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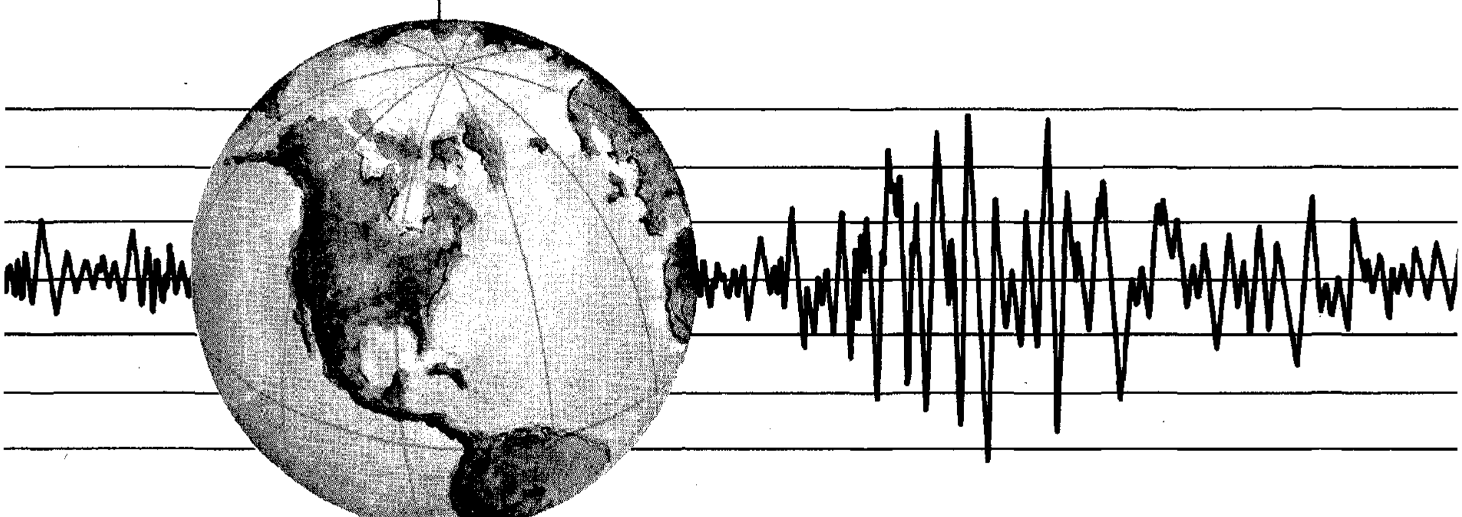
EARTHQUAKE ENGINEERING RESEARCH CENTER

DYNAMIC PROPERTIES OF A TWELVE-STORY PREFABRICATED PANEL BUILDING

by

J.G. BOUWKAMP
J.P. KOLLEGGER
R.M. STEPHEN

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 using experimental vibration data resulted in resonant frequencies and mode shapes showing excellent agreement with the test data. Accounting for the foundation flexibility using actual soil and pile test data did not produce a satisfactory correlation with the dynamic vibration test results.

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ABSTRACT

The results of forced and ambient vibrations studies of a twelve story apartment building, constructed with prefabricated wall panel and slab elements are presented. Dynamic characteristics, such as resonant frequencies, damping, and vertical and horizontal mode shapes of the structure were determined and correlated with analytical results using the computer program TABS-77.

Rigid floor diaphragm action and serious structure-foundation interaction were observed. Including the foundation flexibility in the analytical model using experimental vibration data resulted in resonant frequencies and mode shapes showing excellent agreement with the test data. Accounting for the foundation flexibility using actual soil and pile test data did not produce a satisfactory correlation with the dynamic vibration test results.

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 therefor 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 evaluate the dynamic characteristics of a prefabricated type structure, dynamic tests using both forced and ambient methods were performed on the Wesley Manor Building in Campbell, California, a "Forest City Dillon" type 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 results of both methods and to assess the accuracy of each method in evaluating the dynamic properties of this structure.

