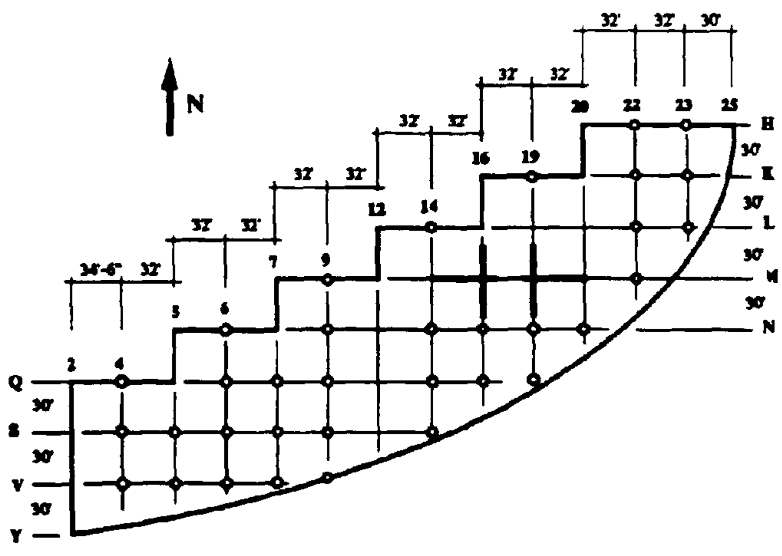


Figure 3.2 Plan View of a Typical Floor of the Portland Building

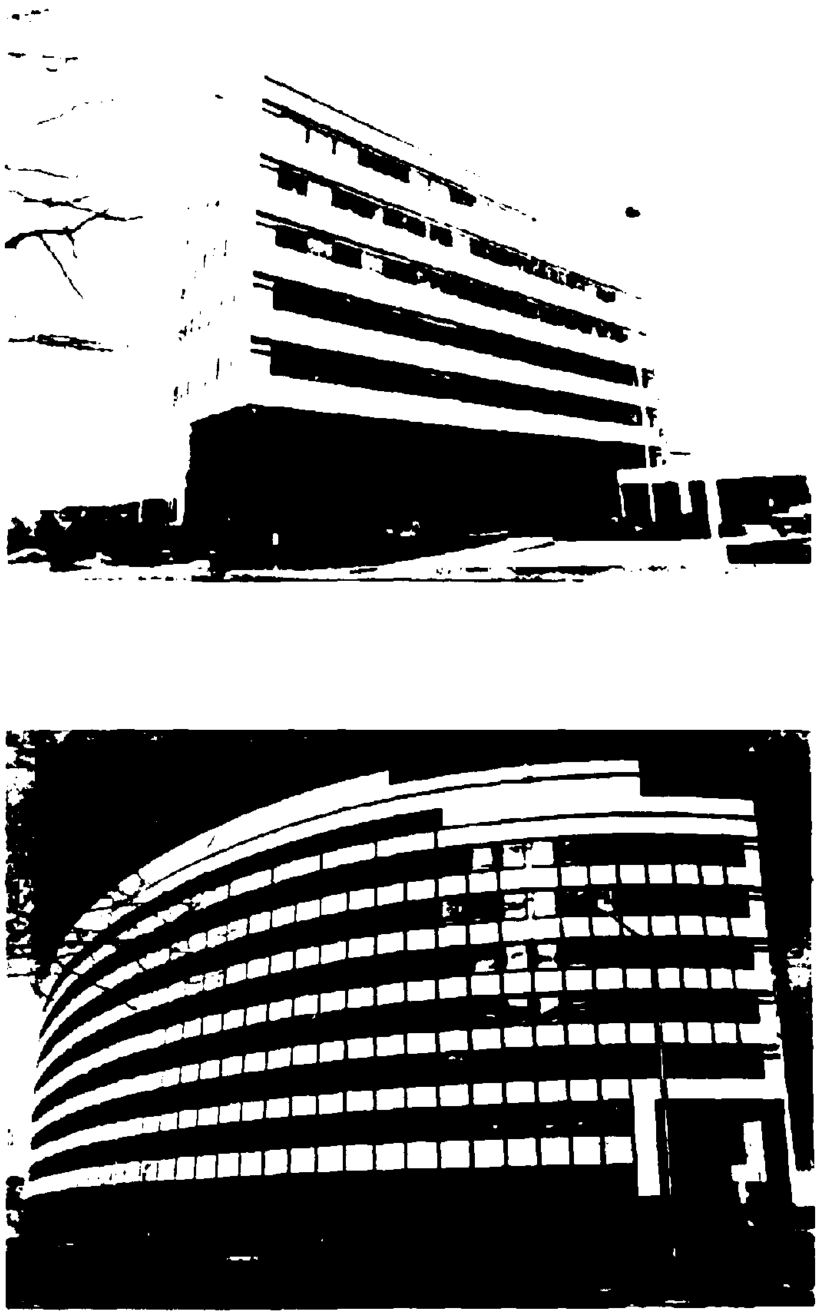


Figure 3.3 Exterior Views of the Portland Building
(a) North Facing
(b) South Facing

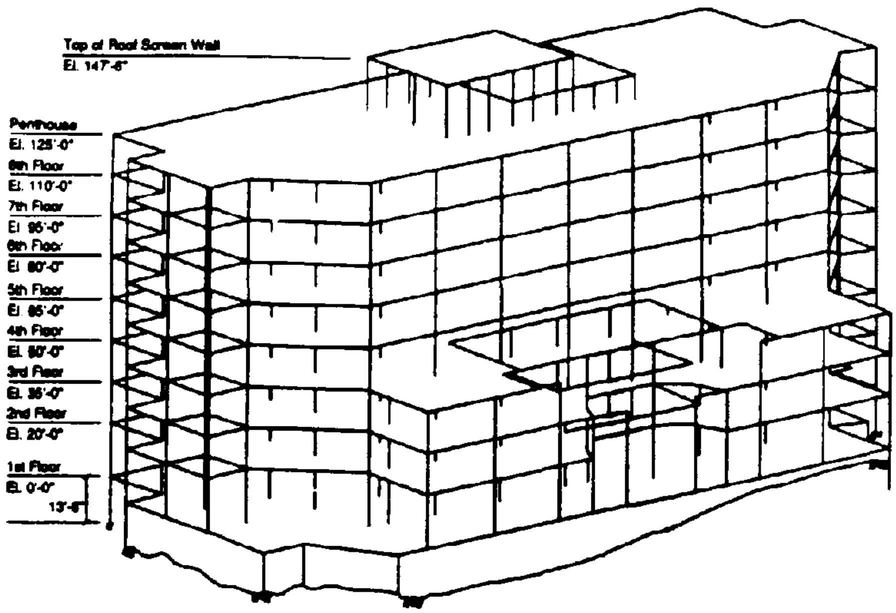


Figure 3.4 Structural View of the Federal Building (Long Beach Building)

