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EARTHQUAKE ENGINEERING RESEARCH CENTER

DYNAMIC PROPERTIES OF AN EIGHT-STORY PREFABRICATED PANEL BUILDING

by

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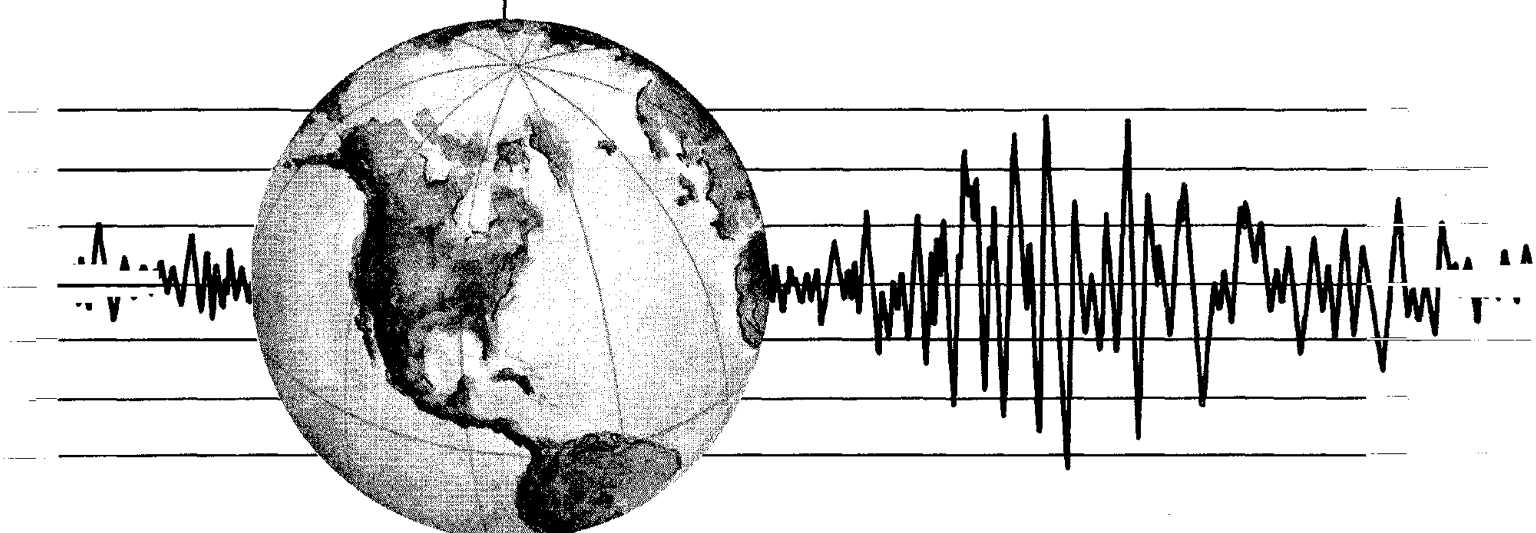
Report to the National Science Foundation



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Rigid floor diaphragm action and serious structure-foundation interaction were observed. Including the foundation flexibility in the analytical model resulted in resonant frequencies and mode shapes showing excellent agreement with the test data.

The results of full scale dynamics studies of an other, structurally identical, 12-story apartment building with a basically identical floor plan, are also presented and indicate a direct proportionality between structural height and fundamental periods.

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A Report to the
National Science Foundation

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J. G. Bouwkamp
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Report No. UCB/EERC-80/30
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ABSTRACT

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1. INTRODUCTION

1.1 General

The design of multistory structures subjected to dynamic forces resulting from foundation motions requires a consideration of both the characteristics of the ground motion and the dynamic properties of the structure. Ground motions as caused by an earthquake are random and, although not prescriptible for aseismic design, have been fairly well studied for certain well-known past earthquakes. The engineer is therefore mainly interested in the dynamic properties of the structure when designing for earthquake forces and is only indirectly concerned with the ground motion characteristics.

High speed digital computers and more sophisticated idealizations and computer model formulations of structures can predict the elastic and, provided that proper non-linear algorithms can be defined, also the inelastic response of such structures when subjected to earthquakes. However, the accuracy of the results in large measure depend upon the computer model formulation of the structure and its foundation. In order to determine the accuracy of the calculated results and to accumulate a body of information on the dynamic properties of structures, especially when these structures have novel design features, dynamic tests have been conducted on full-scale structures (1).

In order to assess particularly the dynamic characteristics of prefabricated type of structures, dynamic tests using both forced and ambient vibration methods were performed on the Los Portales Building in Oakland, California, a "Forest City Dillon" prefab panel structure. Because of the potential advantages of the ambient vibration method in dynamic testing of full-scale structures, it was desirable to compare the ambient and forced vibration results and to assess the accuracy of each method in evaluating the dynamic properties of structural systems.

The building is described in Chapter 2, and the results of the dynamic tests, from the forced, and ambient vibration studies, are given in Chapters 3 and 4, respectively. A comparison of the experimental results obtained from both studies is presented in Chapter 5. For purposes of correlation a mathematical model of the structural system was formulated, and the calculated and experimental dynamic properties were compared. The formulation of the mathematical model and the analytical dynamic properties obtained are described in Chapter 6. Conclusions, incorporating the results of a second, 12-story high "Forest City Dillon" structure with similar floor plan, are presented in Chapter 7.

1.2 Acknowledgement

The authors gratefully acknowledge the financial support provided by the National Science Foundation under Grant PFR-7908257. They also wish to thank the owner, the Spanish Speaking Unity Council of Oakland; the general contractors Hunt Construction Company of Sacramento, Forest City Dillon, Inc. and Teron Pacific; and the Architects, Ogren, Juarez and Gines, especially Mr. Robert A. Ogren, for their help and cooperation in coordinating and carrying out the test program.

2. THE LOS PORTALES BUILDING

2.1 General

The Los Portales Building in Oakland, tested in February 1979, is constructed with reinforced concrete, prefabricated cellular wall panels and solid floor slab elements. At the site reinforcement is placed in the panel cells which are subsequently filled with concrete. As the modular design of the building includes pre-fabricated kitchen and bathroom units, construction progressed at a rate of one story every two days.

2.2 Architectural Layout

The 8-story building (Fig. 2.1) has a height of 70'-8" and overall dimensions of approximately 164' x 80' in plan. It is designed as a housing development for the elderly and therefore modular in concept. The building is serviced by two elevators located in the center, and two stairwells on the east side of either end of the structure (Fig. 2.2).

2.3 Structural System

The structure is a "Forest City Dillon" prefabricated building system. The vertical and horizontal load resistance is provided by reinforced concrete shear walls, oriented in both transverse and longitudinal direction. The wall dimensions, with a typical thickness of 8", are constant over the entire height, except for a few openings on the first floor. The wall elements are cellular. However, placement of vertical reinforcement and 4000 psi concrete in these cells result in a basically monolithic shear wall panel system. The shear walls rest on spread footings which in turn are supported by 12" square prestressed concrete piles, varying in length from 45 to 60 feet.

The "Forest City Dillon" system uses solid reinforced concrete floor elements. The 8 feet wide floor elements have a thickness of 4" and span

22 feet. At the site, a 4" concrete topping is placed on these elements, with reinforcing at the joints between single floor elements (Fig. 2.4). Preassembled kitchen and bathroom units have an 8" thick slab and are constructed with protruding reinforcement to tie into the 4" topping of the adjacent floor panels. Fig. 2.3 shows a typical wall panel system with reinforcing. Details of exterior and interior wall-floor joints are shown in Fig. 2.4.

2.4 Soil Conditions

The following description of the soil conditions at the site and Fig. 2.5 are taken from a report prepared by Converse Davis Dixon Associates, Geotechnical Consultants, San Francisco.

> "From geologic data it appears that the site is underlain by approximately 500 feet of Pleistocene and Holocene age sediments overlying bedrock. The uppermost geologic formation (Temescal) consists of approximately 20 feet of lightly overconsolidated clays and sands and clayey gravels (stream deposits) filling old drainages in the underlying stiff overconsolidated clays.
>
> The exploratory borings encountered two to three feet of low density clayey silt; overlying moderately compressible silty clay and sandy clay extending to depths of five to eight feet. A brown to yellow sandy clay, of slight to moderate compressibility, was encountered below five to eight feet and extended to approximately 15 feet. The clayey soils above eight feet in depth exhibited high strength at their natural moisture content. Upon saturation however, the strength drops significantly.
>
> Below 15 feet a clayey sand and gravel was encountered extending to a depth of approximately 20 feet. The lower two feet of this layer contained coarser gravel and caved rapidly during drilling.
>
> Below 20 feet, stiff to very stiff silty sandy clays with occasional layers of dense sand and clayey sand were encountered extending to the depth of exploration (100 ft.). A definite change in the soil properties occurred at about 40 feet in depth.
>
> Measured strengths below 40 feet are more than twice those between 20 and 40 feet, and densities were on the order of 50 to 10 percent higher below 40 feet.
>
> The groundwater level was observed to be at a depth of 16 feet (elevation 82.8) during drilling. The depth was estimated based on free moisture observed in a sample taken prior to introducing water as drilling fluid. Seasonal fluctations in groundwater level are to be expected."