The building is described in Chapter 2, and the results of the dynamic tests, from forced, as well as 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 including the foundation, as based on actual soil and pile data, and the analytical dynamic results obtained, are described in Chapter 6. A general comparison and discussion of the experimental and analytical results is presented in Chapter 7.

1.2 Acknowledgement

The authors gratefully acknowledge the financial support provided by the National Science Foundation under Grant NSF PFR 79-08257-2NF. They also wish to thank the owner, Wesley Manor, Inc.; the architects, Ogren, Juarez and Givas, especially Mr Bob Ogren; and the contractors, Campbell Construction Company of Sacramento and Tecon Pacific especially Mr. Jim Clark of Tecon Pacific, for their help and cooperation in coordinating and carrying out the test program.

2. DESCRIPTION OF BUILDING

2.1 General

The Wesley Manor Building in Campbell (Fig. 2.1), a Forest City Dillon reinforced concrete prefab building, was tested in October 1978. This building system uses solid slab elements and cellular wall panels. At the construction, reinforcement is placed in the cells which are subsequently grouted. As the modular design of the building system includes prefabricated kitchen and bathroom units, construction progressed at a rate of about one story every two days.

2.2 Architectural Layout

The overall floor dimensions of the Wesley Manor Building are approximately 164' x 80'. The building has 12 stories for a total height of 105'-4". The building contains only apartments, except for a portion of the first floor where space is reserved for laundry rooms, mechanical rooms, lounge and a reception area. The building is serviced by two elevators, located in the center. In addition, stairwells are located on either end of the building as shown in Fig. 2.2.

2.3 Structural System

The vertical and horizontal load-carrying system consists of reinforced concrete shear walls in both the transverse and the longitudinal directions. These walls have a typical thickness of 8 in. over the entire height of the building. Wall reinforcement placed in the cells varies over the height as shown in Figure 2.3. The minimum concrete strength for the panels and grout is specified as $f_c^1 = 4000$ psi. The overall wall design effectively results in a shear wall system.

The solid floor slab elements are 4 in. thick, with plan dimensions of 8' by 22'. At the site additional reinforcement is placed across the 22 ft. long slab functions (#4 x 4'-0" @ 18" c.c.) and a 4 in. concrete topping is placed over these elements, thus resulting in a total slab thickness of 8 in. Details of exterior and interior wall-to-floor panel joints are shown in Figure 2.4. The prefabricated kitchen and bathroom units have a slab thickness of 8 in. with protruding reinforcing bars tying into the adjacent 4 in. thick topping slab to provide continuity.

The structure is founded on piers with a diameter of 24 in. and varying lengths from 30' to 53'. The piers are located at intervals of approximately 5 feet, as shown in Fig. 2.5. The same figure also shows through solid lines the layout of the first-floor walls.

2.4 Soil Conditions

The following excerpts, describing the soil conditions at the site and Figures 2.6 through 2.8 are taken from the report by LeRoy Crandall and Associates, Consulting Geotechnical Engineers, Los Angeles, CA as provided by the project architect.

> "Evidence of existing fill (debris, etc.) was not encountered in the exploration borings."

> "The natural soil beneath the site consists of sandy silt, underlain by sand, sandy silt, silty clay and clayey silt. The upper natural soils are moderately firm to firm at present moisture content but would become weaker and more compressible when wet. Below depths of 30 to 35 feet, the soils are firm to very firm with layers of moderately firm soils."

> "Borings 1 through 3 (as identified in Fig. 2.6) were drilled using rotary wash-type drilling equipment with drilling mud to prevent caving. The mud was removed following completion of the drilling to permit water level measurement. Observation 16½ hours after removal of the mud in Boring 2 indicated no water in the boring.

Borings 4 and 5 were drilled to a depth of 50 feet using conventional bucket-type drilling equipment and water was not encountered within the depth explored. Since raveling occurred in Boring 5; however, casing or drilling mud was not used to extend the boring to the desired depth."

The results of Boring 2, located under the center of the building, is presented in Figure 2.7; the key to log of borings is shown in Figure 2.8.