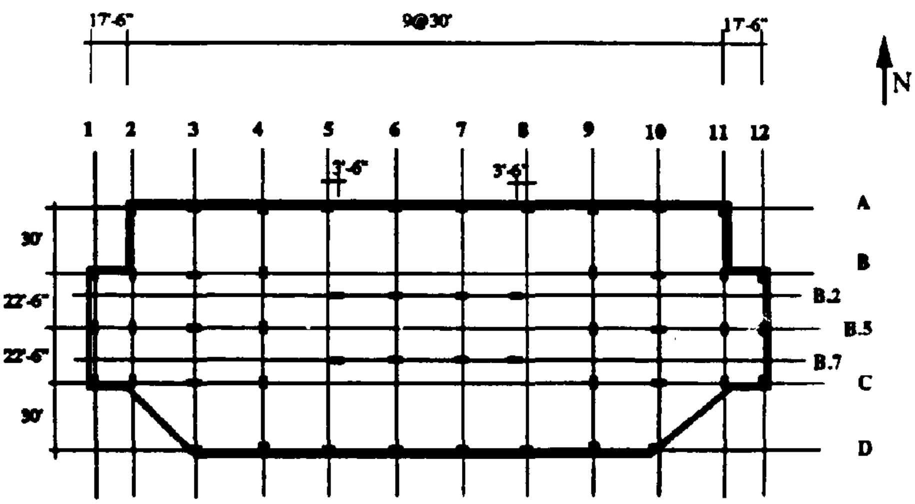


Figure 3.5 Plan View of a Typical Floor of the Long Beach Building

Figure 3.6 South Facing View of the Long Beach Building

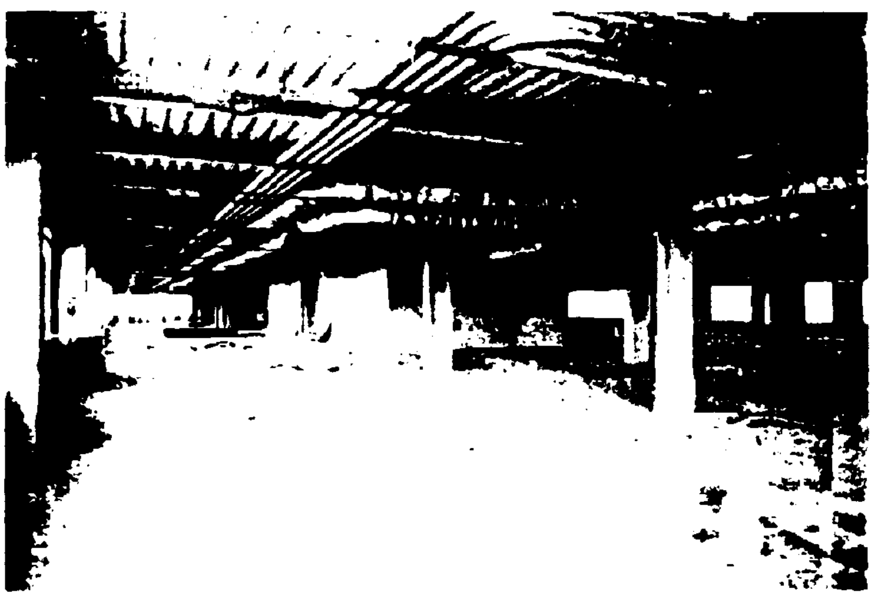


Figure 3.7 Interior View of Long Beach Building's top Floor at Testing

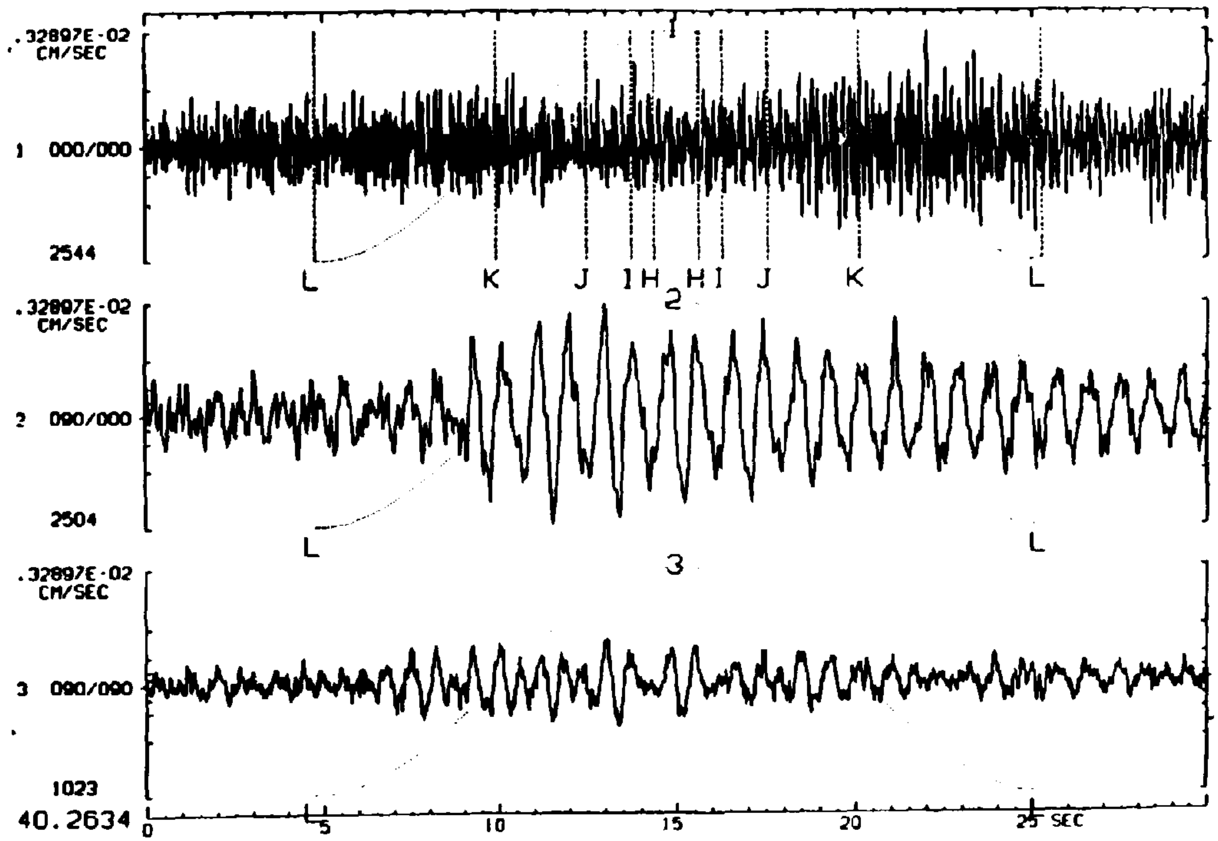


Figure 4.1 Test Velocity Records of Column L23, Portland Building
(a) N-S Component
(b) E-W Component
(c) Vertical Component

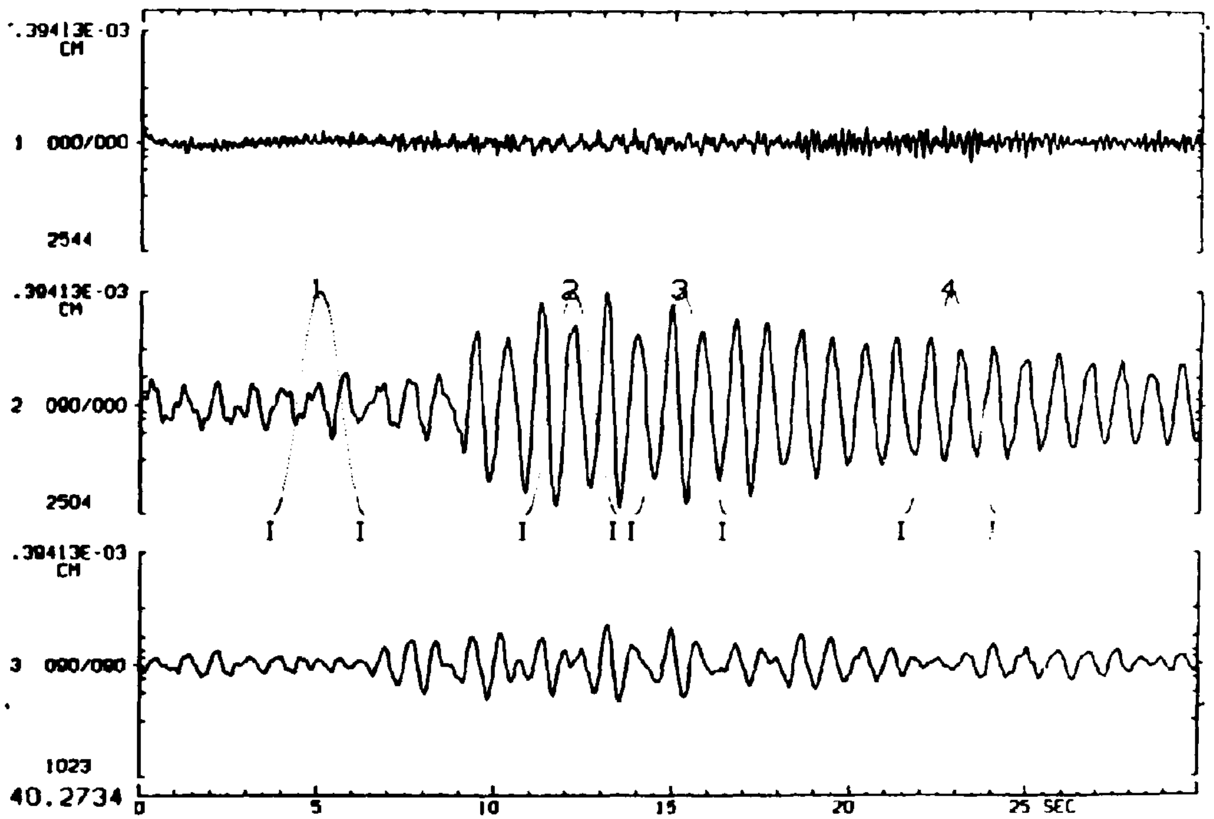
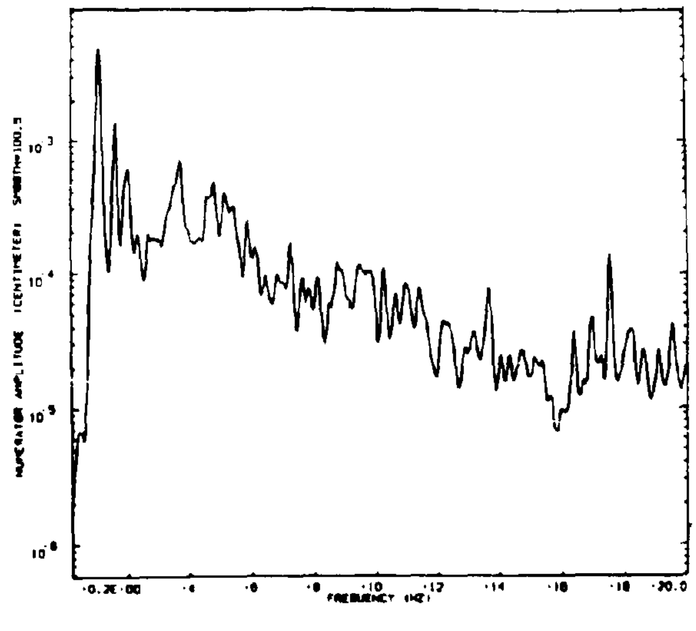


Figure 4.2 Test Time-History Displacements at Column L23, Portland Building, Obtained by Integrating Velocity Records
(a) N-S Displacement
(b) E-W Displacement
(c) Vertical Displacement



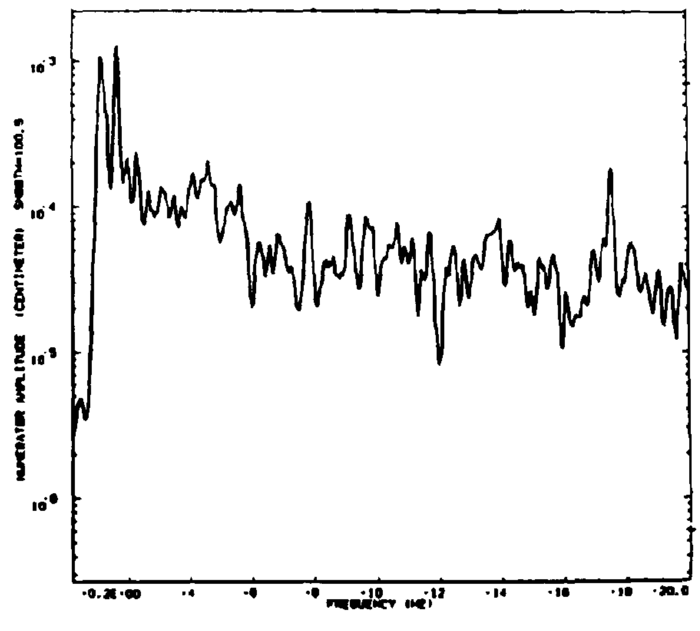


Figure 4.3 Spectra from Test Velocity Records at Columns L23, Portland Building
(a) For the N-S Direction
(b) For the E-W Direction

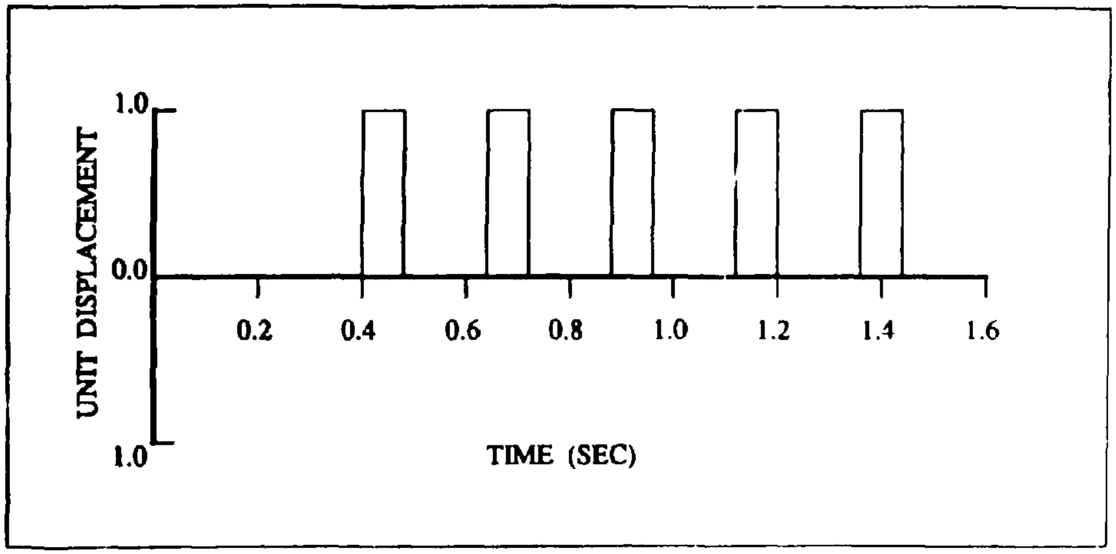


Figure 4.4 Model of Test Excitation

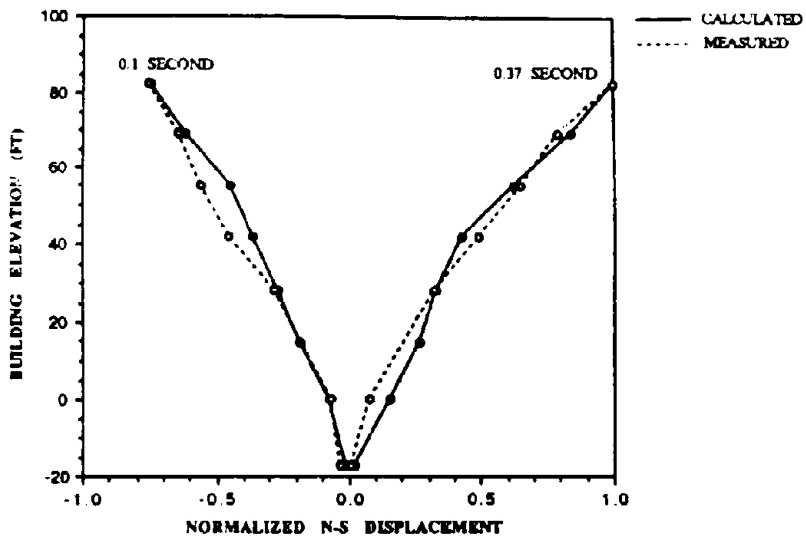


Figure 4.5 Measured and Calculated Mode Shapes in N-S Direction of the Portland Building