Fig. 2.1 Los Portales Building, Oakland CA

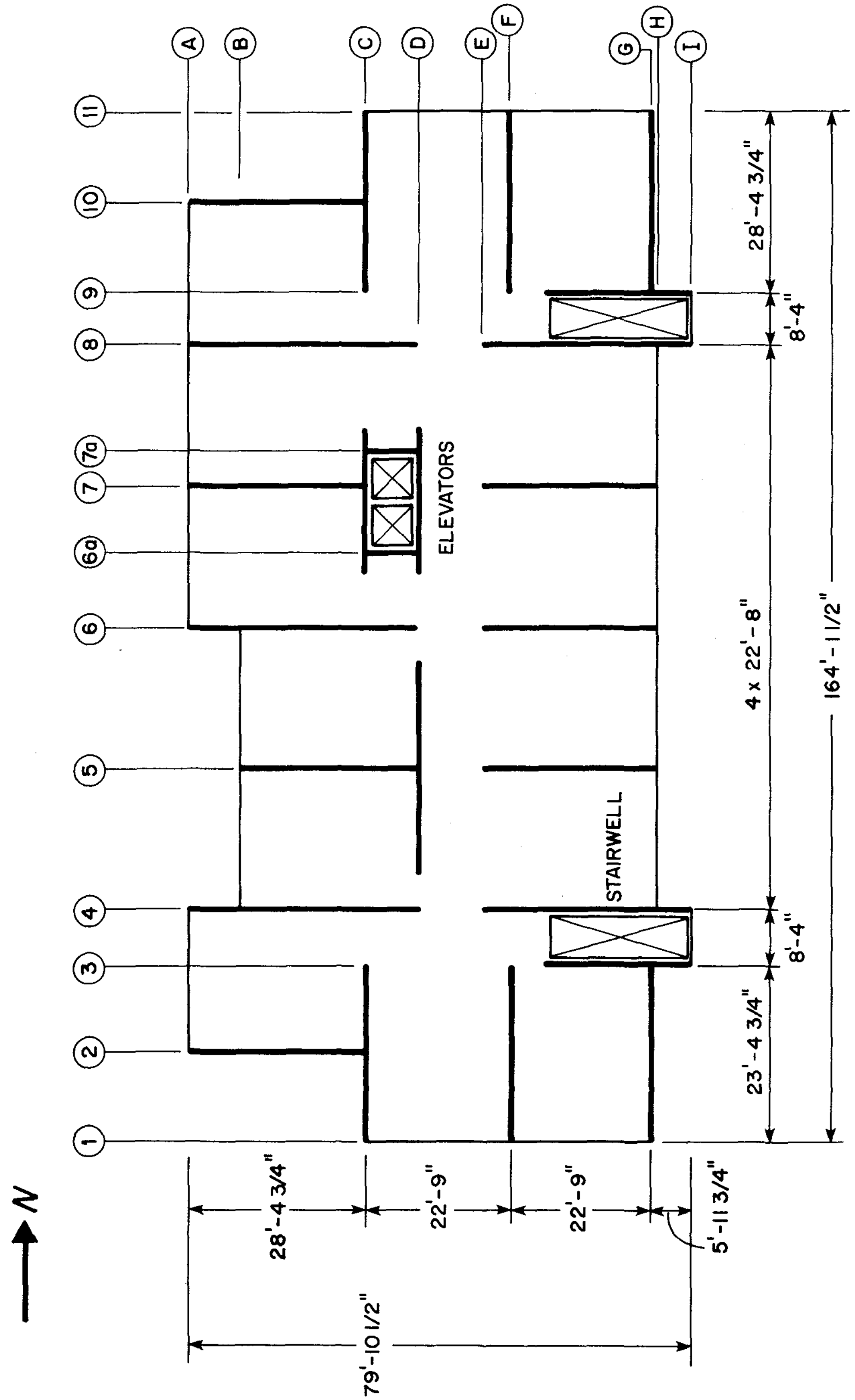


Fig. 2.2 Typical Floor Plan

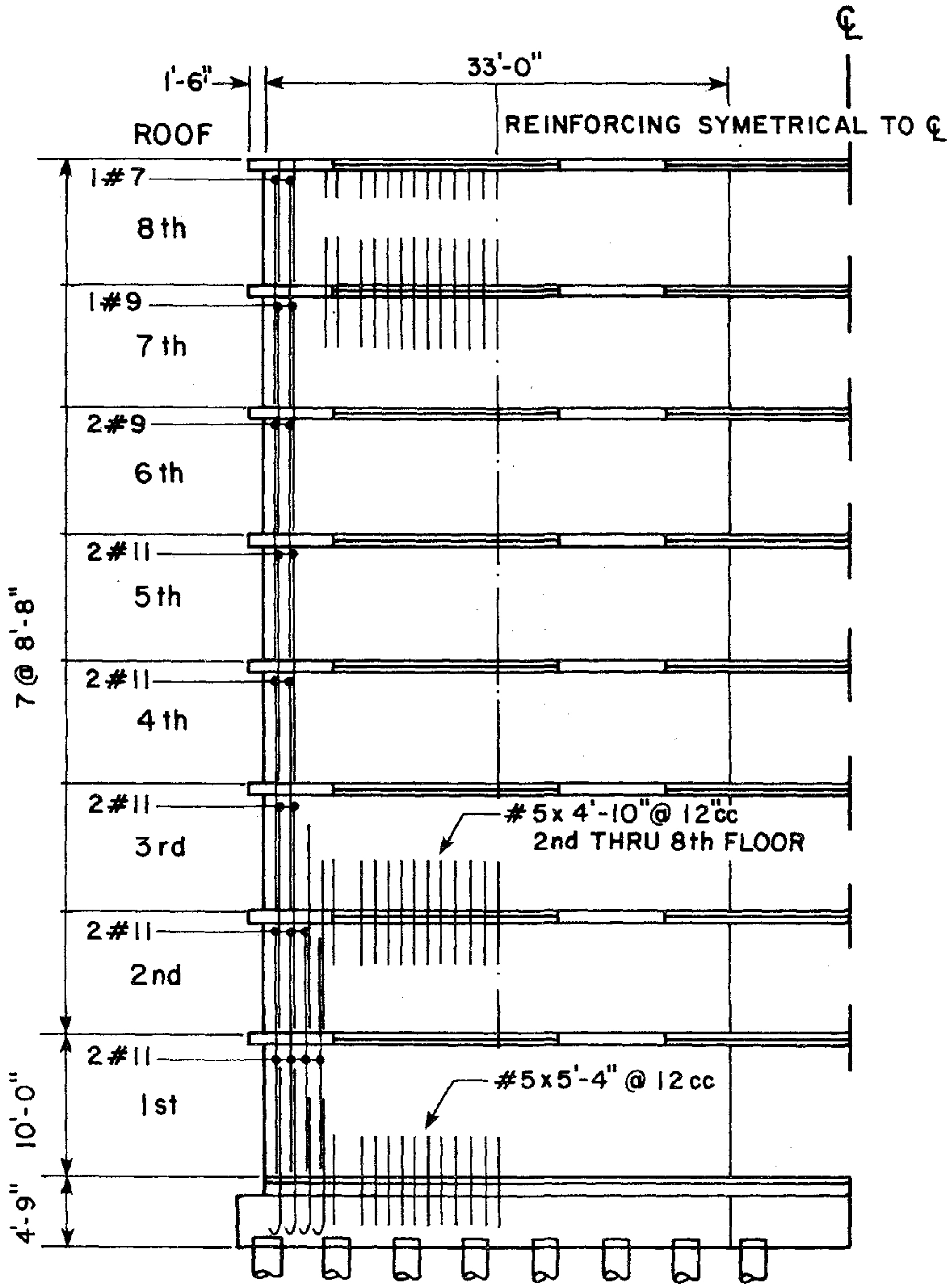


Fig. 2.3 Typical Wall Panel Element

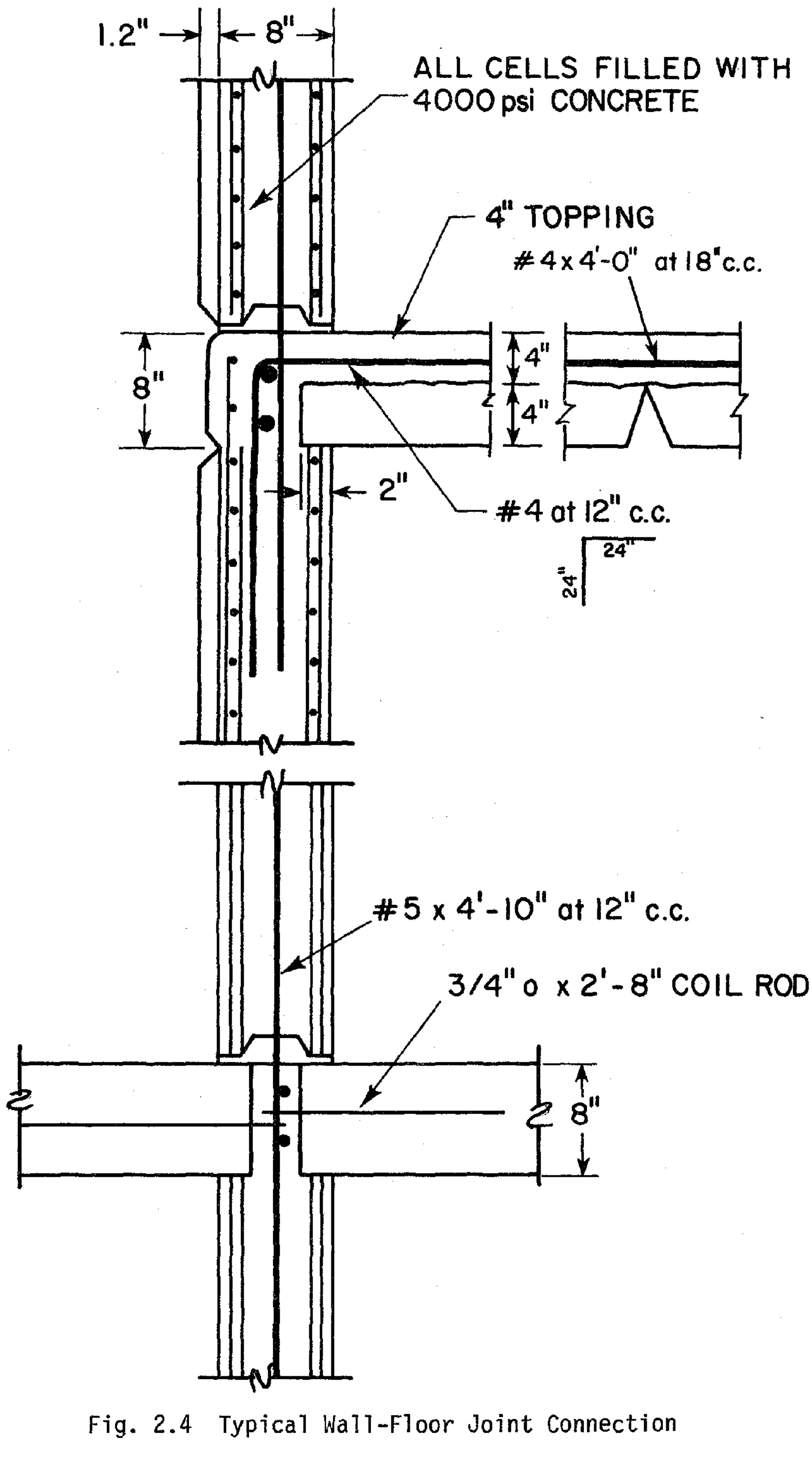


Fig. 2.4 Typical Wall-Floor Joint Connection

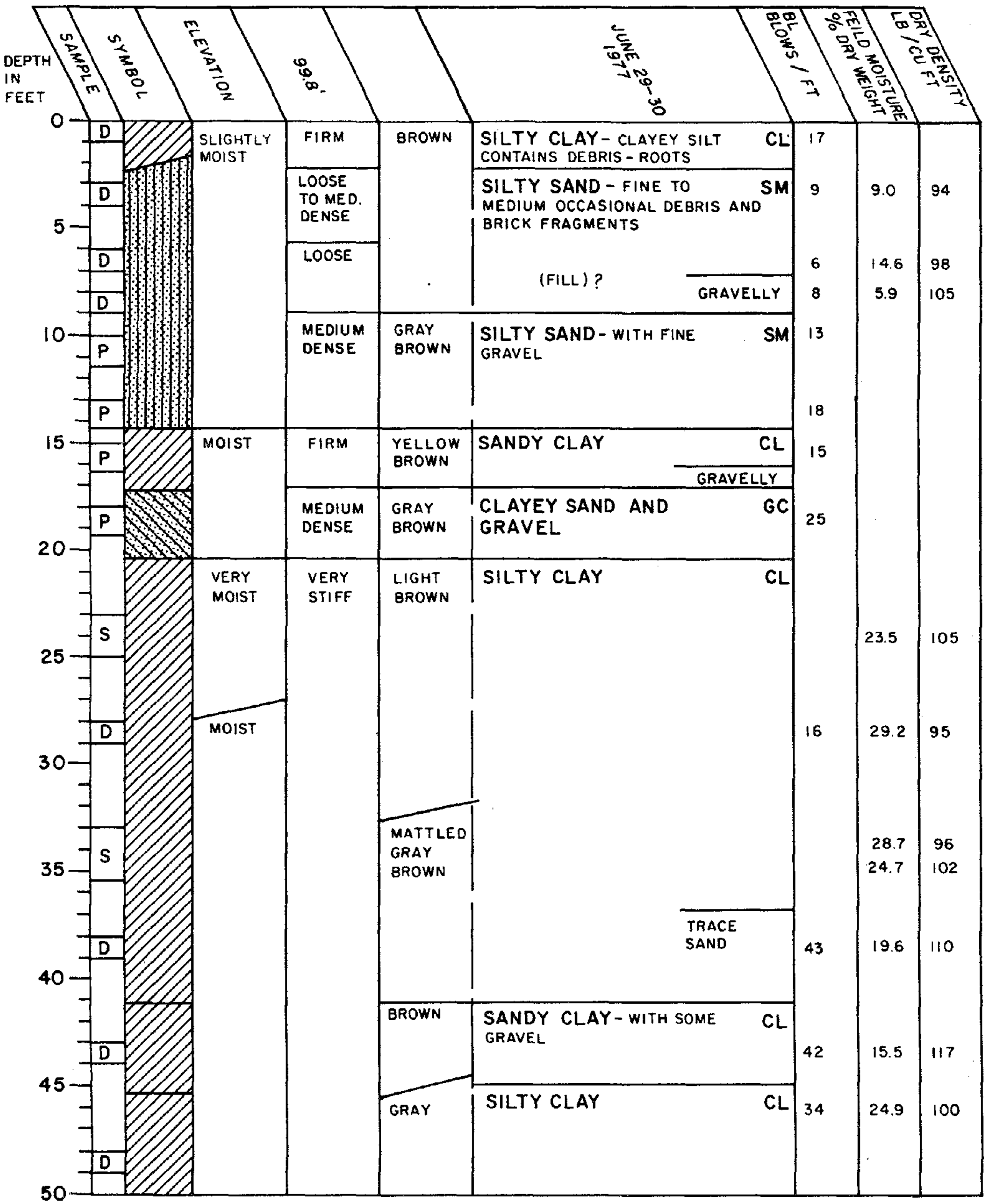


Fig. 2.5 Log of Boring

3. FORCED VIBRATION STUDY

3.1 General

The forced vibration study was carried out and completed during February 1979. The building was structurally completed prior to the experimental work. The experimental apparatus employed in the dynamic test is described below. The general experimental procedures, equipment used, and procedures for data deduction applied, for forced vibration study conducted are also described. Finally, the experimental results are presented and discussed.

3.2 Experimental Apparatus

The experimental apparatus employed in the tests were two vibration generators, twelve accelerometers and equipment for the measurement and recording of the frequency responses. The apparatus is described in the following sections.

3.2.1 Vibration Generators

Forced vibrations were produced by two rotating-mass vibration generators or shaking machines, one of which is shown in Fig. 3.1. These machines were developed at the California Institute of Technology under the supervision of the Earthquake Engineering Research Institute for the Office of Architecture and Construction, State of California. Each machine consists of an electric motor driving two pie-shaped baskets or roters, each of which produces a centrifugal force as a result of the rotation. The two rotors are mounted on a common vertical shaft and rotate in opposite directions so that the resultant of their centrifugal forces is a sinusoidal rectilinear force. When the baskets are lined up, a peak value of the sinusoidal force will be exerted. The structural design of the machines limits the peak value of force to 5,000 lbs. This maximum force may be attained at a number of combinations of eccentric mass and rotational speed, since the output force is proportional to the square of the rotational speed as well as the mass of the baskets and the lead plates inserted in the baskets. The **maximum** force of 5,000 lbs. can be reached for a minimum rotational speed of 2.5 cps when all the lead plates are placed in the baskets. At higher speeds the eccentric mass must be reduced in order not to surpass the maximum force of 5,000 lbs. The maximum operating speed is 10 cps, and the minimum practical speed is approximately 0.5 cps. At 0.5 cps with all lead plates in the baskets, a force of 200 lbs. can be generated. The relationship between output force and frequency of rotation of the baskets for different basket loads is shown in Fig. 3.2.

The speed of rotation of each motor driving the baskets is controlled by an electronic amplidyne housed in a control unit. The control unit allows the machines to be synchronzed or operated 180° out-of-phase. This makes it convenient to excite, in structures with a line of symmetry, either torsional or pure translational vibrations without changing the position of either machine. A complete description of the vibration generators is given in (2).

3.2.2 Accelerometers

The transducers used to detect horizontal floor accelerations of the building were Statham Model A4 linear accelerometers, with a maximum rating of ± 0.25 g.

3.2.3 Equipment for Measurement of Frequency

For the vibration generators, the vibration excitation frequencies were determined by measurement of the speed of rotation of the electric motor driving the baskets. A tachometer, attached to a rotating shaft driven by a transmission belt from the motor, generated a sinusoidal signal of frequency 300 times the frequency measurements was ±1 count in the total number of counts in a period of 1 second (the gating period), i.e., 1/3 of 1% at 1 cps and 1/9 of 1% at 3 cps.

3.2.4 Recording Equipment

The electrical signals for all accelerometers were fed to amplifiers and then to a Honeywell Model 1858 Graphic Data Acquisition System with 8-in. wide chart. In frequency-response tests, the digital counter reading was observed and recorded manually on the chart alongside the associated traces.

3.3 Experimental Procedure and Data Reduction

The quantities normally determined by a dynamic test of a structure are: resonant frequencies, mode shapes, and damping capacities. The experimental procedures are reduction of data involved in determining these quantities are described in the next section.

3.3.1 Resonant Frequencies

With the equipment described on the previous page, resonant frequencies are determined by sweeping the frequency range of the vibration generators from 0.2 to 10 cps.

In the case of the vibration generators, the exciting frequency is increased slowly until acceleration traces on the recording chart are large enough for measurement. Above this level, the frequency is increased in steps until the upper speed limit of the machine is reached. Near resonance, where the slope of the frequency-response curve is changing rapdily, the frequency-interval steps are as small as the speed control permits. These steps are relatively large in regions away from resonance. Each time the frequency is set to a particular value, the vibration response is given sufficient time to become steady-state, before the acceleration traces are recorded. At the same time, the frequency of vibration, either side of the longitudinal centerline, near the center of the building. With the appropriate adjustments to the vibration generator equipment it was possible to apply translational or torsional forces to the building. The first mode in the EW and in the NS directions as well as the first torsional mode were excited.

Frequency response curves for EW, NS and torsional excitation near resonance are shown in Figs. 3.4, 3.5 and 3.6, respectively. The curves are plotted in the form of normalized displacement amplitude versus exciting frequency. The ordinates were obtained by dividing the measured acceleration by the square of the exciting frequency (cps) to obtain acceleration amplitudes for a constant equivalent force amplitude, i.e., the force amplitude that would be generated by the eccentric masses rotating at 1 cps. The values thus obtained are divided by the square of the circular frequency (rad/sec) to obtain normalized displacement amplitudes. The actual exciting force (F_r) and displacement amplitude (u_r) for each of the excited resonancies are given in Figs. 3.4 through 3.6, together with calculated damping values.

The resonance frequencies and damping ratios evaluated from the response curves are summarized in Table 3.1. Also shown in the same table are the

TABLE 3.1 RESONANT FREQUENCIES (f) AND DAMPING RATIOS

EXCITATION	f (cps)	ξ RESONANCE CURVE	ξ DECAY CURVE
EW	3.23	4.7	----
NS	2.68	5.0	4.0
TORSIONAL	2.95	3.7	6.4

damping ratios as derived from the free vibration decay curves. These curves were obtained by recording the building free vibration, following sudden stoppage of the vibration generators. The resulting data for the fundamental NS and torsional frequencies are plotted in the form of logarithmic decay as recorded on a digital counter, is observed and written on the chart with its corresponding traces. Plotting the vibration response at each frequency step results in a frequency-response curve.

Frequency-response curves in the form of acceleration amplitude versus exciting frequency may be plotted directly from the data on the recording chart. However, the curves are for a force which increases with the square of the exciting frequency, and each acceleration amplitude should be divided by the corresponding square of its exciting frequency to obtain so-called normalized curves equivalent to those for a constant force (assuming linear stiffenss and damping for the structural system). If the original acceleration amplitudes are divided by the frequency to the fourth power, displacement frequency-response curves for constant exciting forces are obtained. In cases of fairly low damping (under 5%), there is little difference between results obtained for resonant frequencies and damping capacities measured from the different curves.

3.3.2 Mode Shapes

Once the resonant frequencies of a structure have been found, the mode shapes at each of these frequencies may be determined. In this case, with twelve accelerometers available, it was decided to evaluate the mode shapes by measuring accelerations at each of the eight floors. An extra accelerometer was kept on the 8th floor and three accelerometers were used on the ground floor to record vertical and horizontal base motions.

The structure was vibrated at each of the resonant frequencies, and the vibration amplitude was determined for all accelerometers at each frequency.

It is generally necessary to make corrections to the recorded amplitudes to compensate for differences between calibration factors for each accelerometer. Absolute calibration is not required for mode shapes, and cross-calibration is sufficient. The accelerometers when the structure is vibrated at each of the resonant frequencies. Cross-calibration is generally carried out at the beginning and end of each day. The average calibration factors as derived from the pre- and post-test cross-calibration runs are used to adjust the recorded amplitude.

In general, the number of points required to define a mode shape accurately depends on the mode and the number of degrees of freedom in the system. For example, in dynamic test on a 15-story building (3) four points were sufficient to define the first mode, whereas it required measurements of the vibration of all 14 floors and the roof to define the fifth mode shape accurately.

3.3.3 Damping Capacities

Damping capacities may be found from resonance curves in the normalized frequency-response curves by the formula:

$$\xi = \frac{\Delta f}{2f}$$

where

ξ = dmaping factor,

f = resonant frequency

Δf = difference in frequency of the two points on the resonance curve with amplitudes of $1/\sqrt{2}$ times the resonant amplitude.

Strictly, the expression for ξ is only applicable to the displacement resonance curve of a linear, single degree-of-freedom system with a small amount of viscous damping. However, it has been used widely for systems differing appreciably from that for which the formula was derived, and it has become accepted as a reasonable measure of damping. In this respect, it should be remembered that in the case of full-size civil engineering structures, it is not necessary to measure damping accurately in a percentage sense. It is sufficient if the range in shich an equivalent viscous damping coefficient lies is known. Meaningful ranges might be defined as: under 1%, 1-2%, 5-10%, over 10% (1, 4).

The bandwidth method described above is extremely useful when the damping factor lies in the range of 1-10% of critical. However, if the damping lies below 1%, difficulties may be encountered in observing sufficient points on the resonance curve. Also, the small frequency difference between two relatively large frequencies becomes difficult to measure accurately. Above 10% of critical damping, resonance curves often become poorly defined due to interference between modes, and the results from the bandwidth method have little meaning.

3.4 Experimental Results

The vibration equipment was bolted to the 8th floor throughout the test program as shown in Figure 3.3. Also shown are the centers of stiffness (C.S.) and mass (C.M.) as derived analytically. The decision to position the two vibration generators on either side of the longitudinal axis near the transverse center wall was based on the experience of earlier force vibration studies carried out on the twelve story Wesley Manor Building (5). This structure was also of the Forest City Dollon type and had a floor plan similar to the Los Portales building. However, in these earlier studies the two vibration generators were placed on the 12th floor on either end of the building at the same side of the longitudinal center line. Although the machines were positioned closely to the center line the torsional imput, even under a longitudinal, supposedly translational, forcing condition, prevented the development of a clear translational excitation. In fact, because of the particular layout of the floor plan, the Wesley Manor Building was found to be particularly susceptable to a serious coupling of the longitudinal translational and rotational motions. As a result it was impossible to develop a clear translational resonance conditions under the longitudinal excitation. Based on this experience it was decided to place the two vibration generators in the Los Portales studies on curves in Figures 3.7 and 3.8, respectively. The resulting damping ratios are presented in Table 3.1.

The exciting force generated by both shaking machines and the corresponding resonant amplitudes at the 8th floor level, for each response frequency, are given in Table 3.2.

TABLE 3.2 SUMMARY OF THE BUILDING RESPONSE AT RESONANCE

EXCITATION	f (cps)	FORCE OF MOMENT	RESPONSE AT 8th FLOOR (in)	
			CENTER	NORTH END
EW	3.23	6975 lbs	4.13×10^{-3}	-----
EW	3.23	3910 lbs	2.31×10^{-3}	-----
NS	2.68	6744 lbs	8.69×10^{-3}	-----
NS	2.70	938 lbs	0.97×10^{-3}	-----
TORSION	2.95		-----	6.83×10^{-3}

The vertical mode shapes that were excited are shown in Figures 3.9 to 3.11. The EW and NS components of these mode shapes have been plotted along three axes, located along the vertical center lines of the south, mid and north sections of the building. Typically, motions in the NS direction have been plotted to the right of the vertical axes, and motions in the EW direction to the left. In these studies particular attention was paid to assessing the in-plane floor diaphragm action. Hence, the 8th and 4th floors were studied specifically. The horizontal floor modes at resonance are presented in Figures 3.12 through 3.14.

3.5 Discussion of Experimental Results

Because of operational limitations, frequencies higher than 6.75 cps could not be excited. Hence, only the fundamental EW, NS and torsional resonance frequencies and mode shapes could be excited.

Whereas the transverse, EW, mode is quite clean, with only small contributions of NS and rotational components, the longitudinal, MS, and torsional modes are highly coupled. This phenomenon is reflected in the NS frequency response curve as presented in Fig. 3.5. With a NS forcing direction, the response curve shows closely spaced peaks at about 2.68 and 2.98 cps. Similarly, the torsional frequency response curve, created under a torsional, 180° out-of-phase, forcing condition and presented in Figure 3.6, shows a distinct peak at 2.95 cps and a response plateau at about 2.7 cps. The latter seems to reflect the NS resonant frequency of 2.68 cps. The peak response in each of the two curves clearly identifies the NS resonance frequency as 2.68 cps, and the torsional resonance frequency as 2.95 cps. This observation is supported also by the floor modes, as the degree of torsional coupling between NS translation and rotation, permits identification of the translational (NS) and torsional resonance frequencies; namely, the larger rotation identifies the torsional resonance condition, while the smaller one reflects the translational frequency. As the in-plane floor mode shape at 2.68 cps (see Fig. 3.13) has a smaller rotational component than the one at 2.95 cps (Fig. 3.14), the lower resonance frequency is identified as the fundamental translational NS frequency. On the same basis, the floor mode shape at the 2.95 cps resonance frequency, with the larger rotational component, is termed the fundamental torsional frequency.

The floor modes at resonance were observed for the 8th and 4th floors. The results clearly indicate that the floor slabs behaved basically as rigid diaphragms; an observation essential in the development of the analytical model formulation.