FIG. 2.1 WESLEY MANOR BUILDING

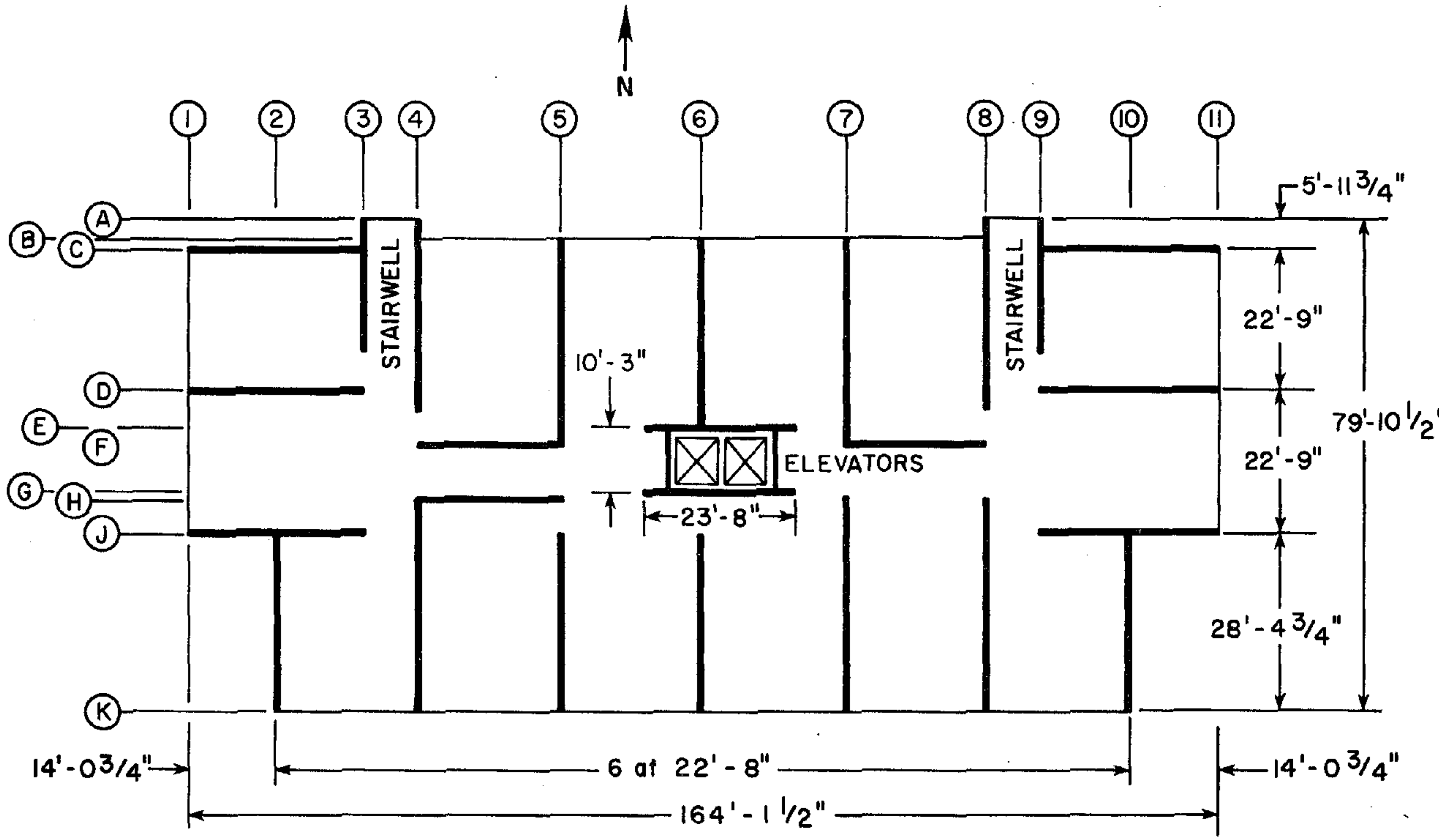


FIG. 2.2 TYPICAL FLOOR PLAN