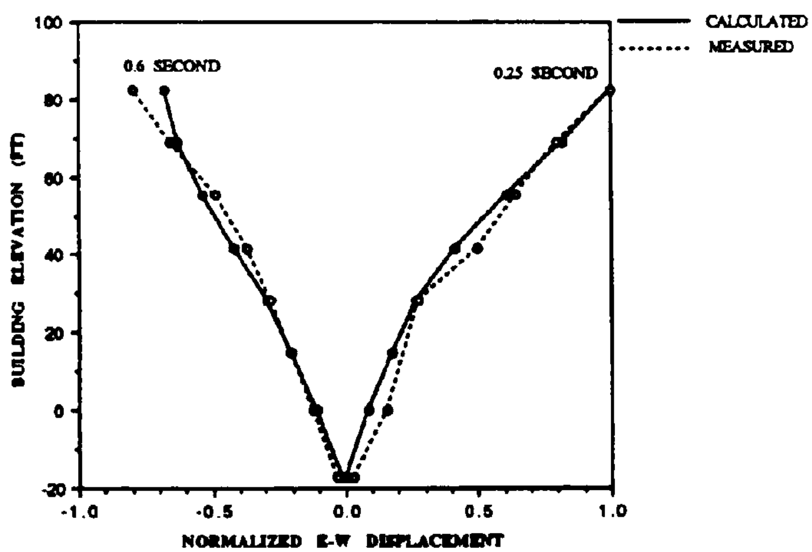


Figure 4.6 Measured and Calculated Mode Shapes in E-W Direction of the Portland Building

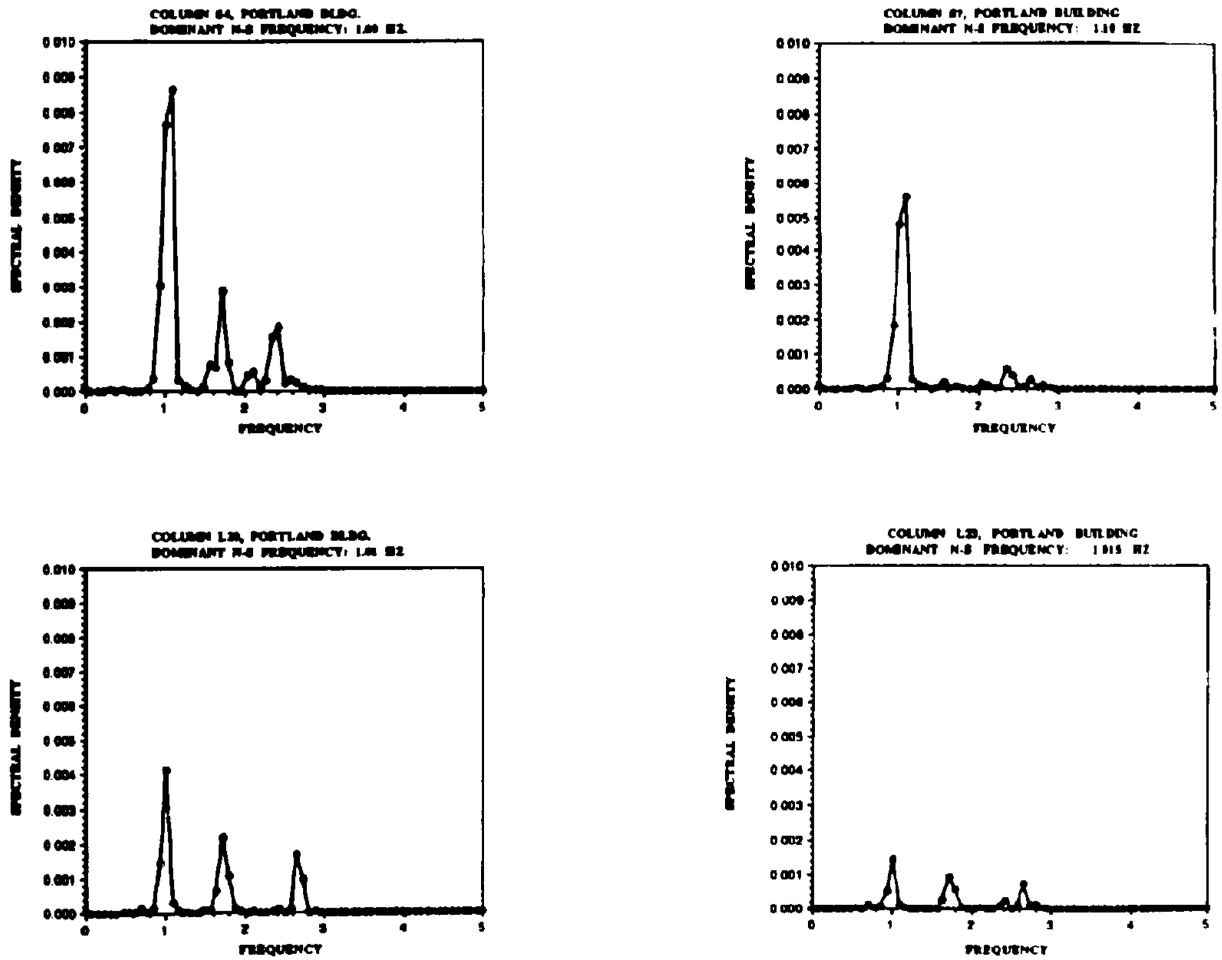


Figure 4.7 Spectra From Analytical Time-History Displacements of Columns on Floor 7, Portland Building
(a) Column S4, N-S Direction
(b) Column S7, N-S Direction
(c) Column L20, N-S Direction
(d) Column L23, N-S Direction

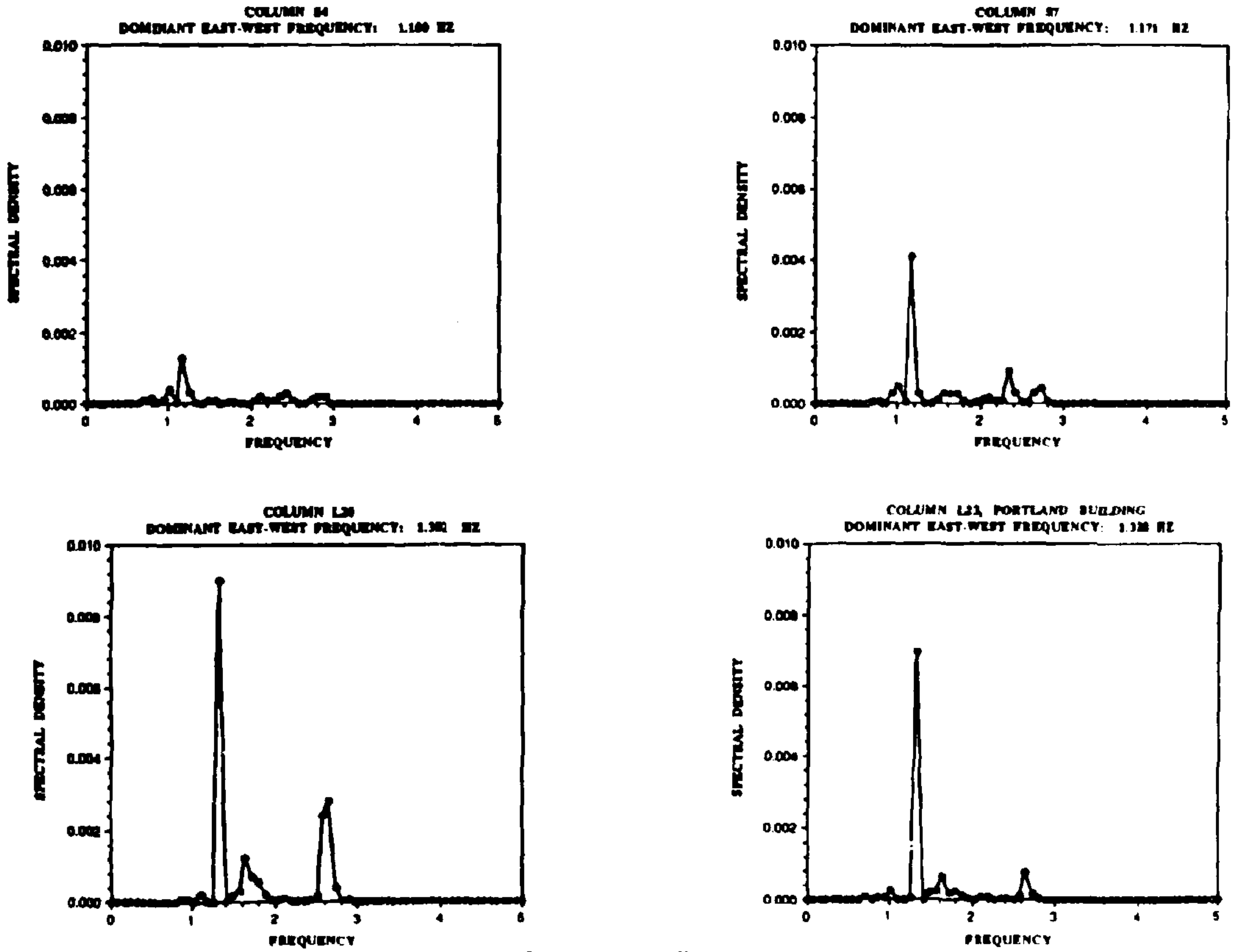


Figure 4.7 (continued)
(e) Column S4, E-W Direction
(f) Column S7, E-W Direction
(g) Column L20, E-W Direction
(h) Column L23, E-W Direction