During the forced vibration tests under EW excitation, pilot tests indicated a noticable movement of the ground surface around the building. In order to gain information about these soil displacements under EW resonance conditions, horizontal and vertical ground accelerations were measured at points located along parallel lines 20 and 40 ft west of the building. Modal ground surface displacement as well as horizontal and vertical modal displacements at the ground floor level of the structure are shown in Figure 3.15. Results indicated that horizontal and vertical ground surface motions, 40 feet away from the building, were as much as 4 and 8% respectively, of the normalized translational motion at the 8th floor.

The effect of the foundation flexibility on the mode shapes was found to be significant. The horizontal modal displacement under EW resonance at the center of the ground floor was 19% of the normalized 8th floor displacement. For NS resonance the corresponding percentage was 13%. Similarly, under torsional resonance conditions, the modal displacements at the midpoints of the north and south walls amounted to respectively 23 and 29% of this NS normalized modal displacement (at the 8th floor). Considering the base displacement and base rotation, approximately 40 to 50% of the total modal displacement at the 8th floor level is due to the foundation flexibility and associated rigid body motion.

Damping ratios were calculated from both the frequency response curves (Figs. 3.4 through 3.6) and the logarithmic decay curves (Figs, 3.10 and 3.11). The results, as presented in Table 3.1, do not show a consistant relationahip between the values derived by the two different methods. However, a general, relatively large, damping ratio of approximately 5% can be noted.

Finally, a comparison of the experimentally derived periods as resonance and those obtained by using the UBC formula $T = 0.05\ H/\sqrt{D}$, is significant. The two differnet resonance data are presented in Table 3.3.

TABLE 3.3 FUNDAMENTAL PERIODS (EXPERIMENTAL AND UBC)

EXCITATION	EXP. PERIOD	UBC PERIOD	DIFFERENCE
EW	0.31	0.40	-24%
NS	0.37	0.28	+32%
TORSION	0.34	--	--

The result clearly indicates that for this type of structure the UBC method provides erroneous values for the fundamental period. Actually, the overall building dimensions (H and D) used in the recommended formula, do not appropriately capture the stiffness effect of the wall layout. In this case, the stiffness in the shorter EW direction is actually larger than the stiffness in the longer NS direction (T_{EW} = 0.31 sec. and T_{NS} = 0.37 sec.). Comparing these values with the Code values of 0.40 sec. and 0.28 sec., respectively, illustrates a serious inadequacy of the Code to capture the true dynamic characteristics.

As the above comparison considers experimental data of a flexible based structure versus code values of a rigid based structure, a certain adjustment seems necessary. Considering that the rigid body base rotation causes about 70% of the total lateral deformation, the actual structural deformation is only about 30%. This would imply that for a rigidly founded structure the experimental periods T_{EW}^* and T_{NS}^* would reduce to 0.17 sec. and 0.20 sec. respectively. These adjusted values show an even larger discrepancy in comparison to the Code determined values. In general these smaller periods, as derived from the experimental data, would result in significantly larger earthquake design forces than the UBC would imply.

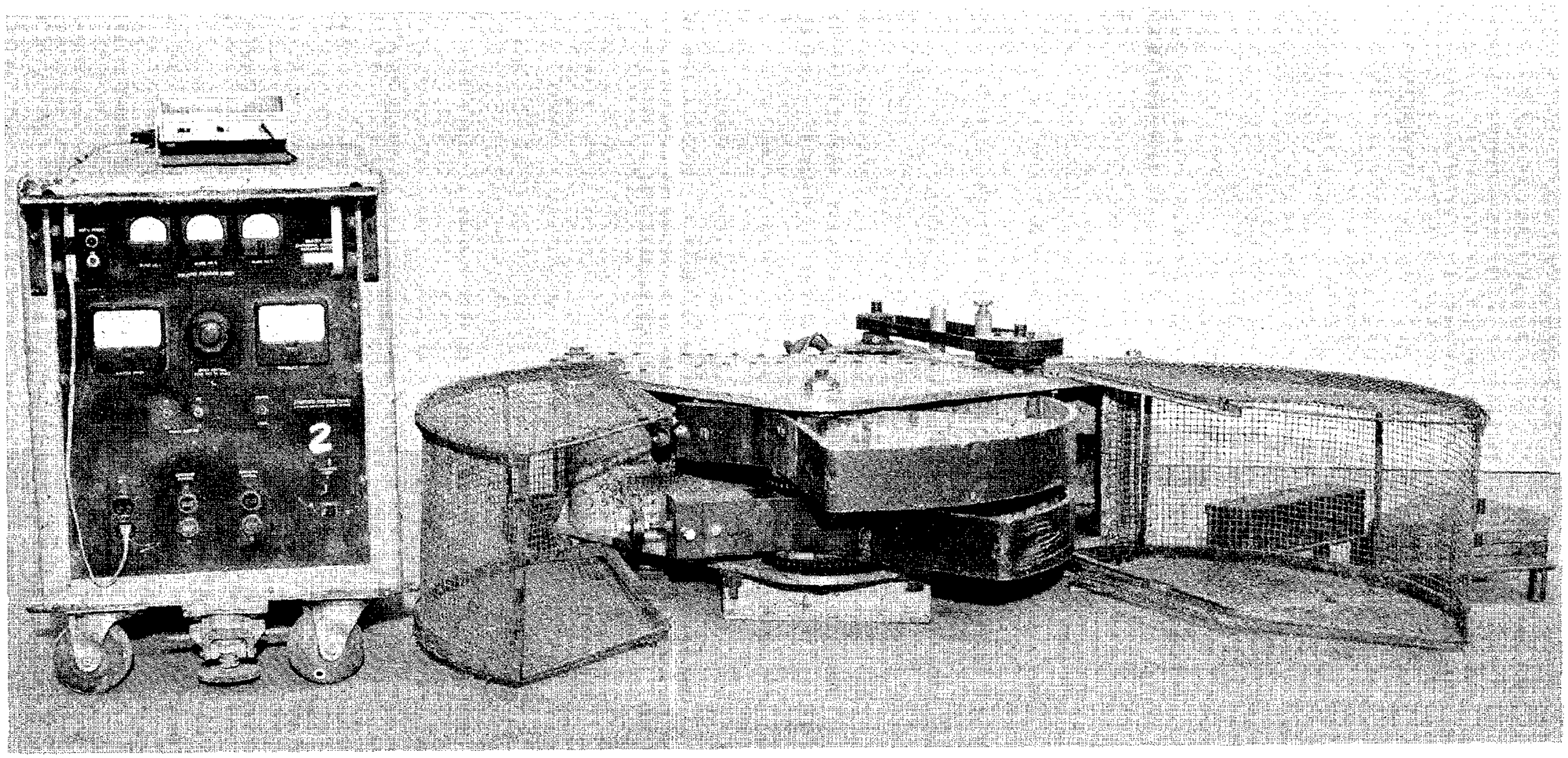


Fig. 3.1 Vibration Generator

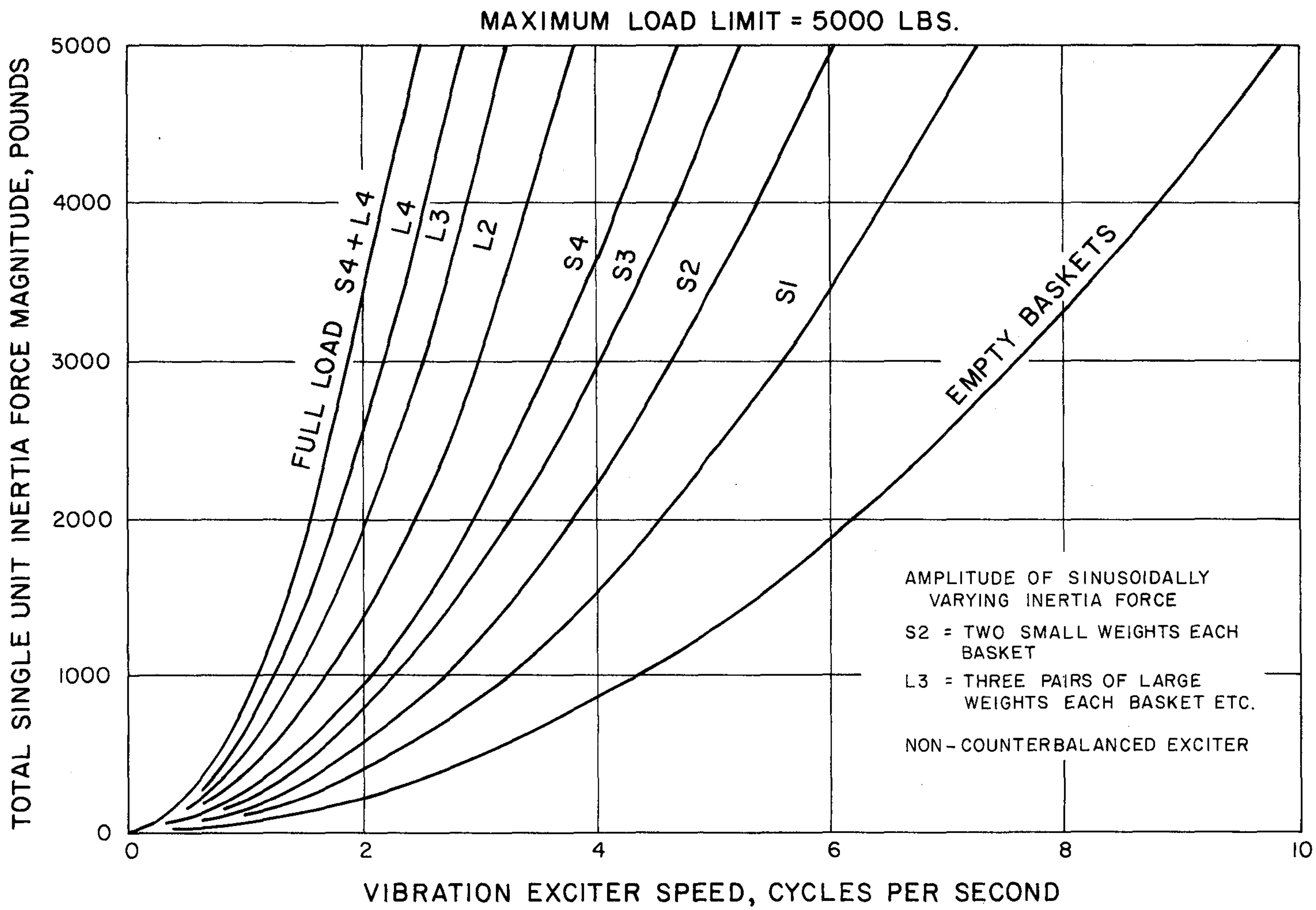


Fig. 3.2 Vibration Force Output vs Speed Non-Counterbalanced

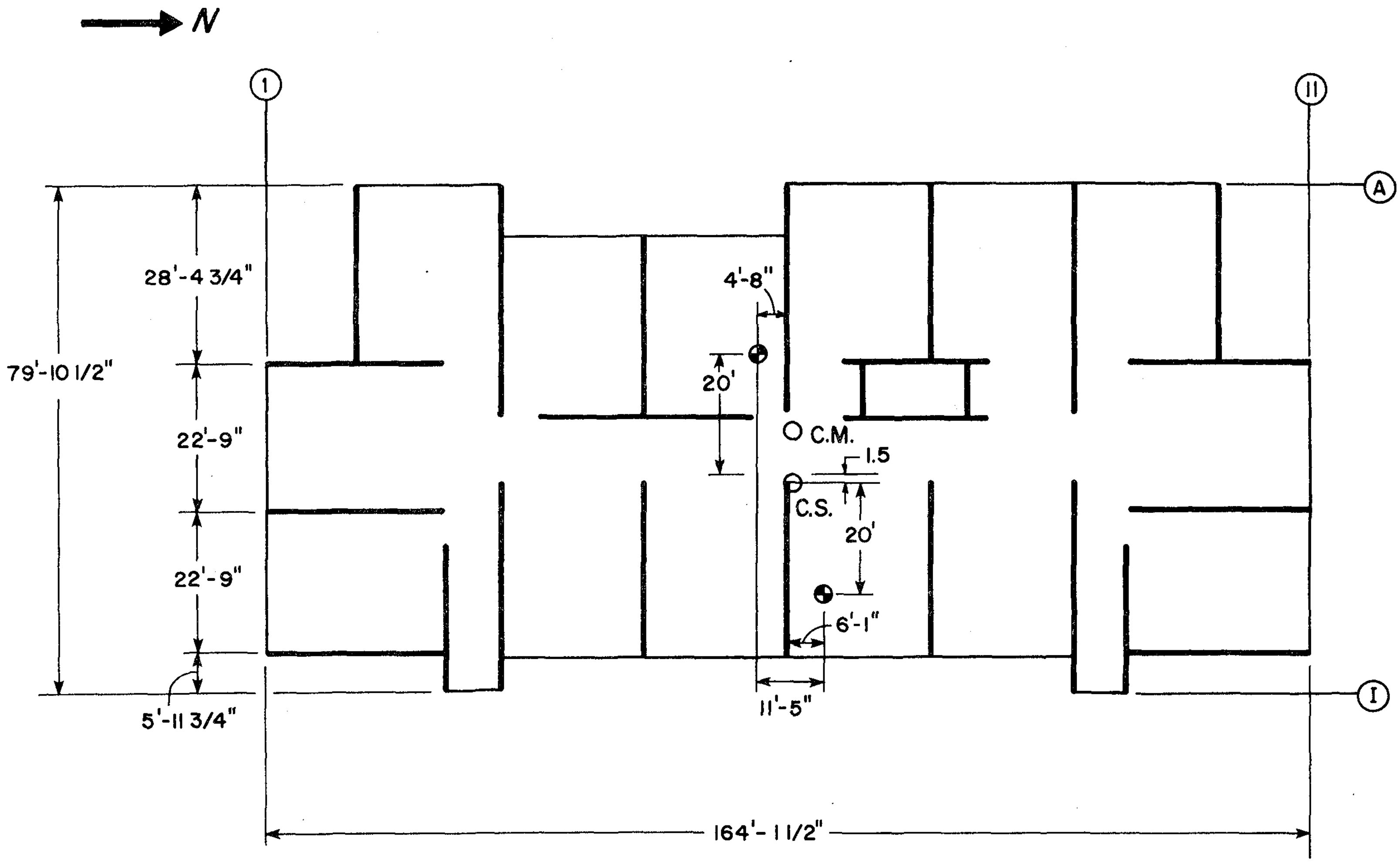


Fig. 3.3 Location of Vibration Generators
Center of Stiffness (C.S.), and Central Mass (C.M.)