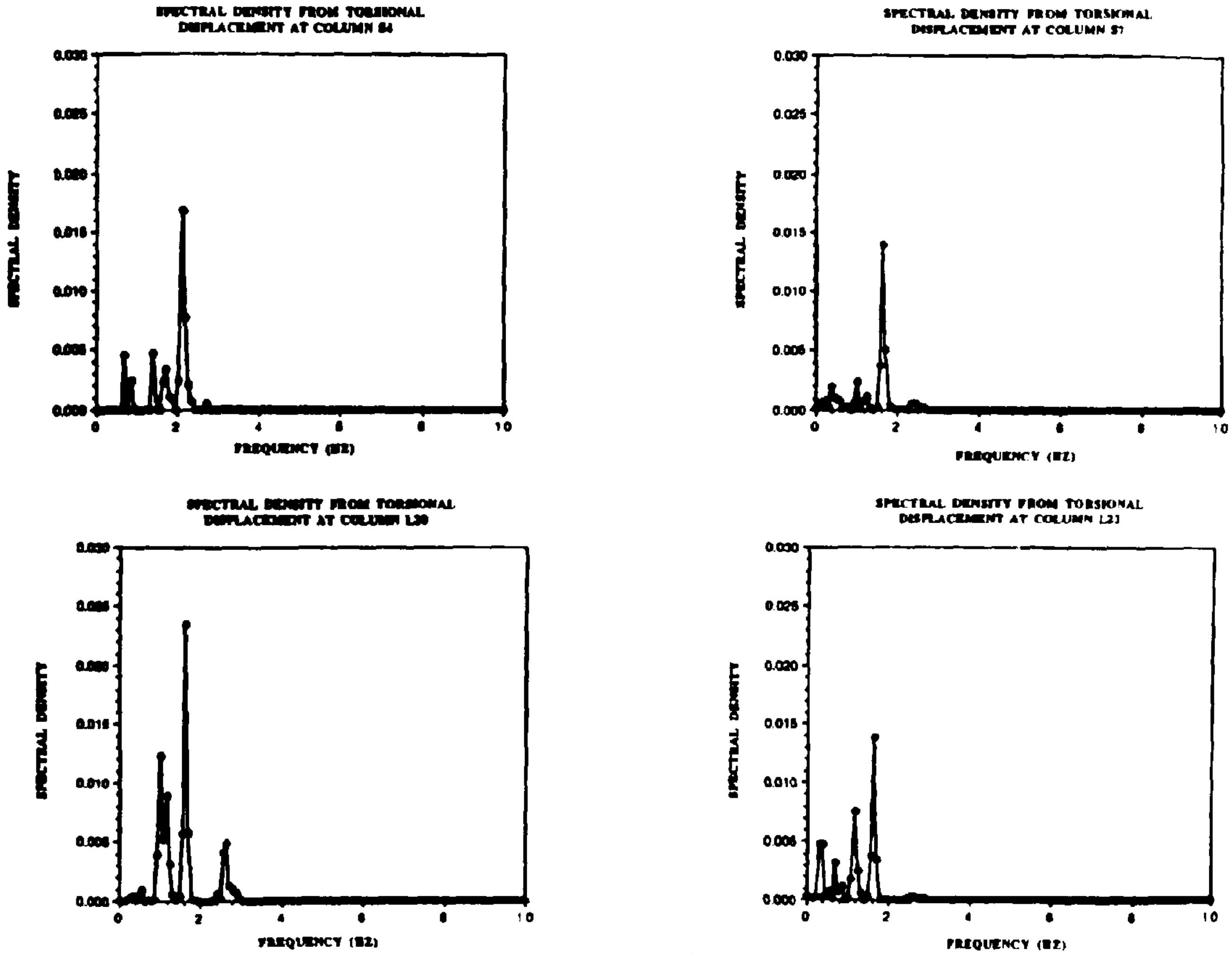


Figure 4.7 (continued)
(i) Column S4, Torsional
(j) Column S7, Torsional
(k) Column L20, Torsional
(l) Column L23, Torsional

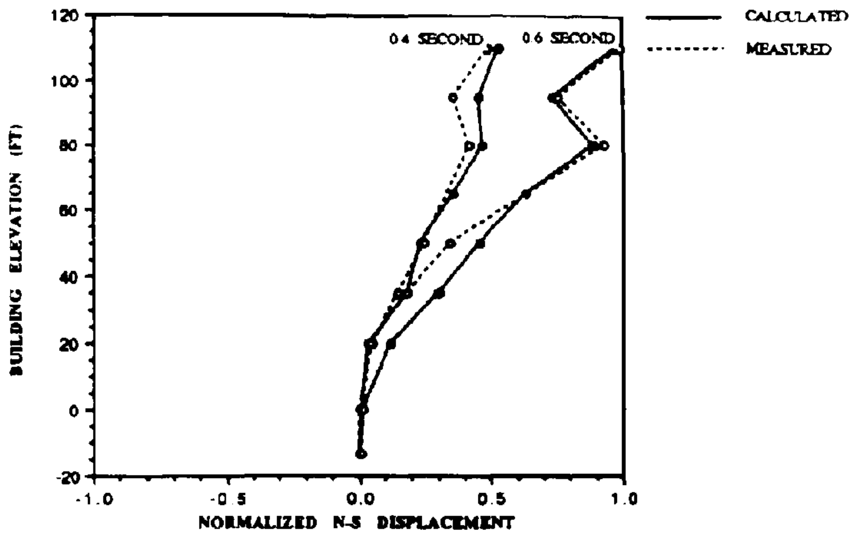


Figure 4.8 Measured and Calculated Mode Shapes in N-S Direction of the Long Beach Building

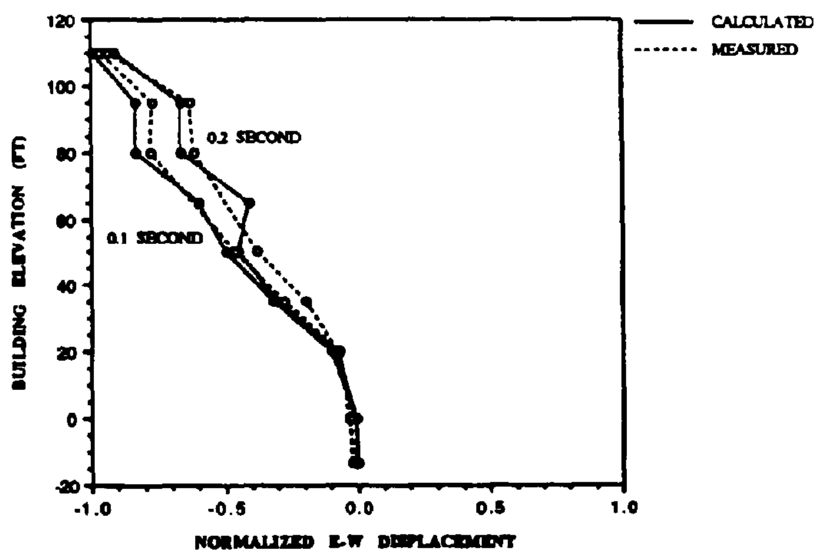


Figure 4.9 Measured and Calculated Mode Shapes in E-W Direction of the Long Beach Building

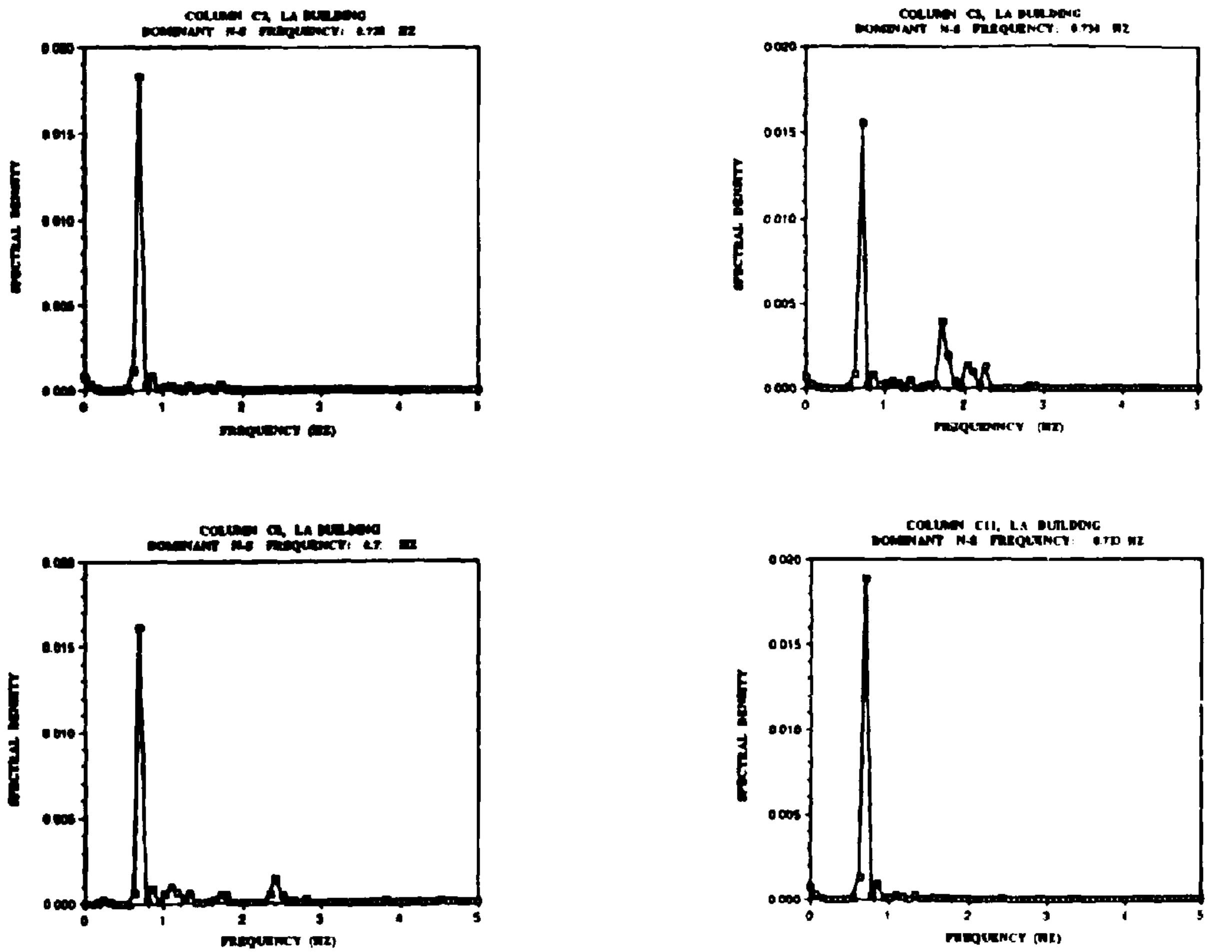


Figure 4.10 Spectra From Analytical Time-History Displacements of Columns on Floor 8, Long Beach Building
(a) Column C2, N-S Direction
(b) Column C5, N-S Direction
(c) Column C8, N-S Direction
(d) Column C11, N-S Direction