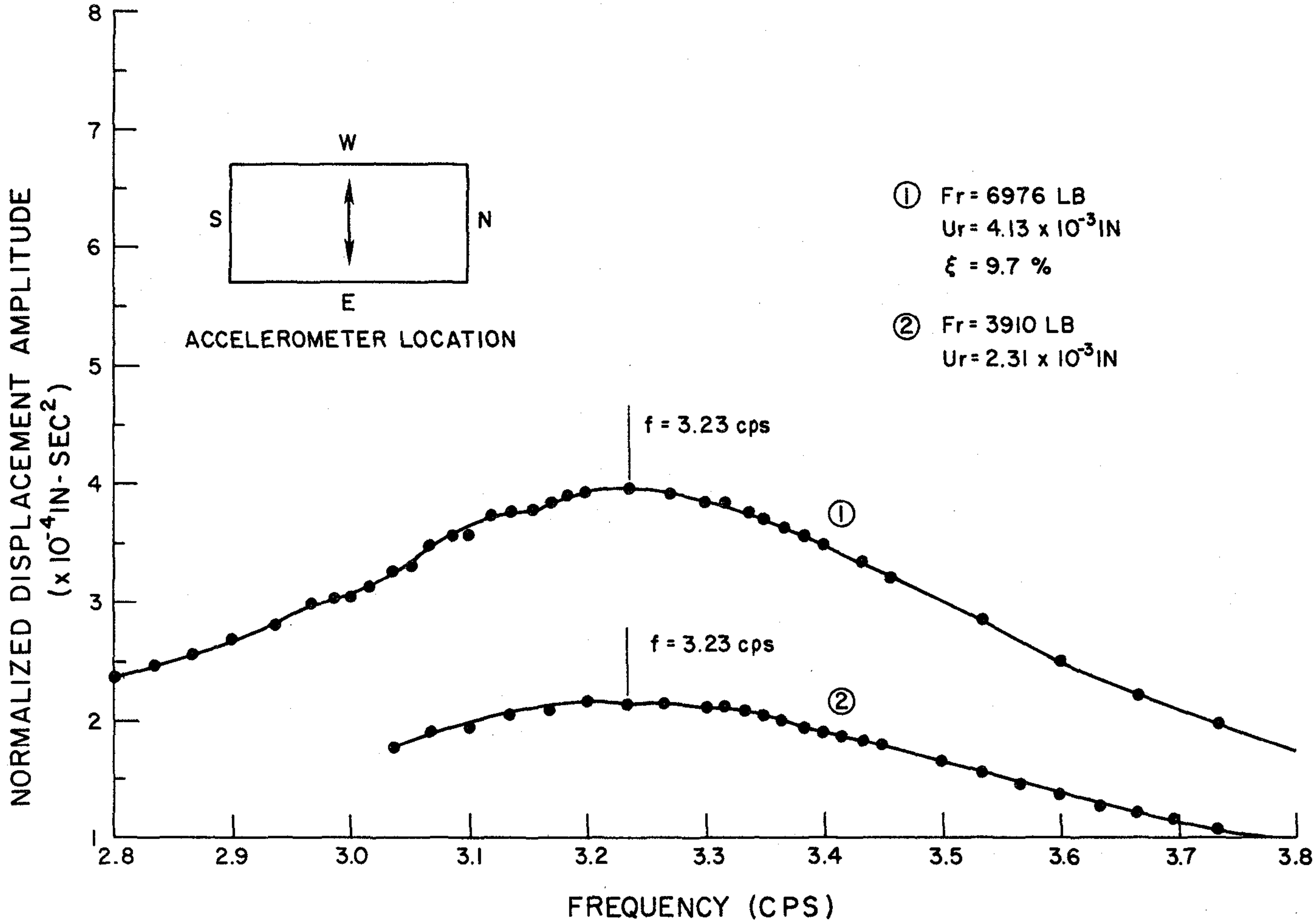


Fig. 3.4 Frequency Response for EW Forcing

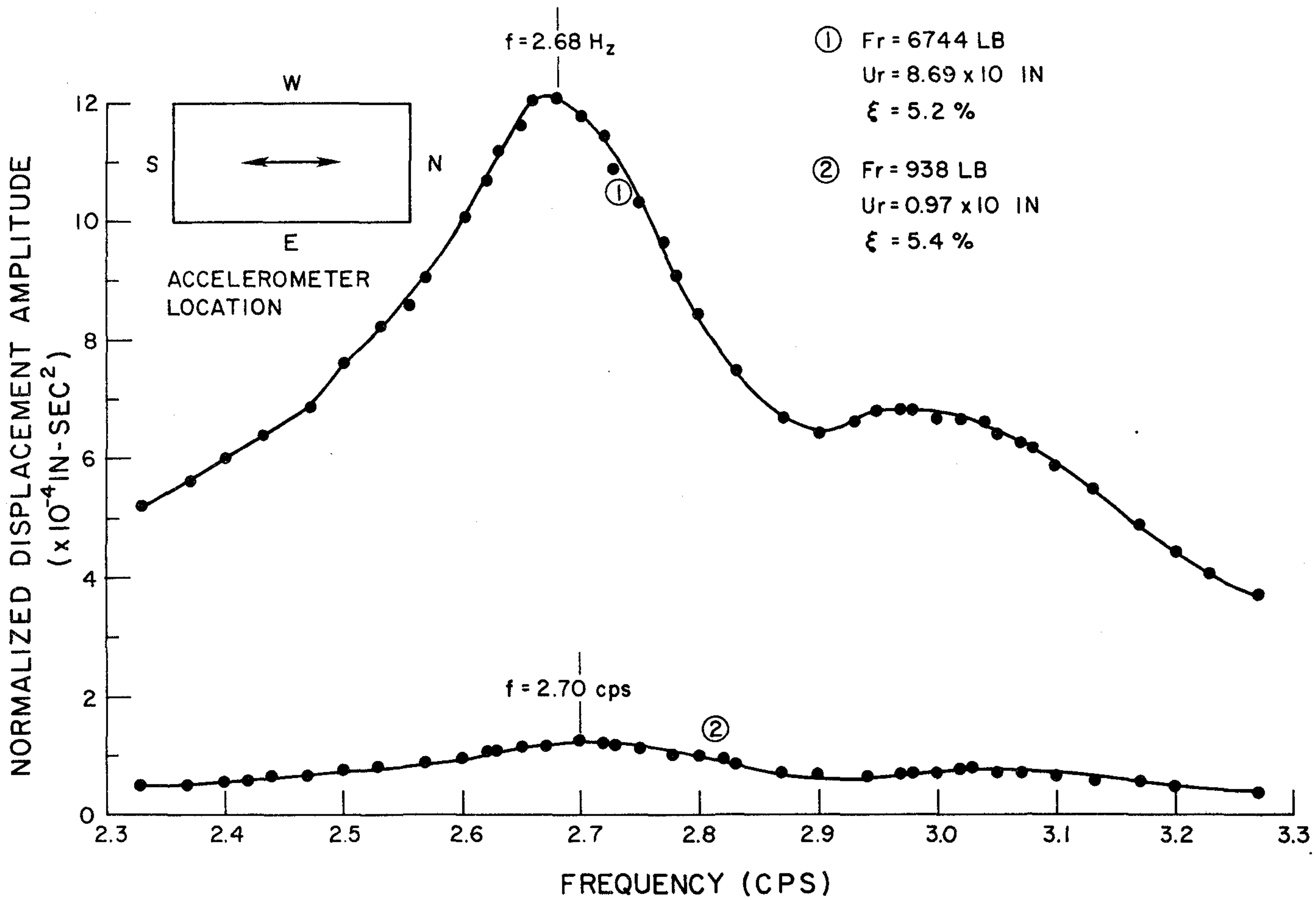


Fig. 3.5 Frequency Response for NS Forcing

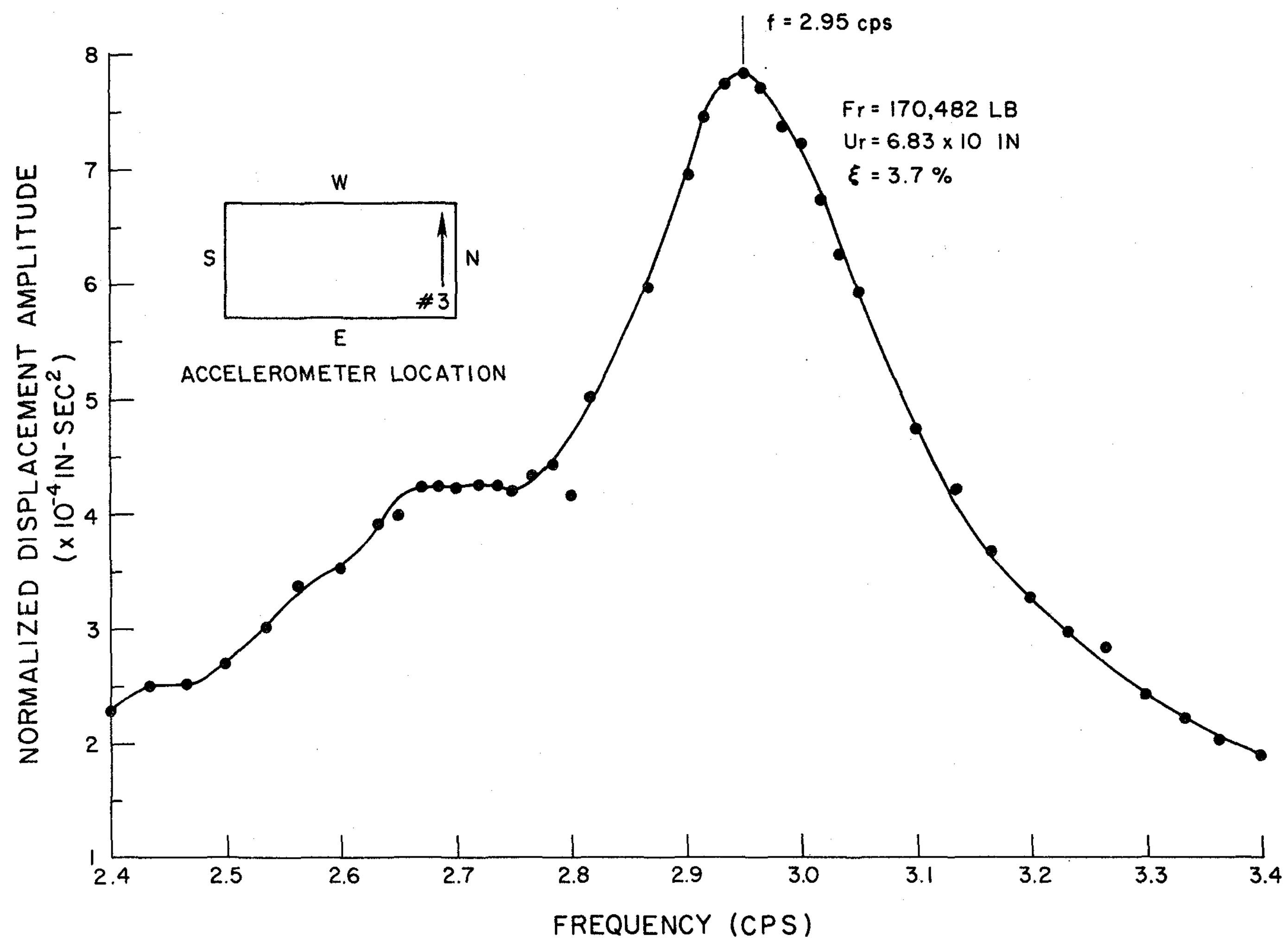


Fig. 3.6 Frequency Response for Torsional Forcing

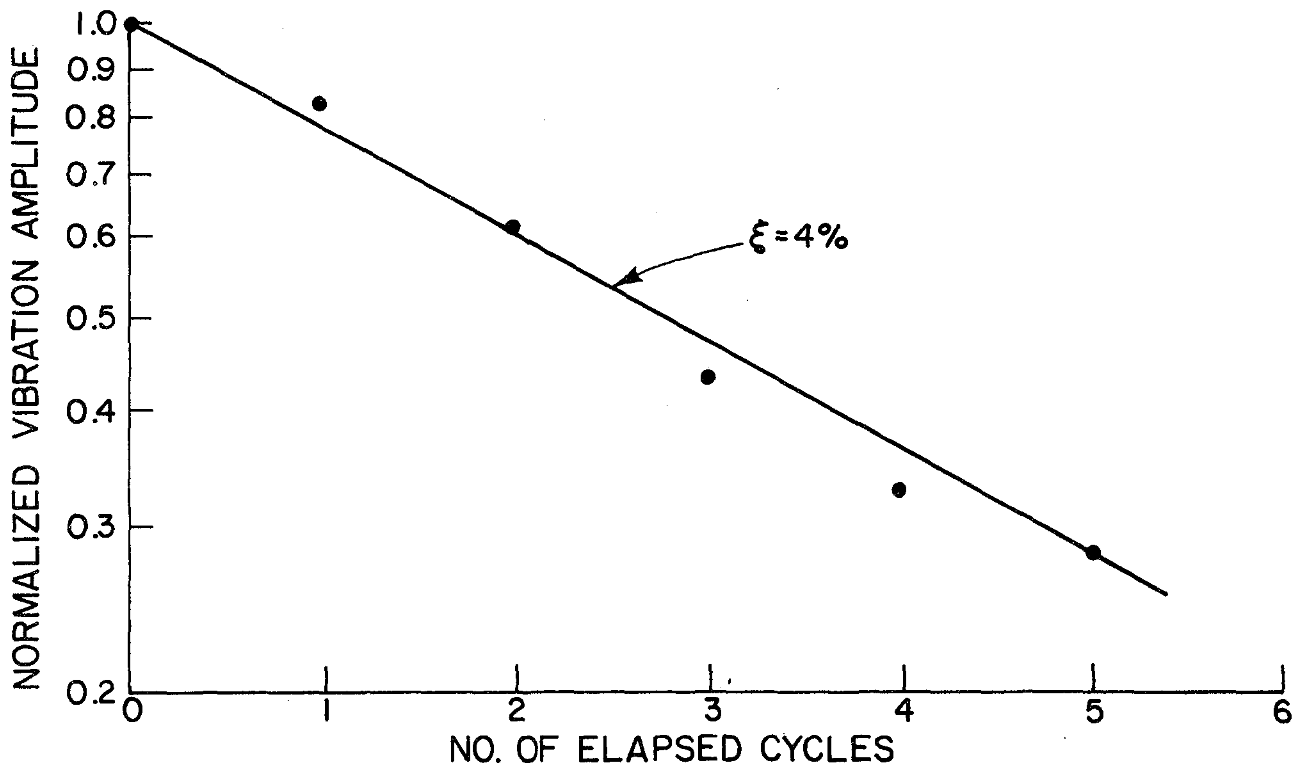


Fig. 3.7 Logarithmic Decay Curve: 1st Mode NS (2.68 cps)

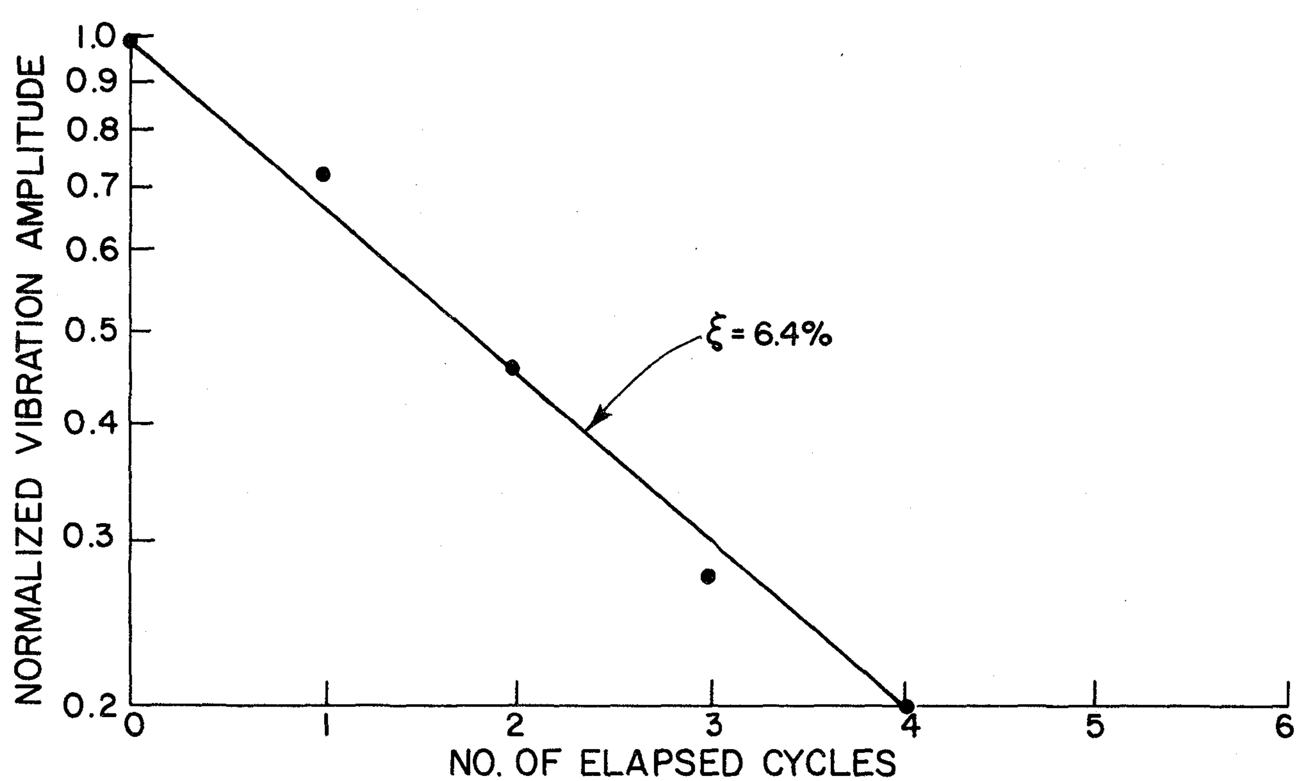


Fig. 3.8 Logarithmic Decay Curve: 1st Mode Torsion (2.95 cps)

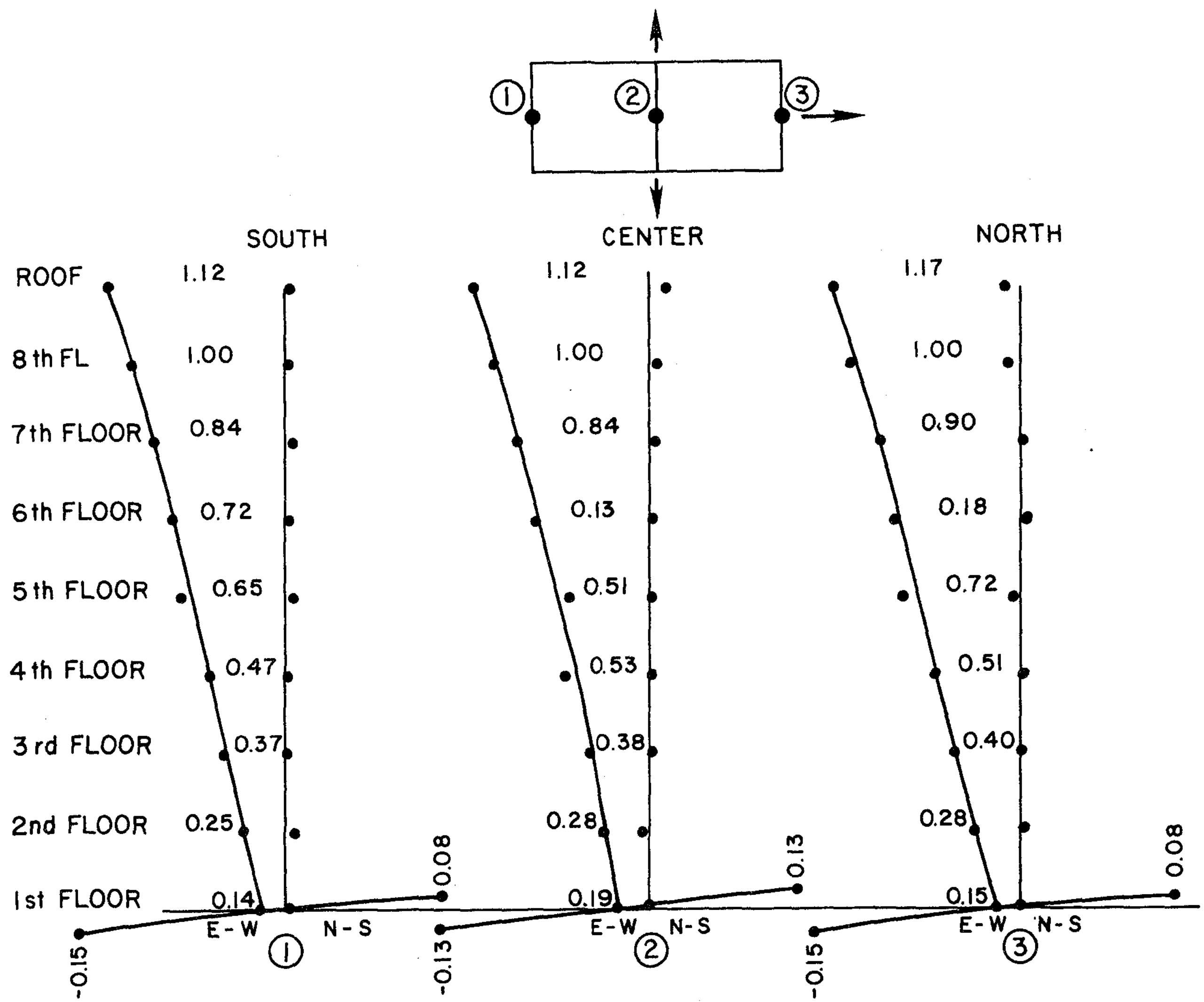


Fig. 3.9 Fundamental EW Mode Shape (f=3.23 cps)