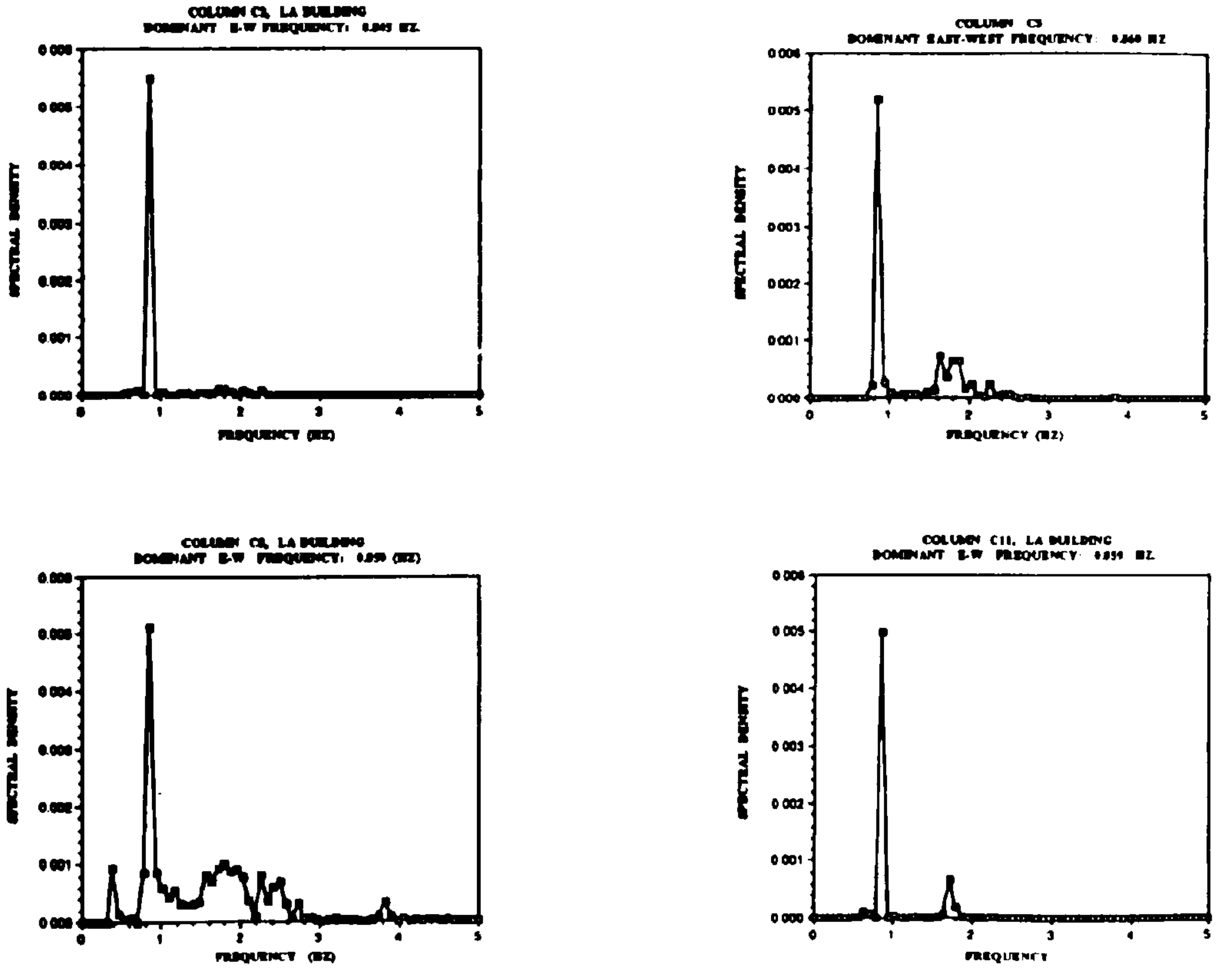


Figure 4.10 (continued)
(e) Column C2, E-W Direction
(f) Column C5, E-W Direction
(g) Column C8, E-W Direction
(h) Column C11, E-W Direction

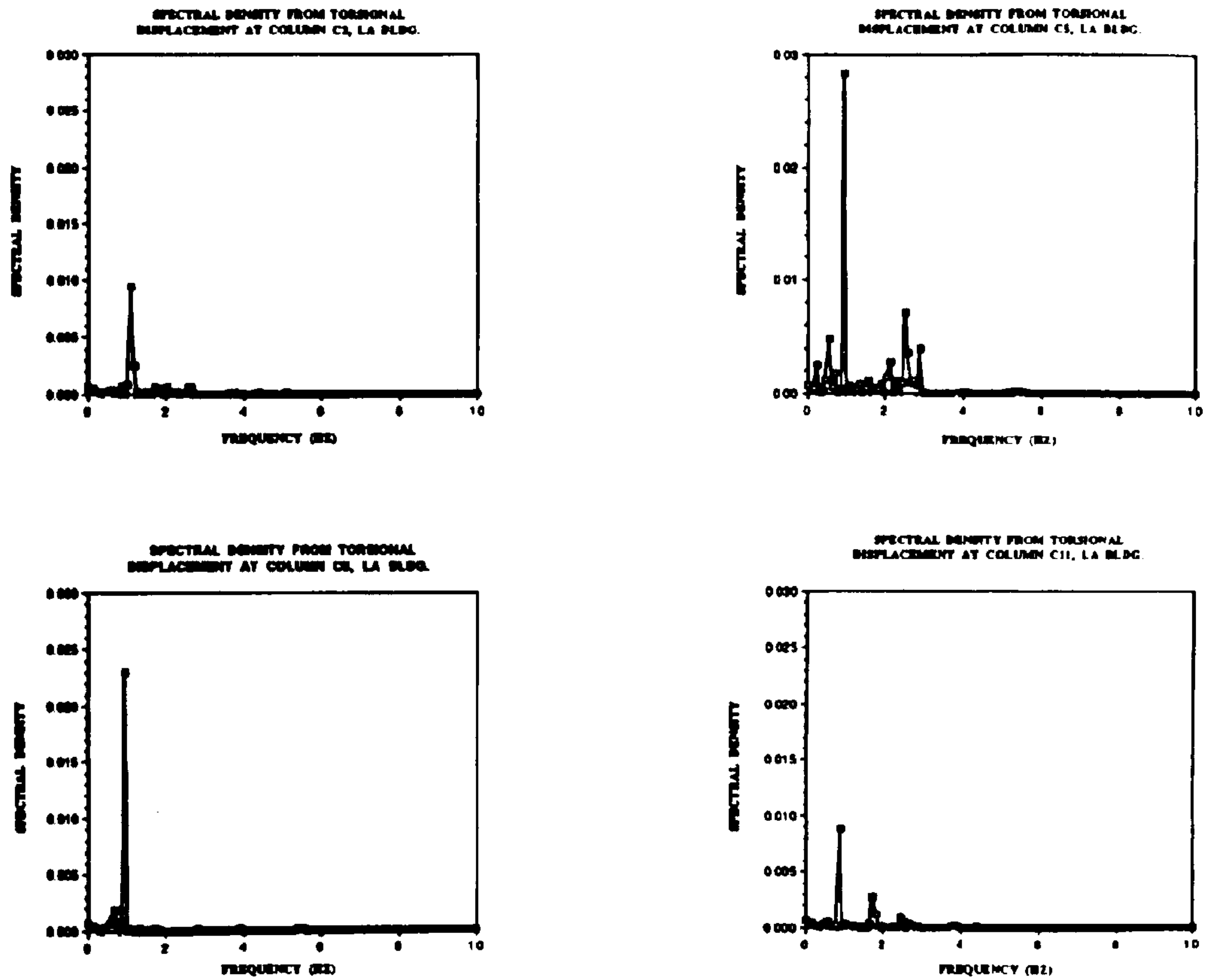
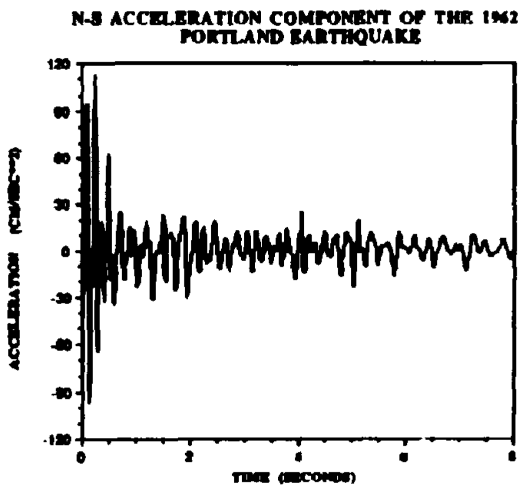
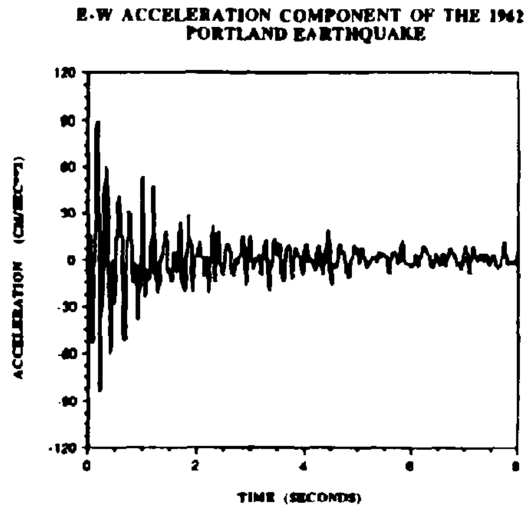


Figure 4.10 (continued)
(i) Column C2, Torsional
(j) Column C5, Torsional
(k) Column C8, Torsional
(l) Column C11, Torsional





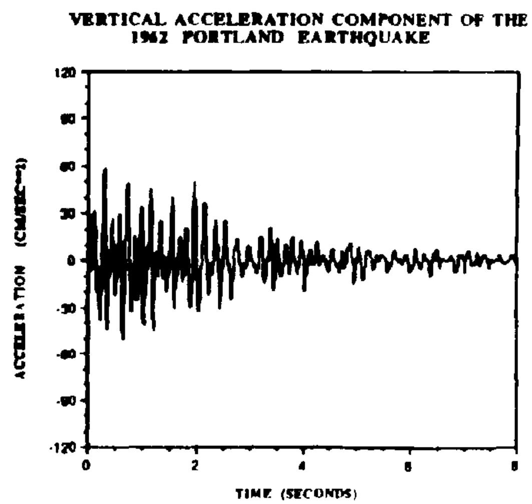


Figure 5.1 Ground Acceleration Records Due to the November 1962-Portland Earthquake
(a) N-S Component
(b) E-W Component
(c) Vertical Component