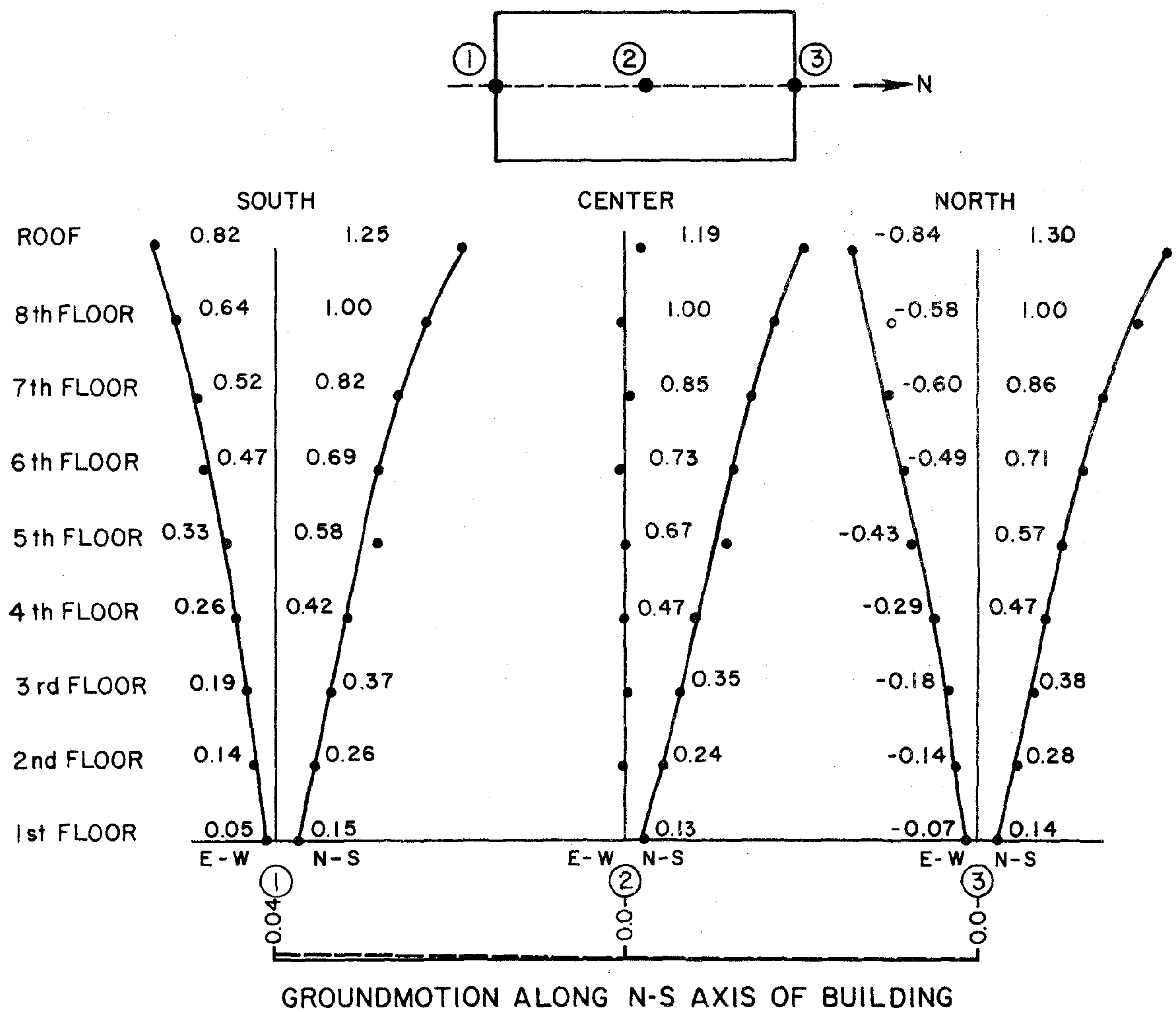


Fig. 3.10 Fundamental NS Mode Shape (f=2.68 cps)

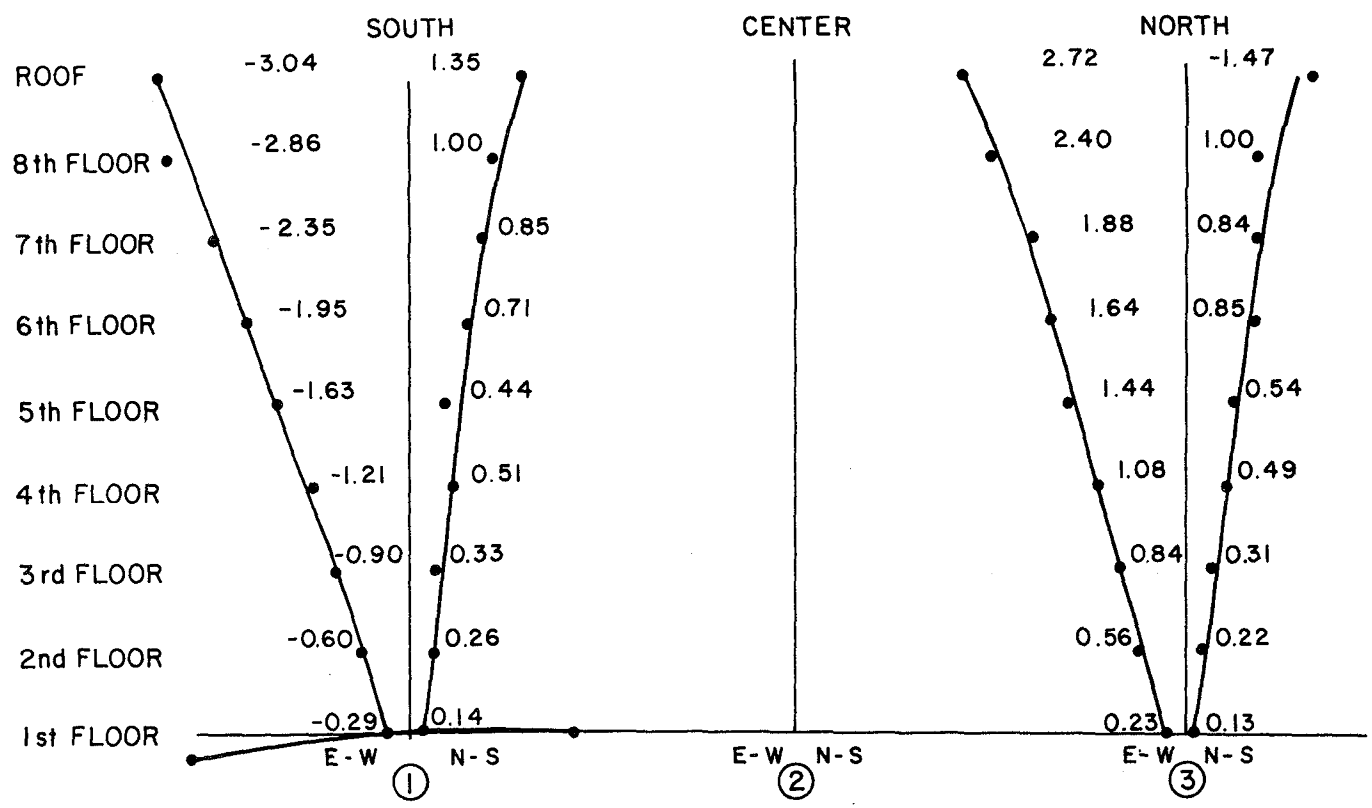
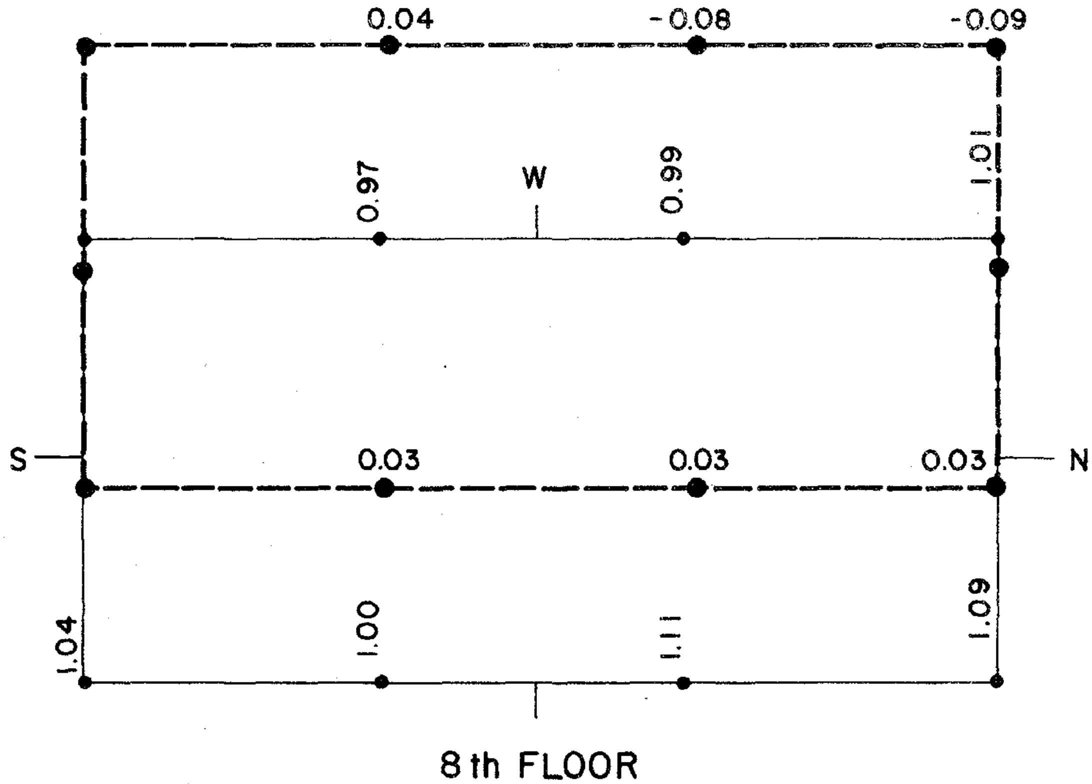


Fig. 3.11 Fundamental Torsional Mode Shape (f=2.95 cps)