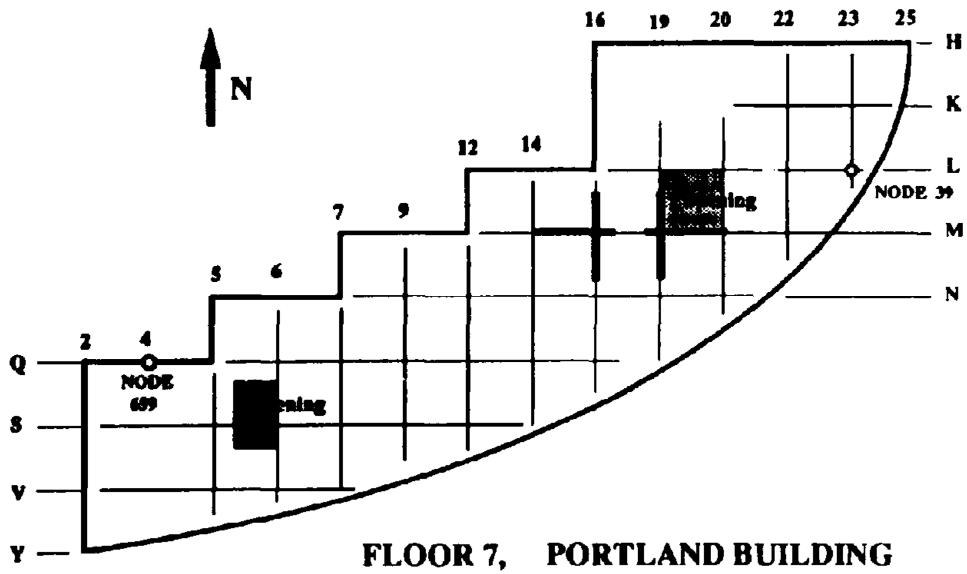


Figure 5.2 Locations of Interested Nodes in the Portland Building

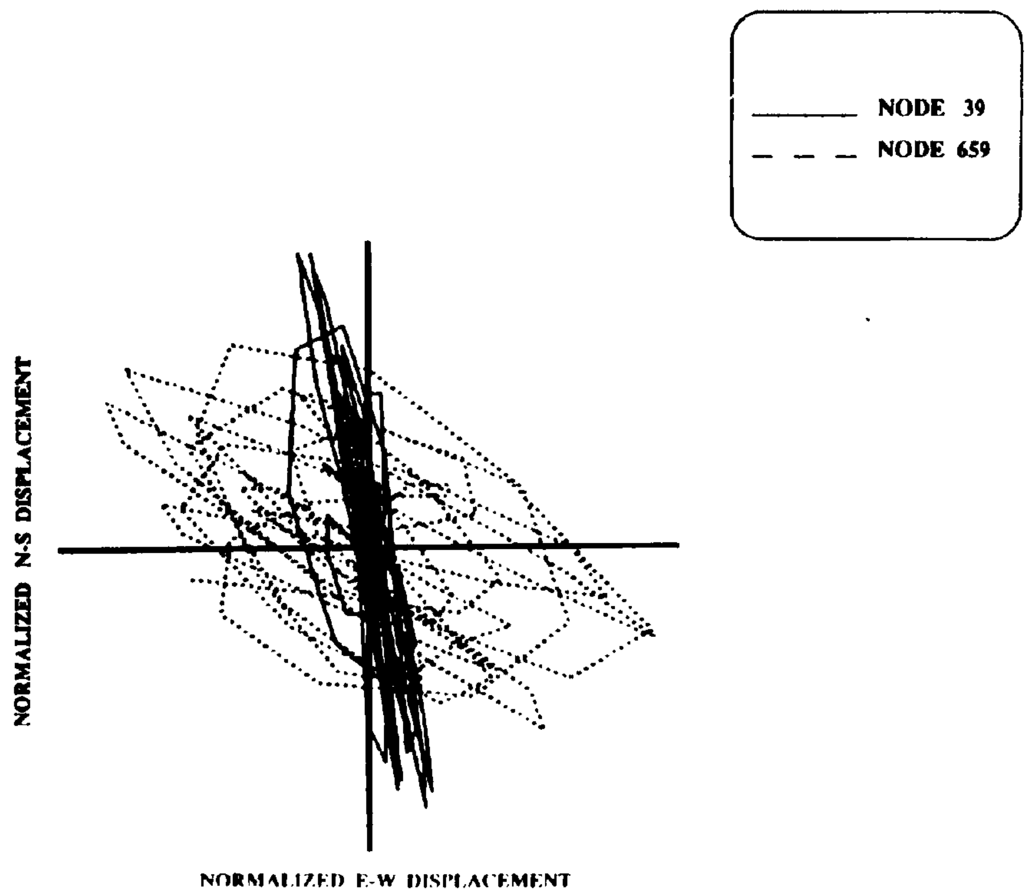


Figure 5.3 Particle Motion of Interested Nodes in the Horizontal Plane (x-y) of the Portland Building

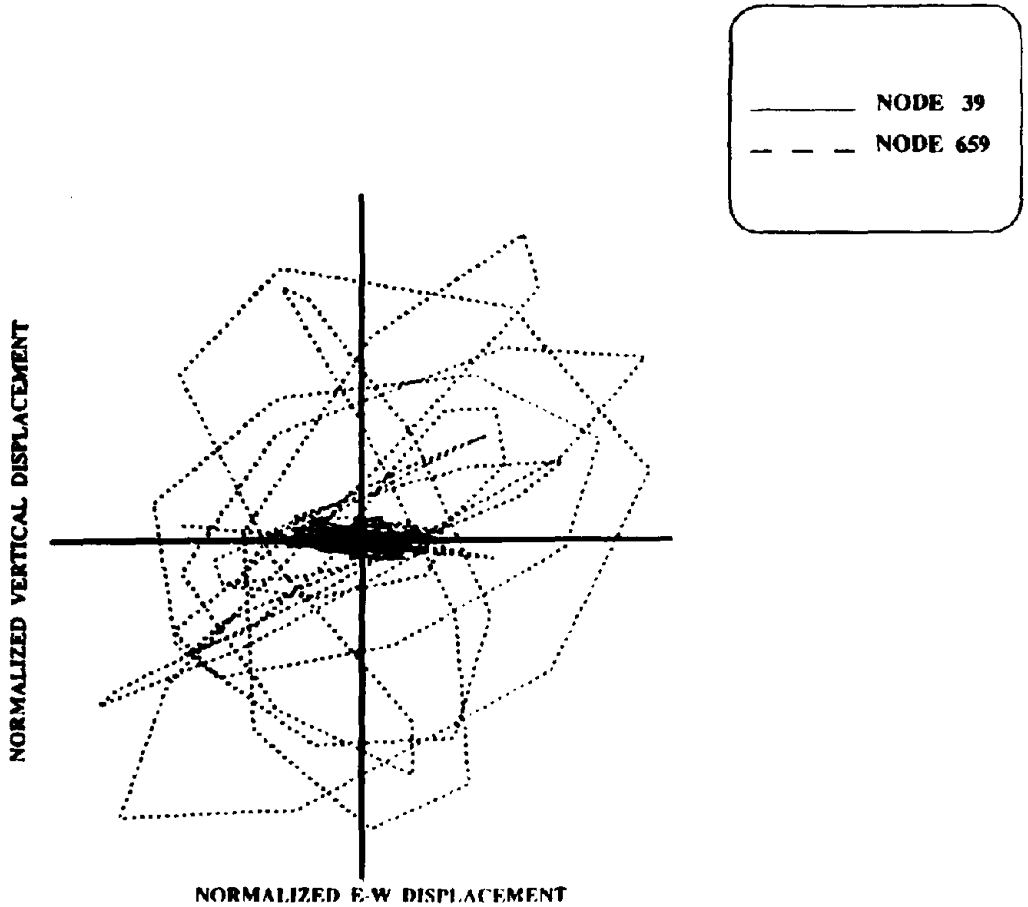


Figure 5.4 Particle Motion of Interested Nodes in the Vertical Plane (x-z) of the Portland Building

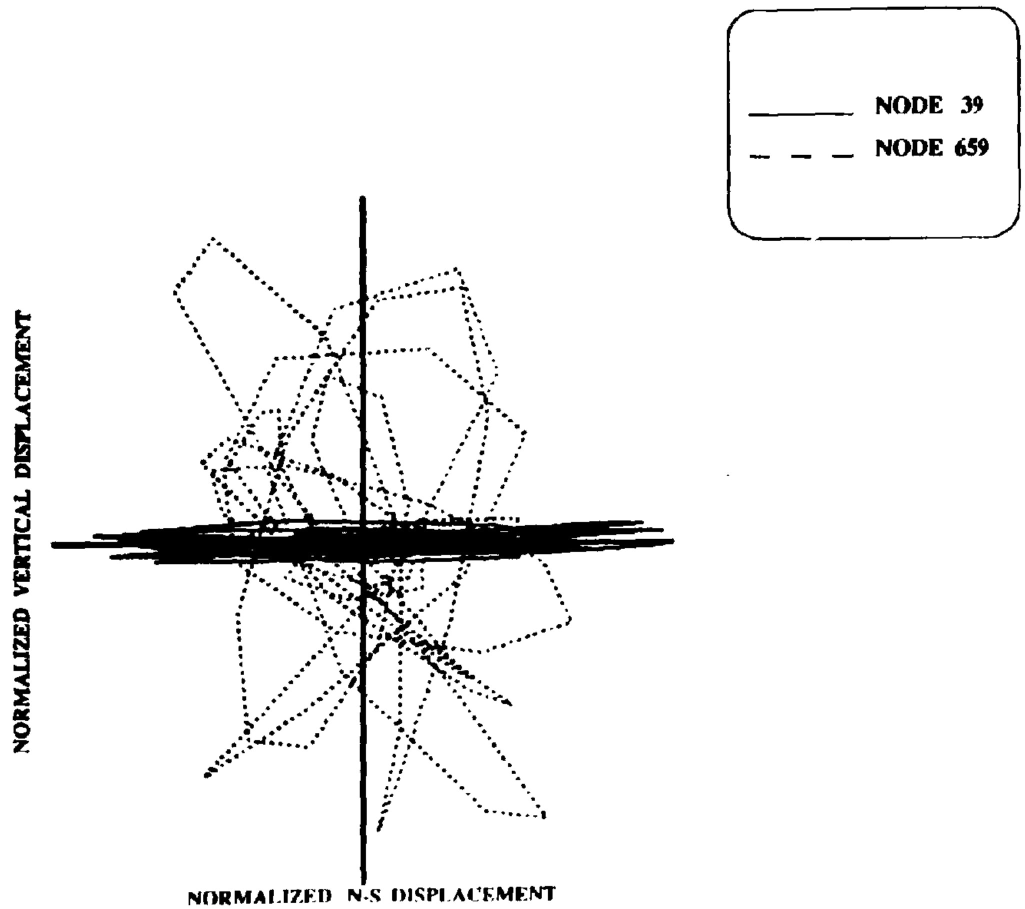
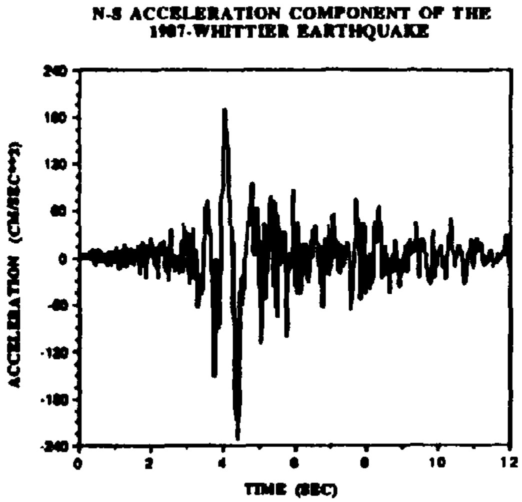
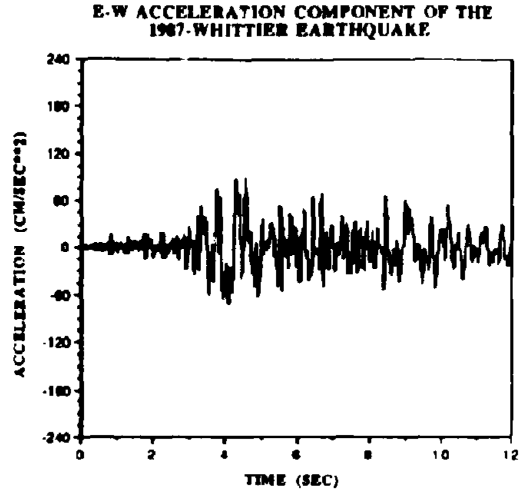