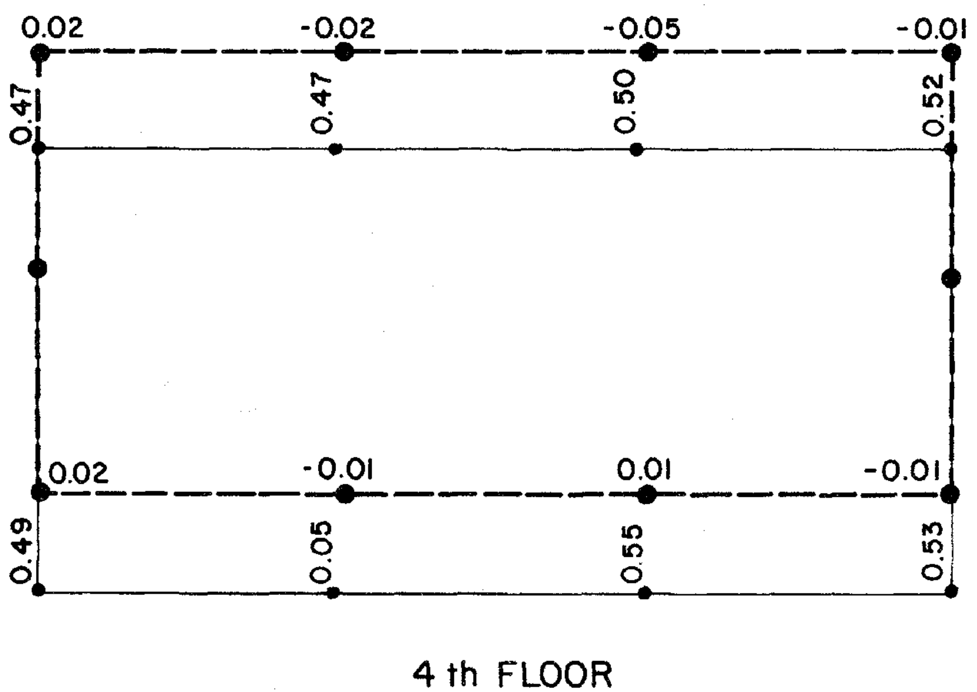


Fig. 3.12 EW Horizontal Floor Mode Shapes

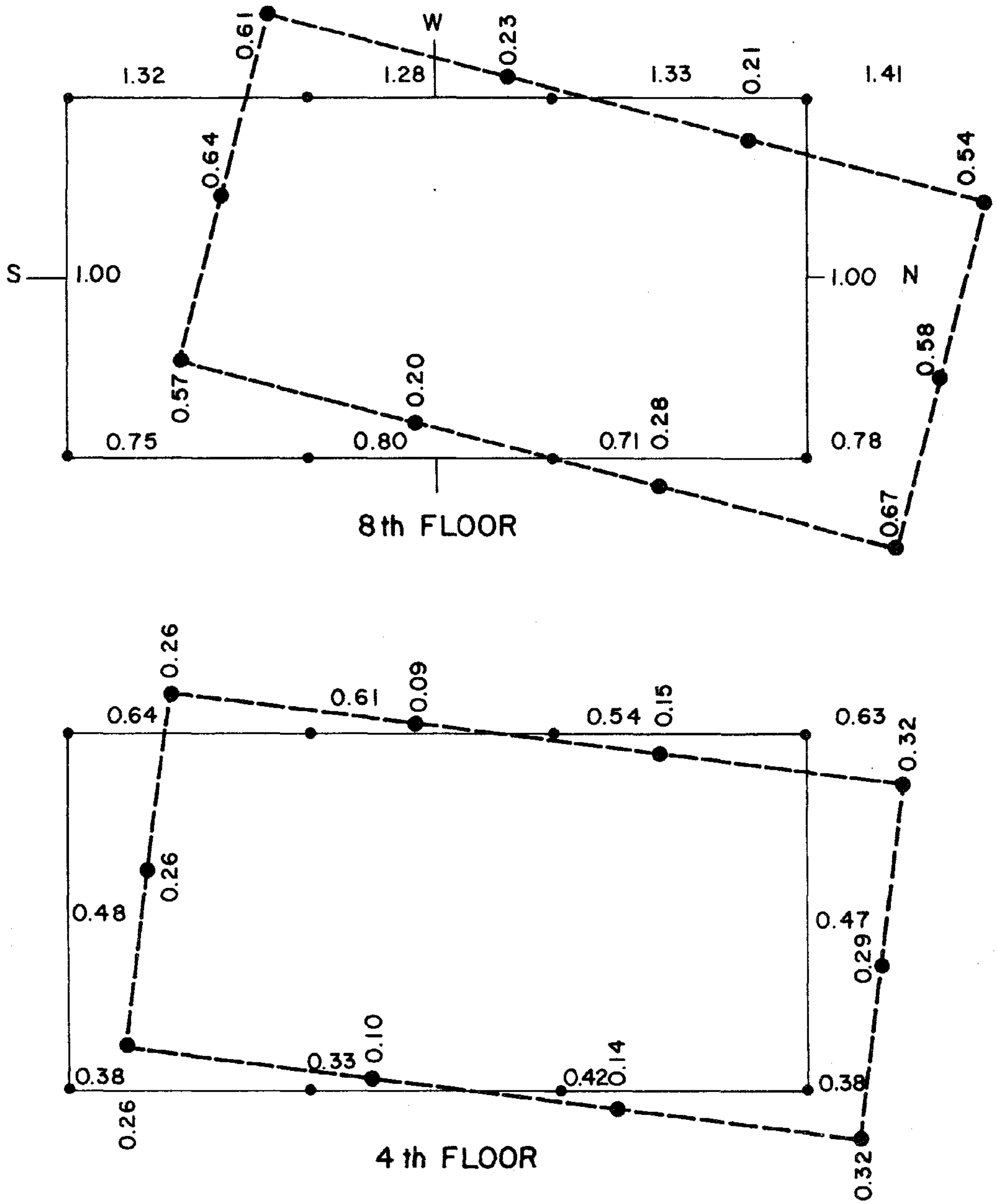


Fig. 3.13 NS Horizontal Floor Mode Shapes

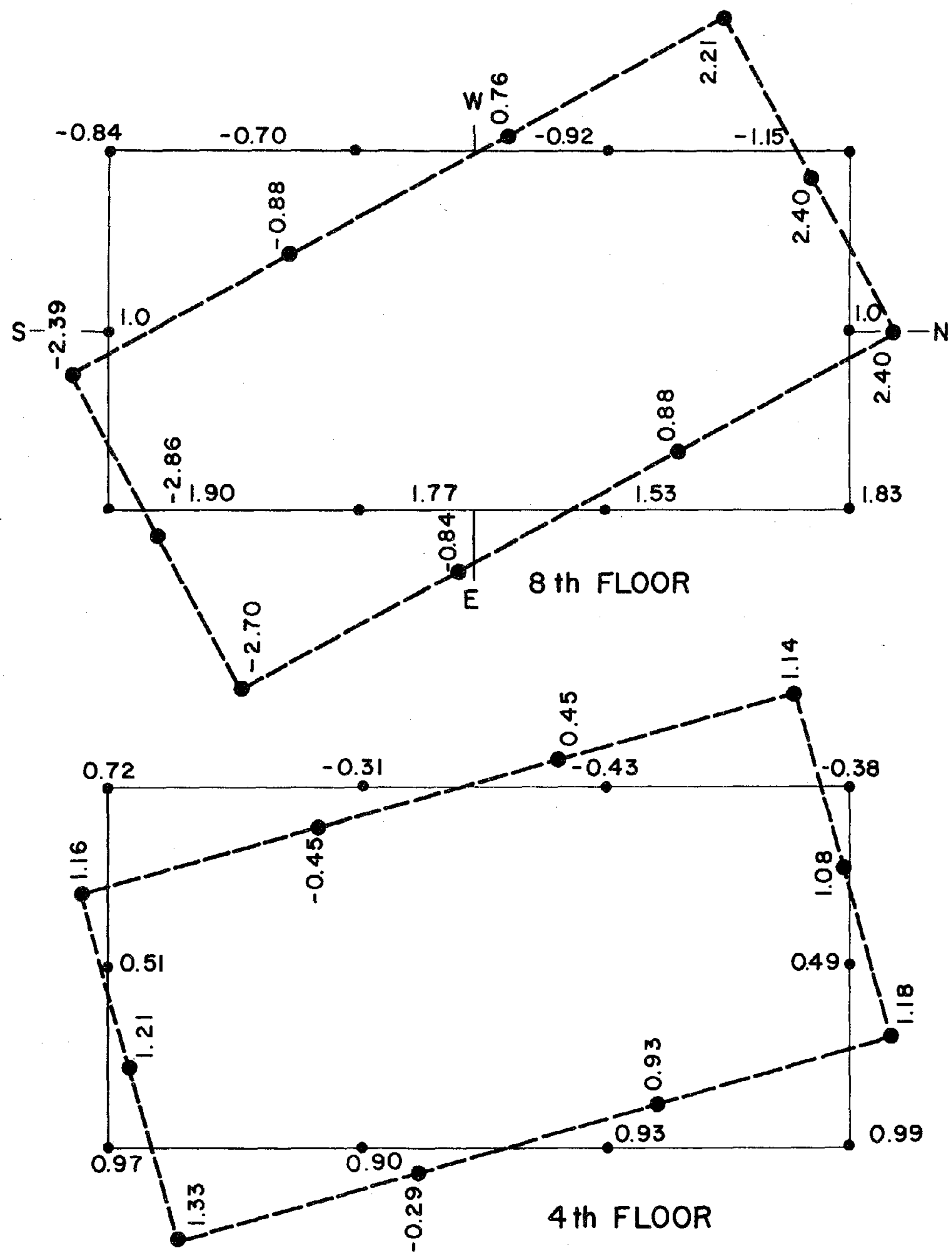


Fig. 3.14 Torsional Horizontal Floor Mode Shapes

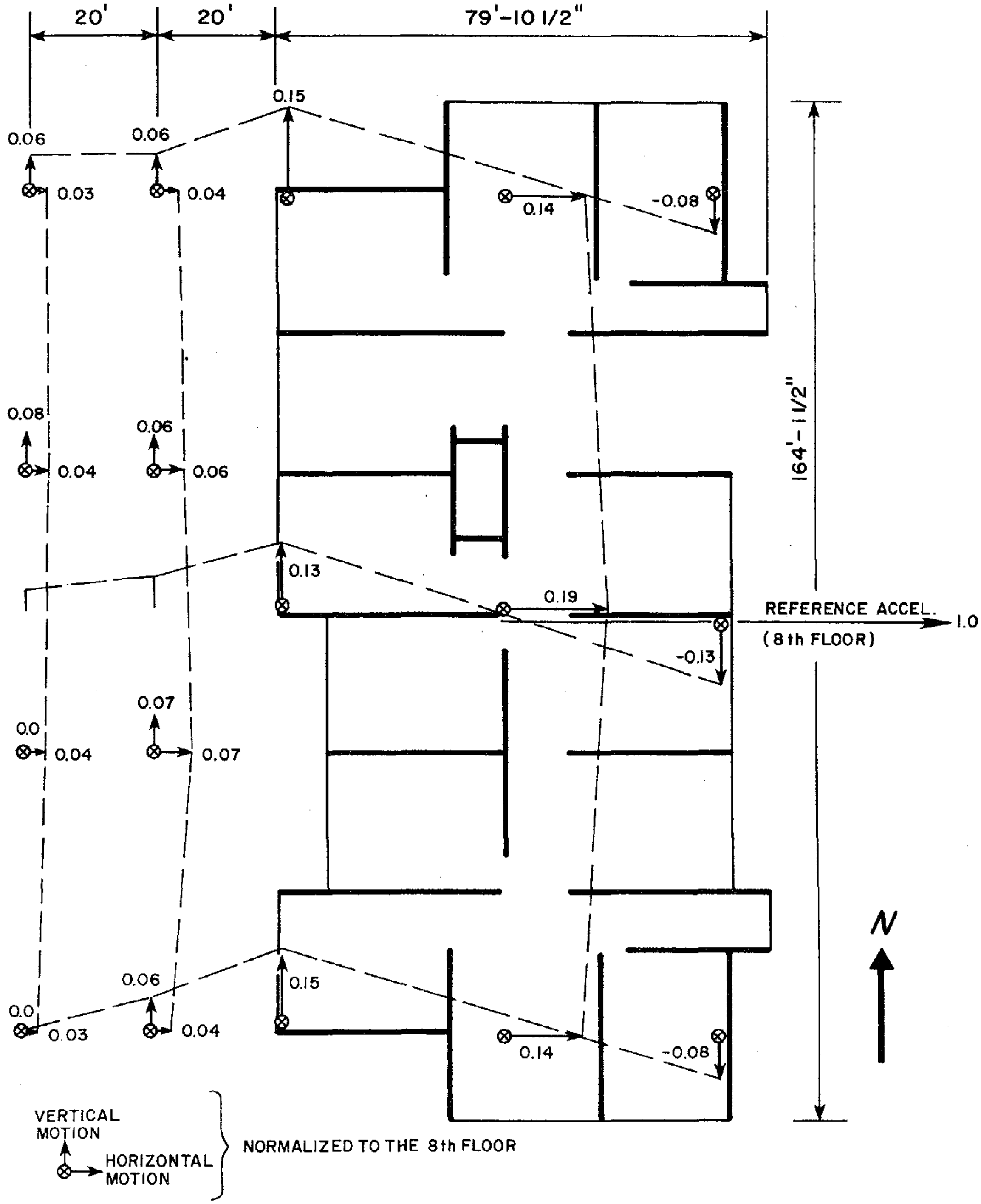


Fig. 3.15 Vertical and Horizontal Ground Motion for EW Forcing (f=3.23 cps)

4. AMBIENT VIBRATION STUDY

4.1 General

Ambient vibration studies use field measurements of wind and microtremor induced vibrations. The method has been in use for 45 years by the United States Coast and Geodetic Survey (6) to measure fundamental periods of buildings. At present it is commonly used to identify higher resonance frequencies and mode shapes (7, 8, 9, 10, 11, 12).

The ambient vibration study of the dynamic properties of the structures is a fast and relatively simple method of field measreuments. It does not interfere with normal building functions, and the measuring instruments and equipment can be installed and operated by a small crew.

The objective of performing the ambient vibration study was to obtain dynamic properties of the building and then compare these results with those obtained from the forced vibration study to assess efficiency of both techniques.

The ambient vibration, experimental and analytical procedures were first suggested by Crawford and Ward (7, 12). An assumption in the analysis technique is that the exciting forces are a stationary random process possessing reasonably flat frequency spectrum. For multistory buildings and other large above ground structures, the largest ambient vibrations are produced by wind. If the frequency spectrum of the vibrational exciting forces is reasonably flat, a structure subjected to this input will respond in all its normal modes.

The vibration measuring equipment employed in the ambient vibration-dynamic test is described below. The general experimental procedures and procedures for data analyses applied are also described. Finally, the experimental results are presented and discussed.

4.2 Field Measurements

4.2.1 Measuring Equipment

The wind induced vibrations were measured using Kinemetrics Ranger Seismometers, Model SS-1. The seismometer has a strong, permanent magnet as the seismic inertial mass moving within a stationary coil attached to the seismometer case. Small rod magnets at the periphery of the coil produce a reversed field which provides a destabilizing forces to extend the natural period of the mass and its suspension.

The resulting seismometer frequency was 1 Hz. Damping was set to 0.7 critical. The output for a given velocity is a constant voltage at all frequencies greater than 1 Hz and falls off at 12 dB/octave for frequencies less than 1 Hz.

The Kinemetrics Signal Conditioner, Model SC-1 (Fig. 4.1) was used to amplify and control simultaneously four seismometer signals. The four input channels have isolated circuitry to integrate and differentiate the amplified input signal . All outputs are simultaneously or independently available for recording. A modification to the signal conditioner allows for outputing each channel separately or for taking the sum or difference on two channels and outputing the average of those channels. Each channel provides a nominal maximum gain of 100,000. An 18 dB/octave low pass filter is available with a cut-off frequency continuously selectable between 1 Hz and 100 Hz for each channel.

The amplified analog signals were recorded and directly converted to digital format using the Kinemetrics Digital Data System, Model DDS-1103. A direct recording oscillograph was provided to display and monitor the four signal levels during tape recordings. The data was digitized at 40 samples per second. The DDS-1103's rate of scan across multiple input channels is 40,000 Hz. This rapid scan rate is sufficient to retain the phase relationship between channels.

A Rockland FFT 512/S Real-Time Spectrum Analyzer was used in order to facilitate the rapid determination of the modal frequencies (Fig. 4.1). This unit is a single channel analyzer with 512 spectral lines calculated but only 400 lines displayed to reduce aliasing errors. Twelve analysis ranges are provided from 0-2 Hz to 0-10 KHz.

4.2.2 Measurement Procedures

When measuring ambient and forced vibrations of the buildings, it is usually assumed that the structure can be approximated by a one-dimensional, damped discrete or continuous system. In most of the cases (10,11, 13), measurements indicate that for the level of excitation applied, floor structures are sufficiently stiff so that the above assumption is acceptable.

In the experimental study of building vibration which is based on the linear model, it is assumed that the resulting motions can be expressed as the superposition of modes associated with the discrete frequencies (14, 15). This approach then requires a simultaneous measurement of motion in a given direction at at least two different floors to obtain their relative amplitude and phase, the two quantities needed to determine mode shapes. During the measurements of wind induced vibrations, it is not necessary to find the actual amplitudes that are recorded because all that is ever used in determining mode shapes is the relative amplitude of the same two instruments.

The modal frequencies were obtained by placing seismometers near the outer walls on the north and south and east and west sides of the 8th floor of the building (see Fig. 4.2). They were oriented so that the signals from the meters on the north and south sides could be used to determine the east-west frequencies. Similarly, the signals from those on the east and west sides were used in determining the north-south frequencies.

The signal conditioner was set so that seismometers 1 and 2 would be output as channel 1, giving the average of the sum of these two readings, and channel 2, the average of the difference of seismometers 1 and 2. The output of seismometers 3 and 4 were similarly averaged. In this way, the translational frequencies could be obtained from the average of the sum of the seismometer readings and the torsional frequencies from the average of the difference of the seismometer readings. Typically, the data was recorded for a total of 300 seconds.

For determining the translational and torsional modes, one pair of seismometers always remained at the 8th floor, placed near the outer walls along either one of the building centerlines (see Fig. 4.3). The second pair of seismometers was oriented in the same way and located successfully on each floor to allow the evaluation of the modal response over the height of the building (Fig. 4.4). As before, the sum of the two seismometer signals at each floor was averaged to give translational modal data. The ratio of the two pairs of averaged readings provided a modal data point normalized to the 8th floor. Torsional modal information was obtained in a similar manner, except that the difference of the seismometer signale at each floor level was used. On each channel the low pass filter was set at 10 Hz to attenuate all higher frequencies, thus completely removing electrical noise and other possible high frequency vibrations. The voltage output to the recorder was adjusted to not exceed about $\pm$ 1.5 volts. The unattenuated calibration constant for the seismometers used was approximately 4.32 volts/in/sec. Corresponding first mode acceleration and displacement were about $\pm 1.0 \times 10^{-5}$ g and $\pm 1.1 \times 10^{-5}$ inches, respectively.

4.3 Data Analysis

4.3.1 Fourier Analysis

It is convenient to use Fourier transforms to analyze low level structural vibrations (16) and exhibit the frequency content of the recorded vibration, thus identifying modal frequencies when the input force frequency spectrum is reasonably flat. Comparing measured amplitude and phase between various points on the structure provides an estimate of the mode shape.

4.3.2 Data Processing

Four simultaneous outputs were recorded on magnetic tape during each run. All runs were digitized at a sample role of 40 discrete points per second. Because of the high frequency filtering present in the field instrumentation, no significant frequencies above 10 Hz were found in the recordings. For the reasonant frequency runs, 4096 data points were selected for the transitional and torsional modes. A total of 10 transforms separated by 890 points were calculated and averaged over the 12107 data points gathered.

For each mode shape run, 1024 data points were selected and a total of 10 transforms were taken. The Fourier amplitude spectrum was an average of the 10 transforms computed.

The spectral estimates were smoothed by 1/4, 1/2, 1/4 weights. The 1024 spectral estimates are uniformaly distributed between 0 Hz and 40 Hz, giving a frequency resolution of 40/1024, or about 0.0391 Hz.

4.3.3 Frequencies and Modes of Vibrations

The natural frequencies of the excited modes are given in Table 4.1. Mode shapes were calculated for both the translational EW abd NS modes as well as the torsional mode. These results, together with modal data resulting from the forced vibration studies, are presented in Fig. 4.4.

TABLE 4.1 RESONANT FREQUENCIES

Excitation	Frequency (cps)
EW	3.28
NS	2.73
Torsional	3.00

4.3.4 Damping

Under forced vibrations, damping in the structure can be determined by the bandwidth method or by measuring the free vibration decay response. During the ambient vibrations the first method can only be used when wind excitations are random and stationary in time (12). Despite gusty wind conditions at the time of the tests, ambient vibration data were used to estimate the viscous damping coefficients using the bandwidth method.

TABLE 4.2 DAMPING RATIOS

Excitation	Damping Ratios
EW	1.5 %
NS	2.0 %
Torsional	< 1 %

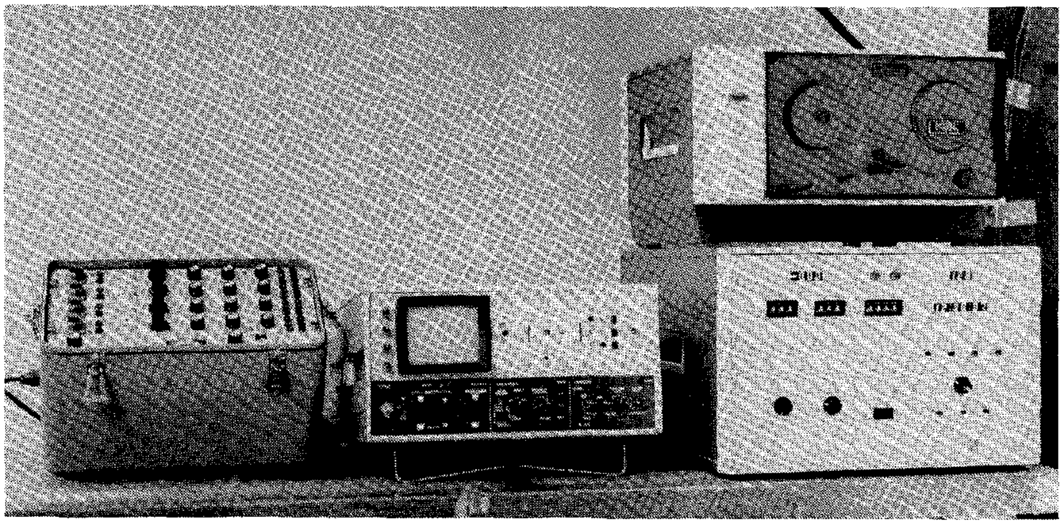


Fig. 4.1 Ambient Vibration Equipment

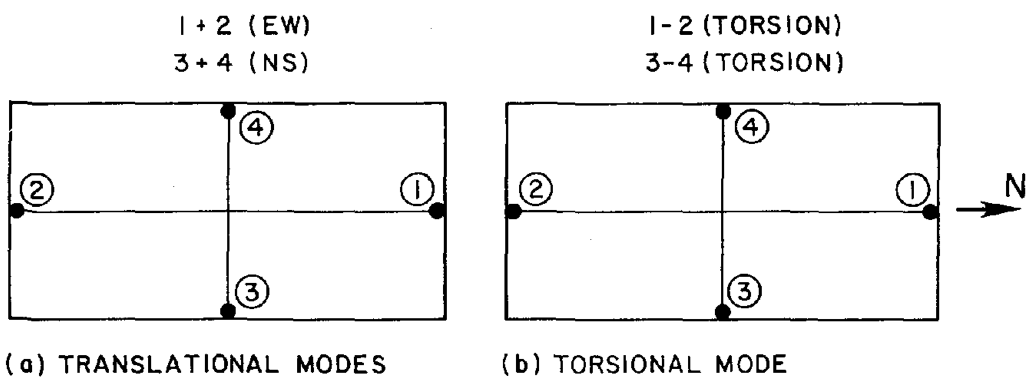


Fig. 4.2 Location of Ranger Seismometers on the 8th Floor for Resonant Frequency Response

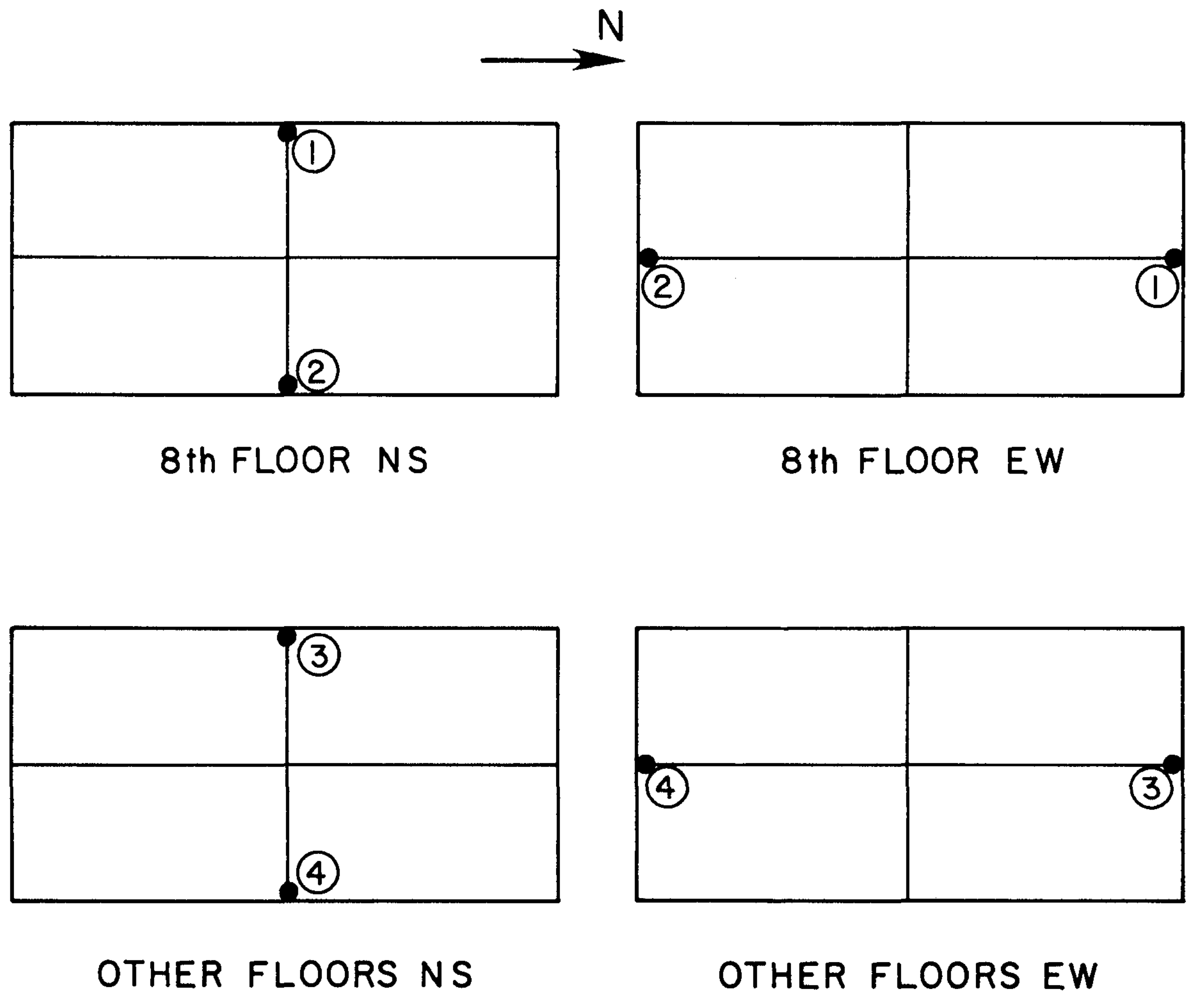


Fig. 4.3 Location of Ranger Seismometers for the Mode Shapes

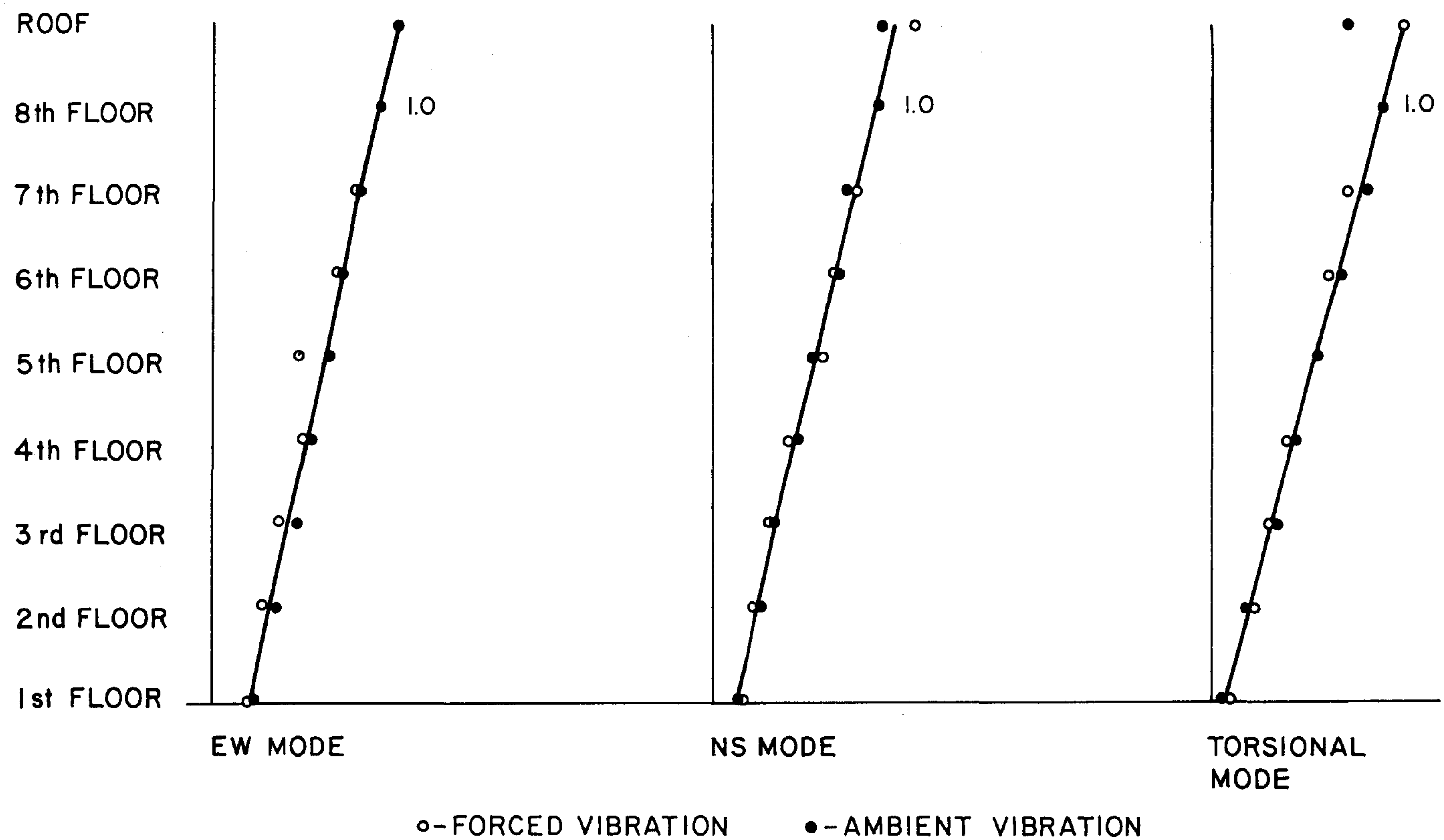


Fig. 4.4 Vertical Mode Shapes

5. COMPARISON OF FORCED AND AMBIENT VIBRATION STUDIES

The dynamic properties (resonant frequencies, modes of vibration and damping values) were determined by full-scale dynamic tests using both forced and ambient vibration methods. Resonant frequencies and damping factors from both studies are summarized and compared in Table 5.1.

The resonant frequencies from the forced vibration tests are 1% to 2% smaller than those from the ambient vibration tests. This nonlinear aspect may be due to a stiffness deterioration resulting from larger excitations under forced vibrations.

Equivalent viscuous damping factors show some difference. It appears that it is rather difficult to obtain appropriate damping values under ambient vibrations. However, considering the different displacement amplitudes for the two vibration methods, the smaller damping values under ambient conditions are not unrealistic. However, in general, the results can be more appropriately viewed as an indication of the range of damping, rather than as specific damping ratios associated with each mode of vibration.

Vertical mode shapes associated with both the translational and torsional resonance frequencies are compared in Fig. 4.4 and show very good agreement.

Because of a total lack of vibration generation requirements and the ease of equipment handling, the total field effort for ambient vibration studies is significantly smaller than for forced vibration studies. Also, because accurate frequency response data can not be generated under ambient conditions, fewer measurements are required. Furthermore, each measurement requires less time. On the other hand, data analysis is slightly more complicated because of computer use for Fourier analyses.

TABLE 5.1 COMPARISON OF RESONANT FREQUENCIES AND DAMPING FACTORS

MODE	FORCED VIBRATION			AMBIENT VIBRATION	
	FREQUENCY (cps)	DAMPING FACTORS FROM		FREQUENCY (cps)	DAMPING FACTOR
		DECAY CURVE	FREQ. RESP. CURVE		
EW	3.23	-	4.7 %	3.28	1.5 %
NS	2.68	4.0 %	5.0 %	2.73	2.0 %
Torsional	2.95	6.4 %	3.7 %	3.00	< 1 %

6. FORMULATION OF MATHEMATICAL MODEL

6.1 General

A mathematical computer model of the Los Portales Building was formulated to assess its dynamic characteristics. The model was formulated using both a rigid base and a flexible base. The computer program employed in the dynamic analysis of the model and the models themselves are described below.

6.2 Computer Program

TABS-77, a general computer program developed by the Division of Structural Engineering and Structural Mechanics of the Department of Civil Engineering at the University of California, Berkeley, was used to calculate the frequencies and mode shapes of the building. A complete description of this program is given in reference (17).

The dynamic analyses in this investigation were performed on a CDC 6400 digital computer using the standard TABS-77 program. The program considers the floors rigid in their own plane and to have zero transverse stiffness. All elements are assembled initially into planar frames and then transformed, using the previous assumption, to three degrees of freedom at the center of mass for each story level (2 translational, 1 rotational).

6.3 Modelling of the Structure

The basic model of the building was formulated as a system of frames and shear wall elements interconnected by floor diaphragms which were rigid in their own plane and fixed at the 1st floor level.

All walls were treated as "wide columns". This required a reduction of properties (I, A, A_v) to the elastic centroid of each wall. Where a wall is met by a perpendicularly oriented wall, a portion of the latter wall is assumed acting as a flange and thus included in the resonant of inertia calculation. For a "half-flange" condition, where two panels form a single corner, the effective flange width is considered as 1/6 of the overall building height, or 11' - 9". In case of a "full-flange" condition, as shown in Figure 6.1, the effective width is 1/3 of the height, or 23' - 6". The above assumption is based on the fact that the walls are effectively interconnected at each floor level. The resulting dowel action over the height of the building seems to justify the assumed wall coupling, at least under small amplitude vibrations.

Although not entirely correct for flanged shear walls, the shear area (A_v) for all walls were set to 5/6 A, where A is the area of the wall parallel to the direction of motion.

Wherever shear walls were positioned in one line parallel to the direction of motion, it was assumed that those walls would be coupled by a portion of the floor slab, having a width of 18 times the thickness of the floor, or 12'. The effective span of these coupling girders was reduced to the clear distance between the walls; a possible option of the TABS program. In as far as the story deformation occur only over the clear distance between two stories, this effect needs to be captured. This can be achieved by increasing the moment of inertia of each wall by the ratio $(L_0/L_1)^3$, where L_0 is the story height and L_1 is the clear distance between stories. In this case amplification factor was typically $(8.67/8)^3 = 1.27$. Fig. 6.1 illustrates the manner in which the shear walls were idealized for the analysis.

In the analysis the modulus of elasticity for the reinforced concrete was assumed as 4000 ksi. The effective floor mass, considering the floor slab and structural walls only, was estimated at 53 kips. sec^2/ft . The rotational mass, with a radius of gyration of 48.8 ft, was taken as 126,220 kips. sec^2 ft.

The center of stiffness for a typical story was calculated with the assumption that the shear walls are damped on both ends. The center of stiffness and the center of mass were shown in Fig, 3.3.

From initial analyses, considering a rigidly supported structural model, and experimental frequency data,a significant difference in the natural frequencies was noted. In fact the analytical values were 50 to 80% larger than the experimental frequency data. Hence, it was found to be essential to include the foundation stiffness in the overall analytical model.

6.4 Modelling of the Foundation

TABS-77, the computer program used to determine the dynamic properties of the structure, does not permit the input of the rotational, lateral and vertical springs at the foundation level. This makes it necessary to model a so called "dummy story" below the foundation level to account for the soil stiffness. This can be achieved by determining for both the NS and EW directions dummy stories which properly reflect the translational and rotational foundation stiffnesses for each direction. The solution of each of the two 2-degree of freedom systems (see Fig. 6.2) follows from the force-displacement relationship:

$$\begin{bmatrix} M_o \\ V_o \end{bmatrix} = EI \begin{bmatrix} \frac{4}{L} & -\frac{6}{L^2} \\ -\frac{6}{L^2} & \frac{12}{L^3} \end{bmatrix} \begin{bmatrix} r_m \\ r_v \end{bmatrix} \quad , \text{ where}$$

M_o = overturning moment at the base,

V_o = base shear,

r_m = base rotations,

r_v = base displacement,

EI = flexural rigidity of dummy story, and

L = height of dummy story.

With estimated constant masses of 53 kpis. sec^2/ft for every story and the measured floor accelerations for the two fundamental transitional modes, it is possible to calculate the base shear and the overturning moment using the dynamic forces at the resonance frequency (Fig. 6.3). Calculating the actual base displacement from the measured acceleration at the base and approximating the base rotation by the secant of the mode shape between the first and second floor, the force-displacement relation can be solved for EI and L. The results for the uncoupled EW and coupled NS/torsional modes are presented in Table 6.1.

TABLE 6.1 DUMMY STORY PROPERTIES

EXCITATION	FREQUENCY (cps)	LENGTH (ft)	EI (kips. ft^2)
EW	3.23	35.6	3.292×10^{10}
NS	2.68	21.6	0.962×10^{10}

As the dummy story height for both NS and EW directions has to be the same it is essential to develop an optimum dummy story element. Hence, the structure was analyzed with several different dummy stories with lengths varying from 20 to 35 feet. The dynamic properties of the structural system using a 20 feet high dummy story were found to be in very close agreement with the experimentally observed results. The lateral and rotational stiffness of the dummy story as computed from experimental data and actually used in the analytical model are compared in Table 6.2. There is reasonable agreement between experimental and analytical stiffnesses, with the exception of the transverse rotational stiffness, which according to the experimental data should have been much higher. However, using the higher experimental stiffness in the analytical model resulted in a significantly larger EW frequency than observed experimentally.

TABLE 6.2 LATERAL AND ROTATIONAL SOIL STIFFNESS

	h (ft)	EW KLAT = $\frac{KEI}{L3}$ (kips/ft)	EW KROT = $\frac{L_1EI}{L}$ (kips ft/rod)	NS KLAT = $\frac{12EI}{L^3}$ (kips/ft)	NS KROT = $\frac{L_1EI}{\ell}$ (kips ft/rod)
EXPERIMENT	--	8.7×10^6	3.7×10^9	11.5×10^6	1.8×10^9
ANALYTICAL MODEL	20	6.1×10^6	0.8×10^9	11.2×10^6	1.5×10^9

6.5 Analytical Results

The modal results from the TABS program list two translational and one rotational component. The notation as to the direction of the modal frequencies is governed by the predominate component, i.e., an EW mode is a general 3-dimensional mode in which the mode shape is governed by the EW component.

The frequencies for the rigid base model, as well as for the flexible base model, are compared with the experimental results in Table 6.3. The vertical modes and floor modes, as obtained from the forced vibrations tests and computer analyses with a flexible base, are shown in Figs. 6.4 through 6.6. In these figures the experimental floor modes have been reduced to 3 degrees of freedom per floor. Very good agreement between the experimental resonant frequencies and mode shapes and those for the model with flexible base can be noted.

TABLE 6.3 RESONANT FREQUENCIES (cps)

EXCITATION	EXPERIMENT: FORCED VIBRATION	EXPERIMENT: AMBIENT VIBRATION	ANALYSIS: RIGID BASE	ANALYSIS: FLEXIBLE BASE	CODE
EW	3.23	3.28	5.99	3.30	2.53
NS	2.68	2.73	4.03	2.67	3.63
Torsional	2.98	3.00	5.29	3.09	--