Figure 5.5 Partical Motion of Interested Nodes in the Vertical Plane (y-z) of the Portland Building





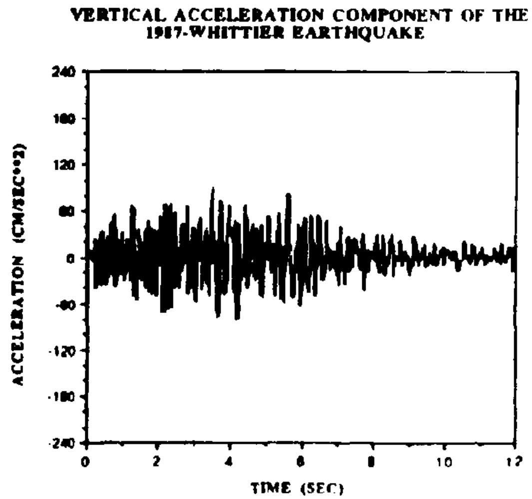
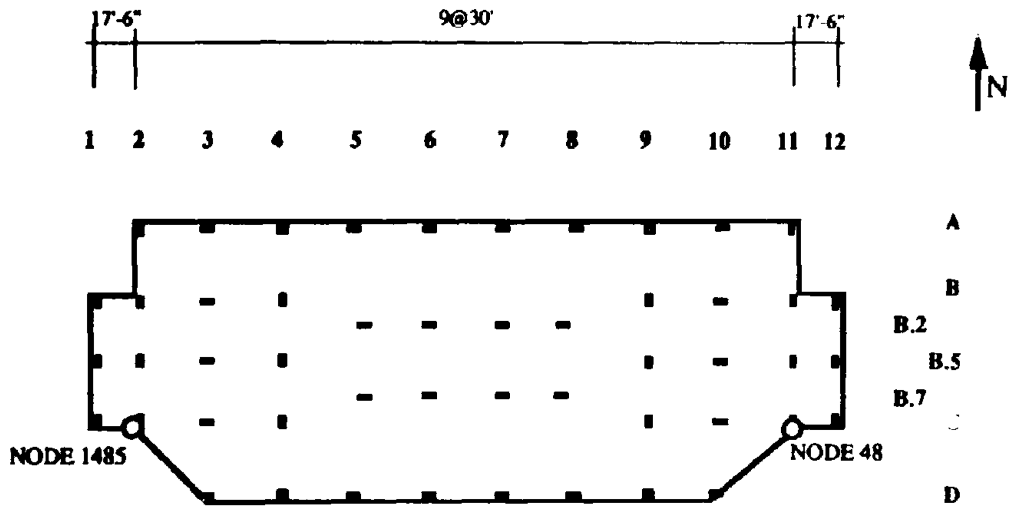


Figure 5.6 Ground Acceleration Records at 12400 Imperial Highway in Norwalk, California, Due to the 1987-Whittier Narrows Earthquake
(a) N-S Component
(b) E-W Component
(c) Vertical Component



FLOOR 8, LONG BEACH BUILDING

Figure 5.7 Locations of Interested Nodes in the Long Beach Building

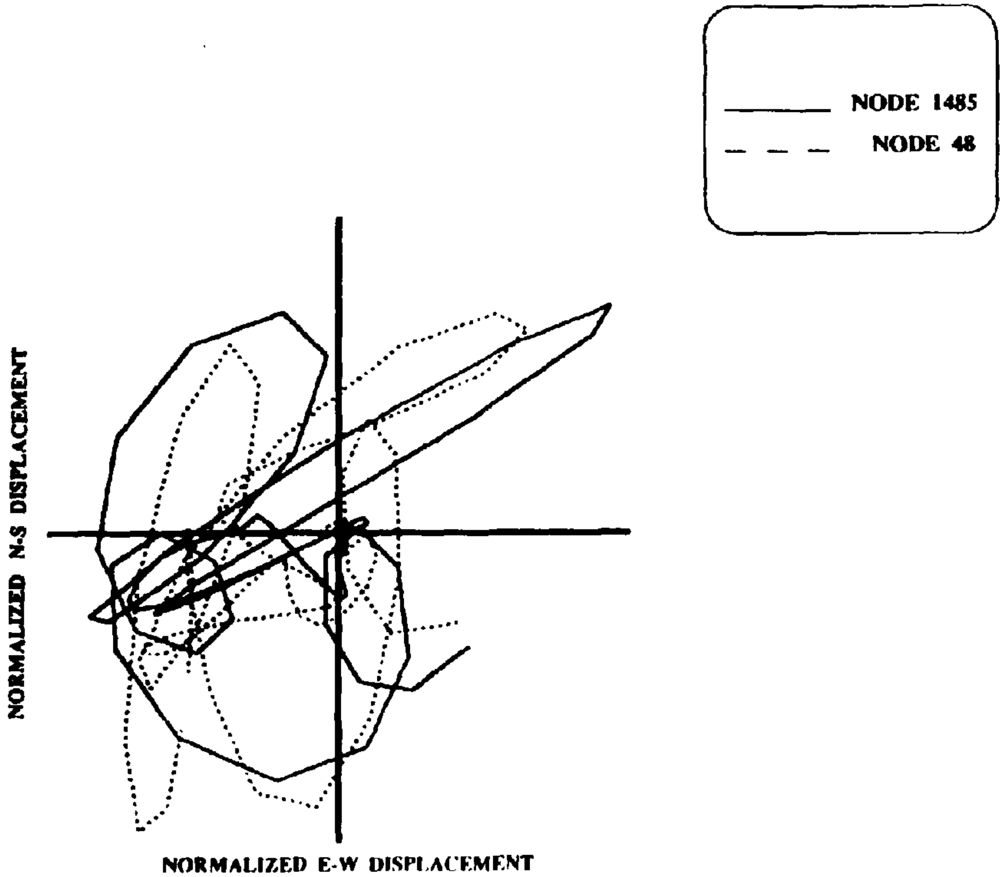


Figure 5.8 Particle Motion of Interested Nodes in the Horizontal Plane (x-y) of the Long Beach Building

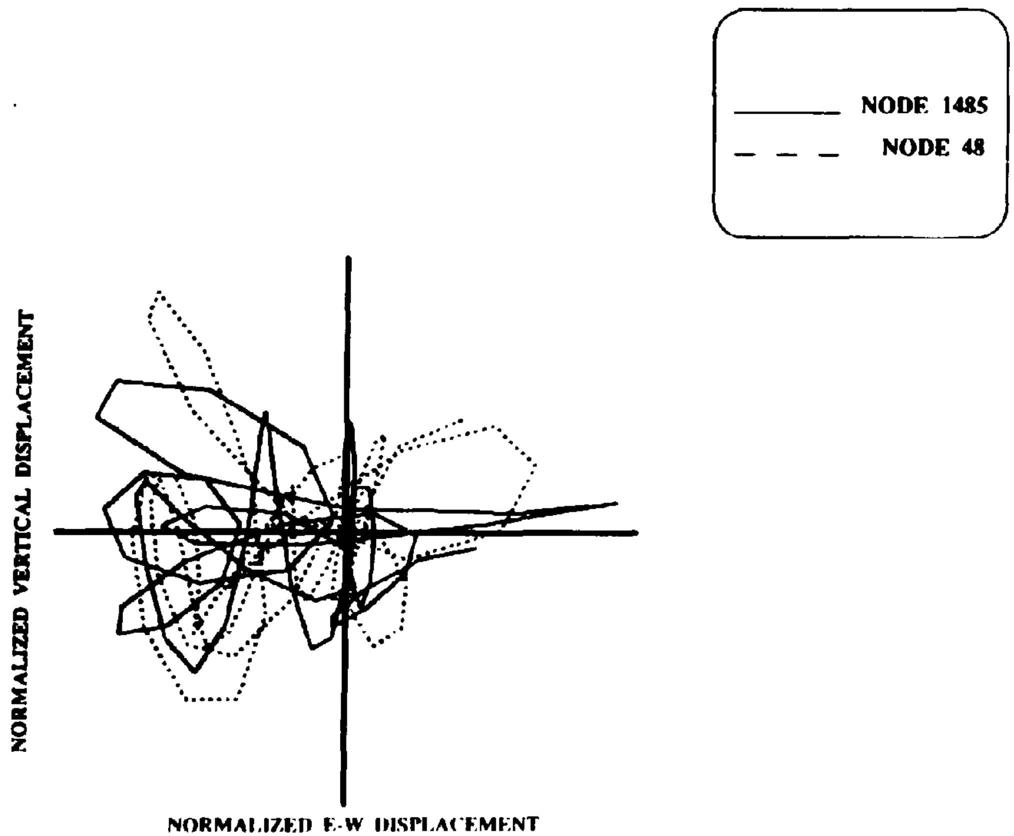


Figure 5.9 Particle Motion of Interested Nodes in the Vertical Plane (x-z) of the Long Beach Building

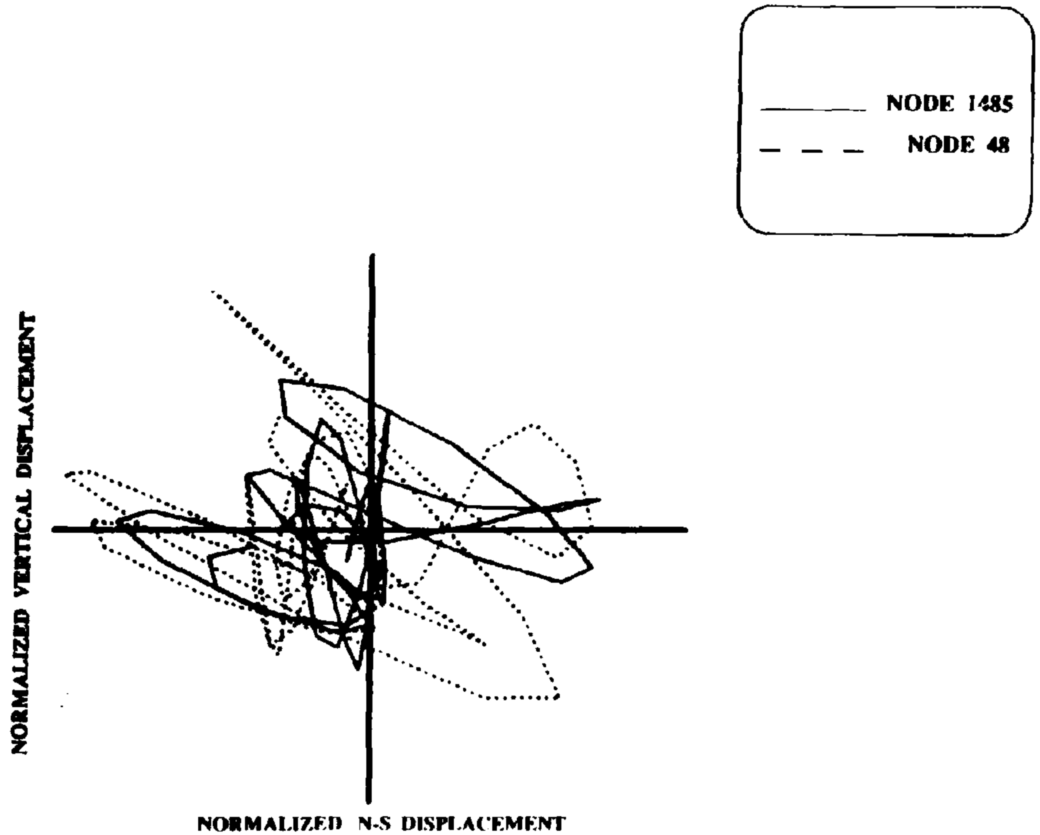


Figure 5.10 Particle Motion of Interested Nodes in the Vertical Plane (y-z) of the Long Beach Building

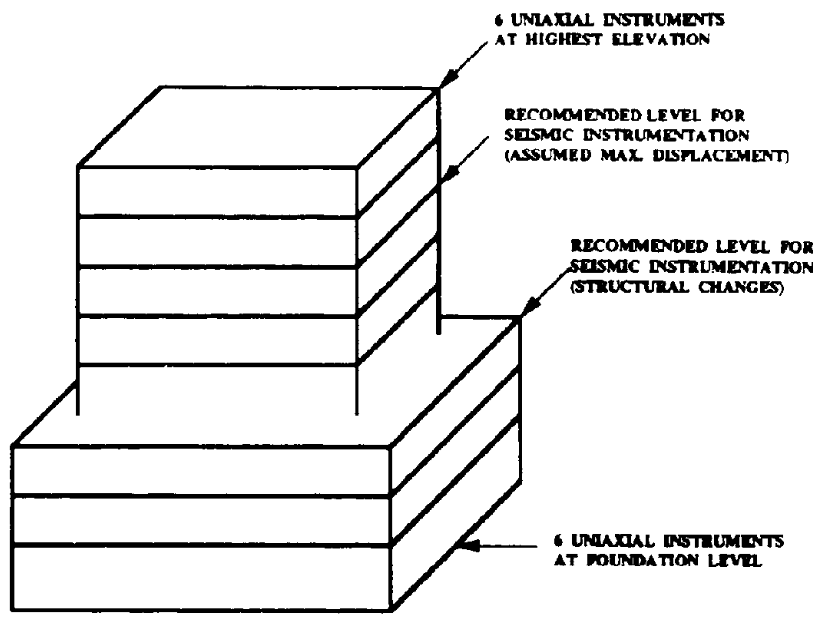


Figure 6.1 Schematic for Seismic Instrumentation along Building Elevation

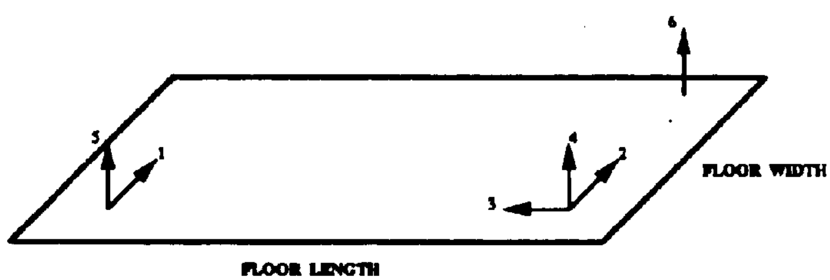


Figure 6.2 Schematic for Deployment of Instrumentation Within a Rigid Floor

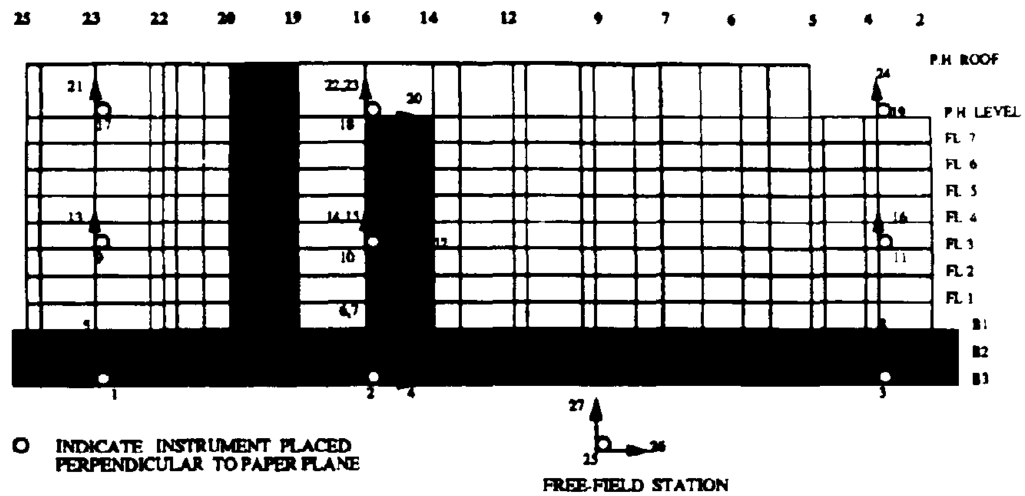


Figure 6.3 Recommended Locations for Seismic Instrumentation in the Portland Building

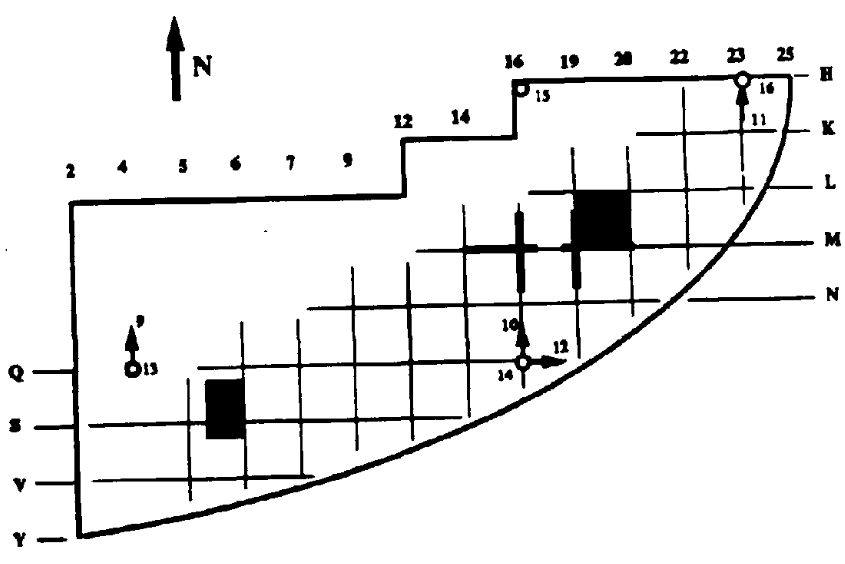


Figure 6.4 Recommended Within-Floor Arrangement of Instrumentation in the Portland Building

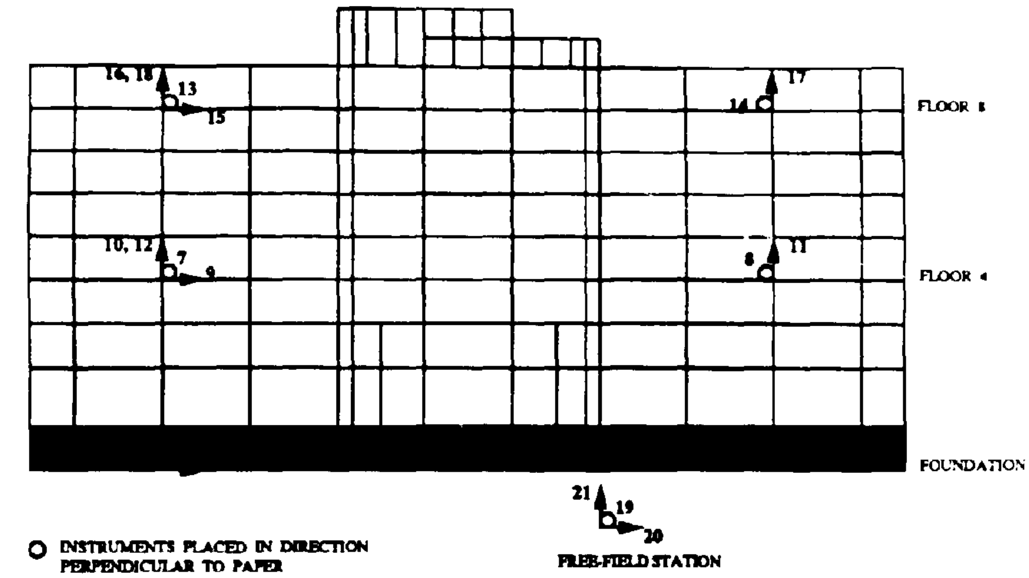


Figure 6.5 Recommended Locations for Seismic Instrumentation in the Long Beach Building

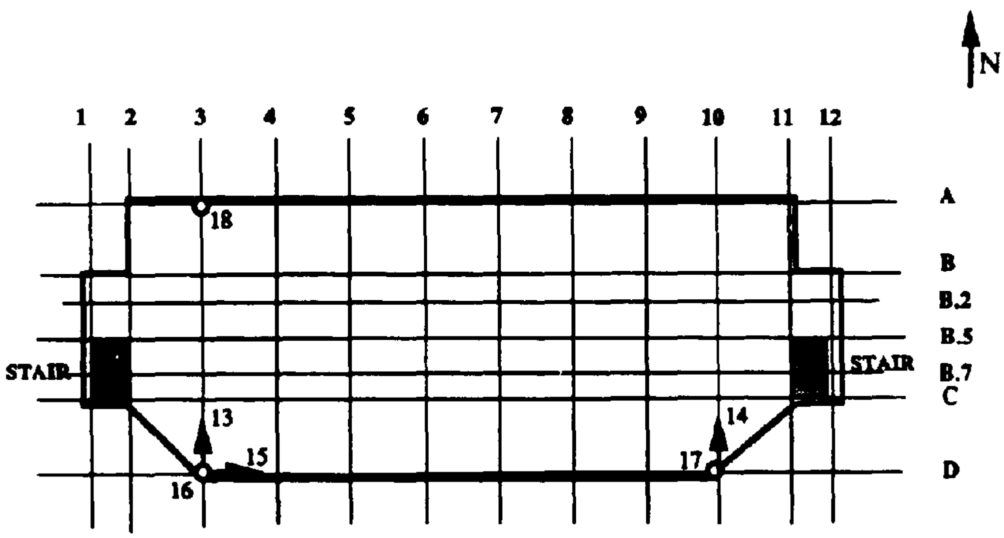


Figure 6.6 Recommended Within-Floor Arrangement of Instrumentation in the Long Beach Building