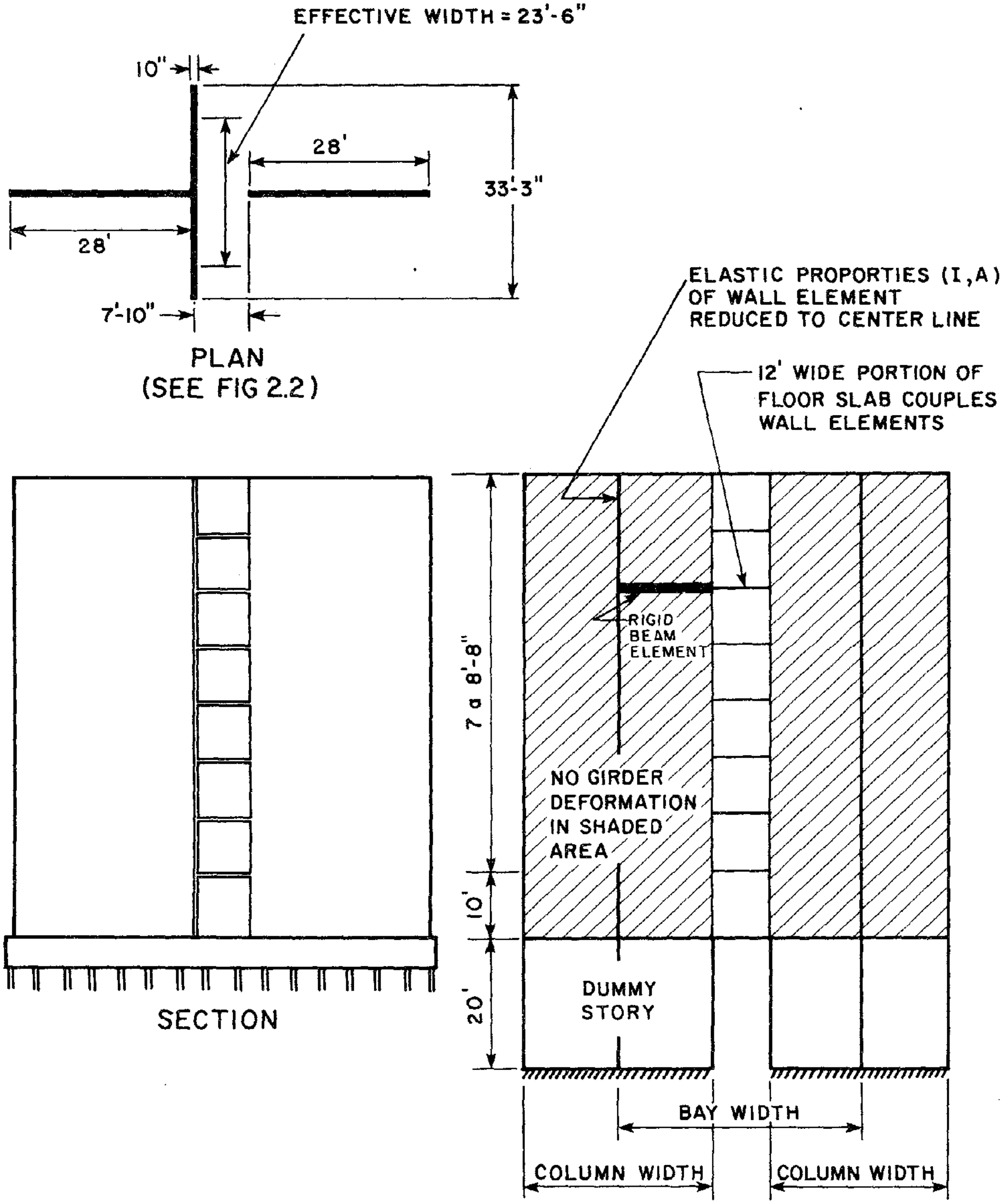


Fig. 6.1 Typical Wall Element Formulations

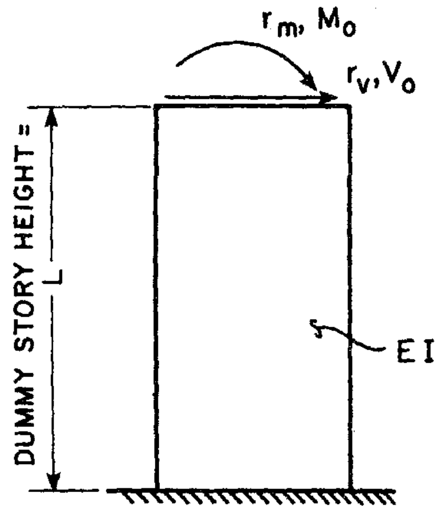


Fig. 6.2 2 Degree of Freedom Model for the Dummy Story

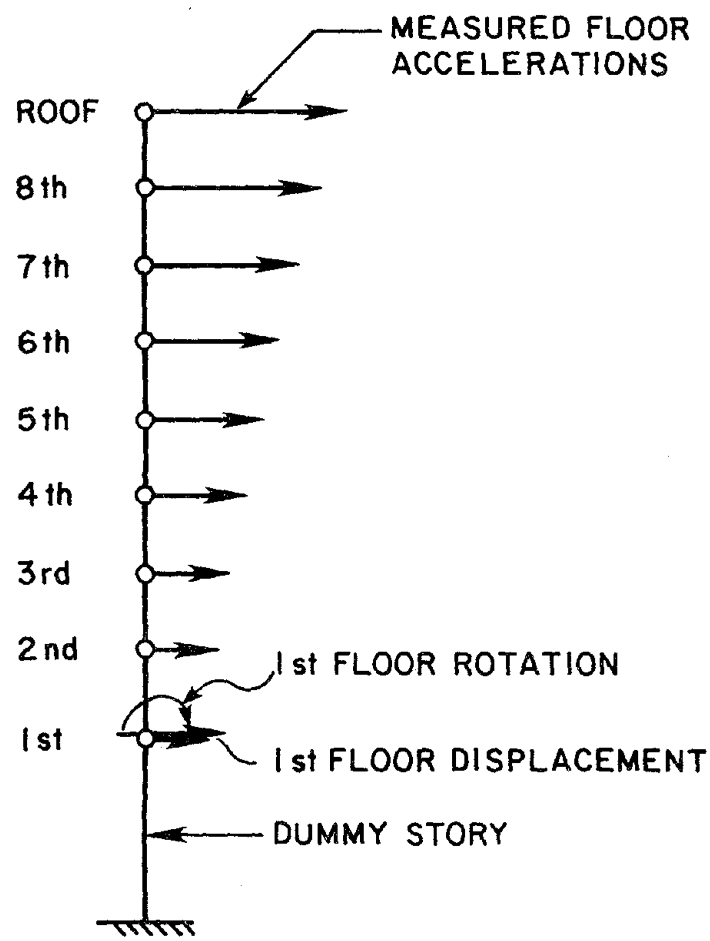


Fig. 6.3 Available Experimental Data

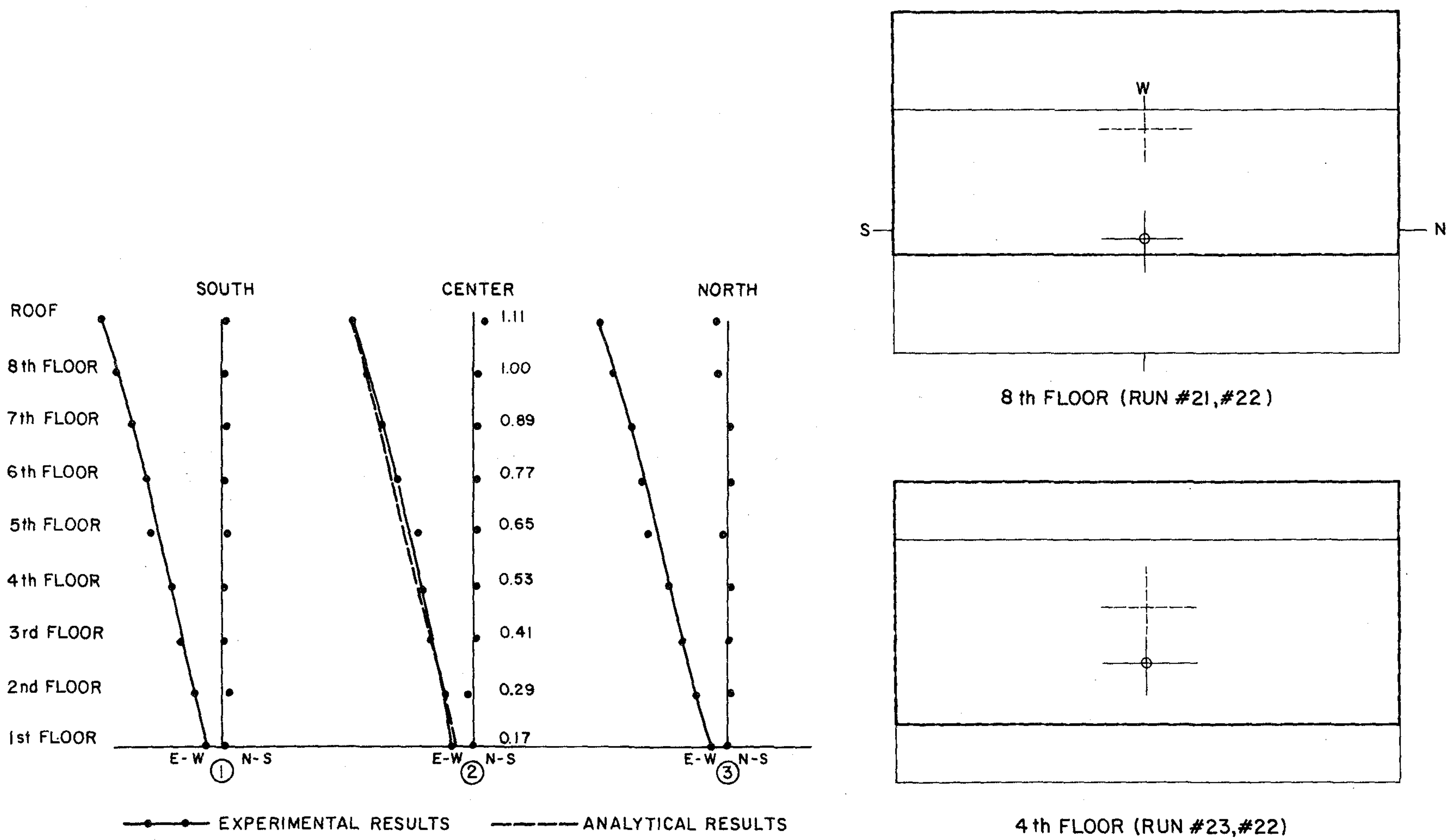


Fig. 6.4 EW Mode Shape

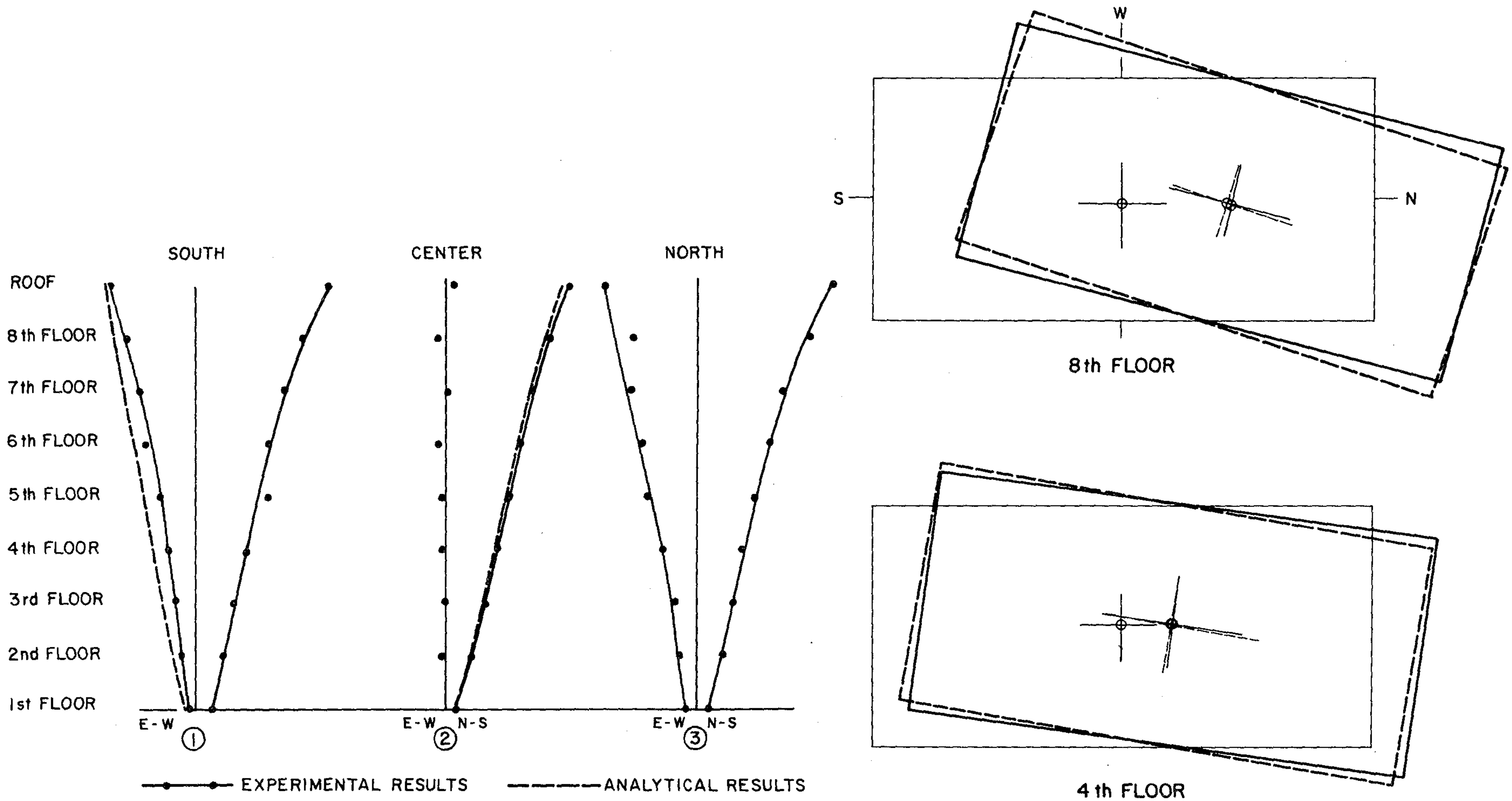


Fig. 6.5 NS Mode Shape

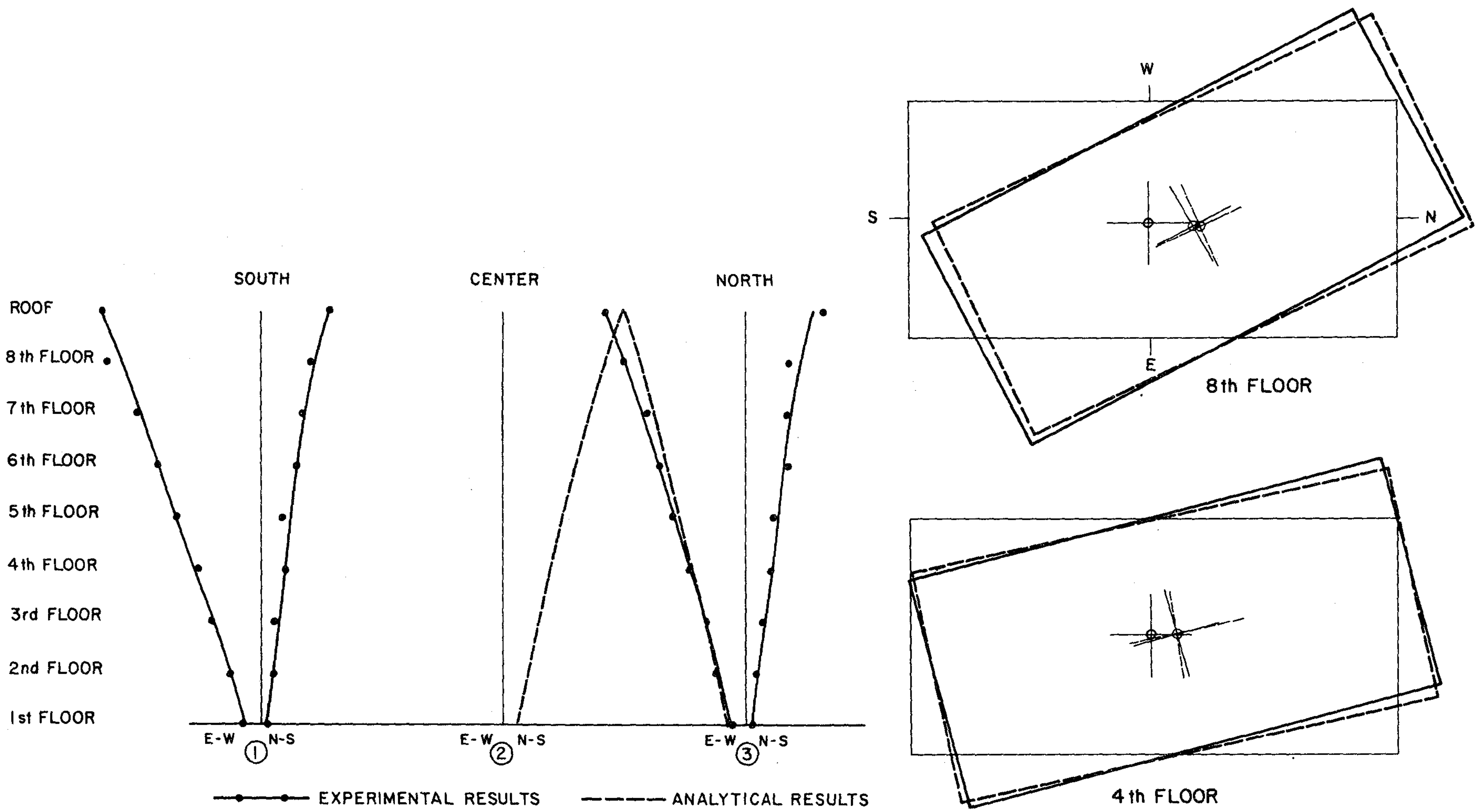


Fig. 6.6 Torsional Mode Shape

7. CONCLUSIONS

The results presented herewith clearly show that forced and ambient vibration studies can be carried out effectively. In comparing experimental and analytical solutions, good agreement can be noted for frequencies and mode shapes. Considering a frequency range up to 10 Hz, only the three fundamental modes of vibration could be identified, thus indicating that the building would basically respond to seismic excitation in a first mode motion. The dynamic tests indicate a high coupling between NS and torsional modes. This highly coupled response could possibly be reduced by changing the floor plan layout. It may be noted that a similar highly coupled behavior was also observed for the 12 story Wesley Manor Building in Campbell, California of the same system and with almost the same floor plan. The periods of the two structures are plotted versus building height and reveal, as shown in Fig. 7.1, an almost linear increase with height.

The frequencies using UBC code provisions ($T = 0.05\ .h/\sqrt{D}$), and based on the actual building dimensions, are 22% too low in the EW (transverse) direction and 35% too high in the NS (longitudinal) direction when compared with the experimental data (18). These inconsistent results clearly indicate the need for a detailed dynamic analysis, considering the actual wall layout, stiffness distribution, and foundation conditions. Neglecting the foundation flexibility (rigid base model), shows an overestimation of the experimental frequencies by 50% to 80%. Thus, in the analysis of rigid structures on flexible foundations, the soil-structure interaction must be considered.

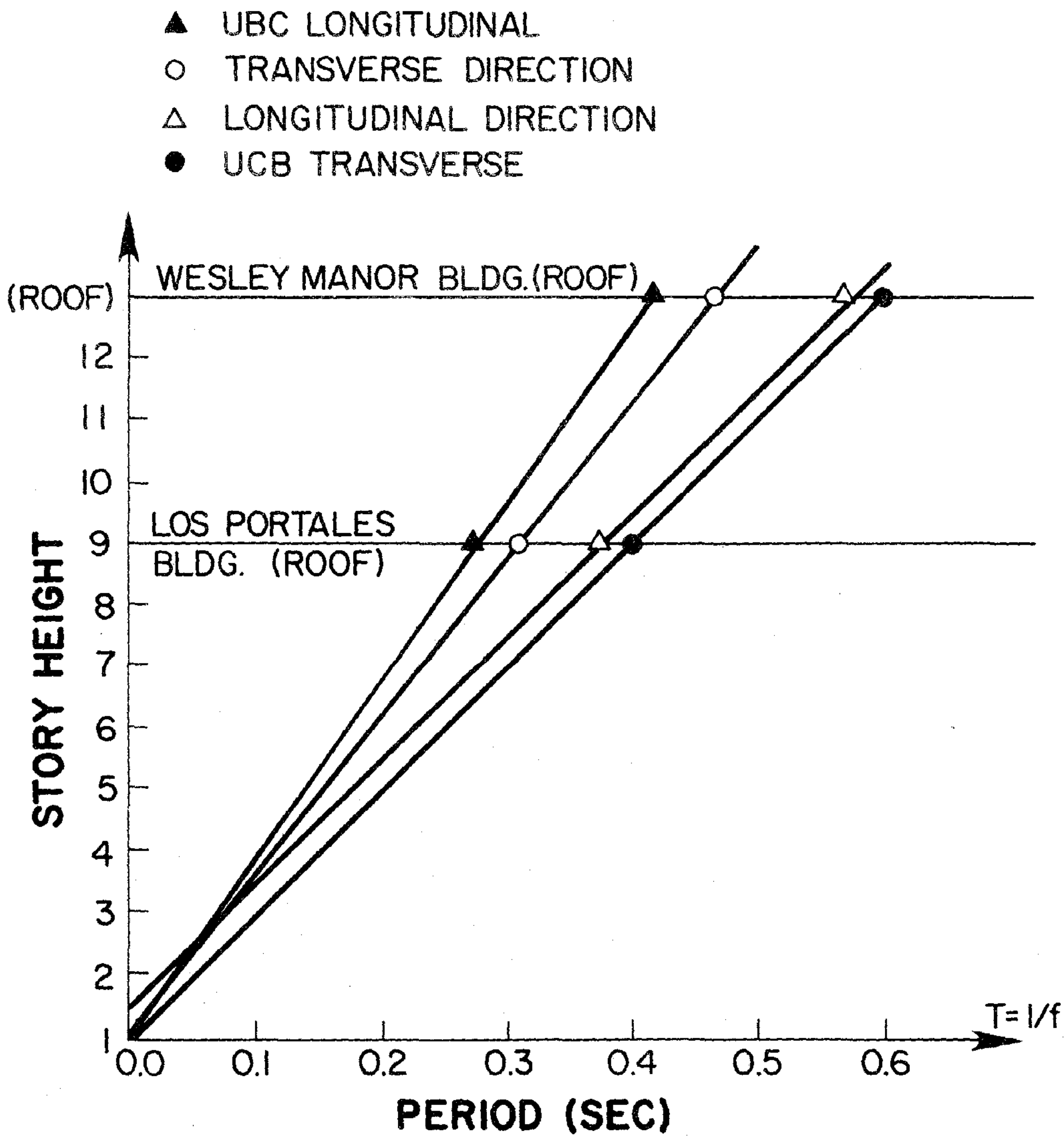


Fig. 7.1 Period Versus Story Height

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