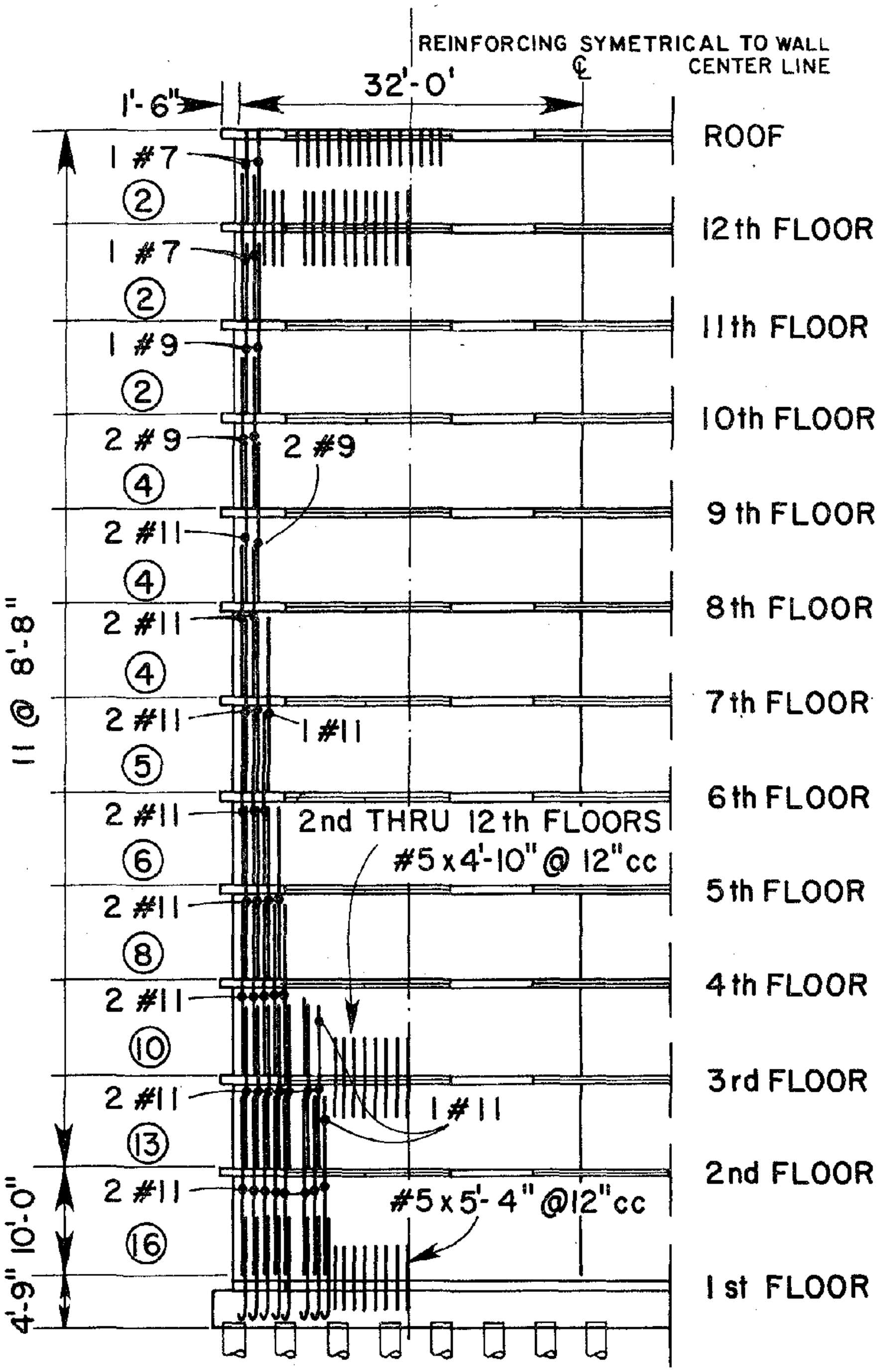


FIG. 2.3 TYPICAL WALL PANEL ELEMENT

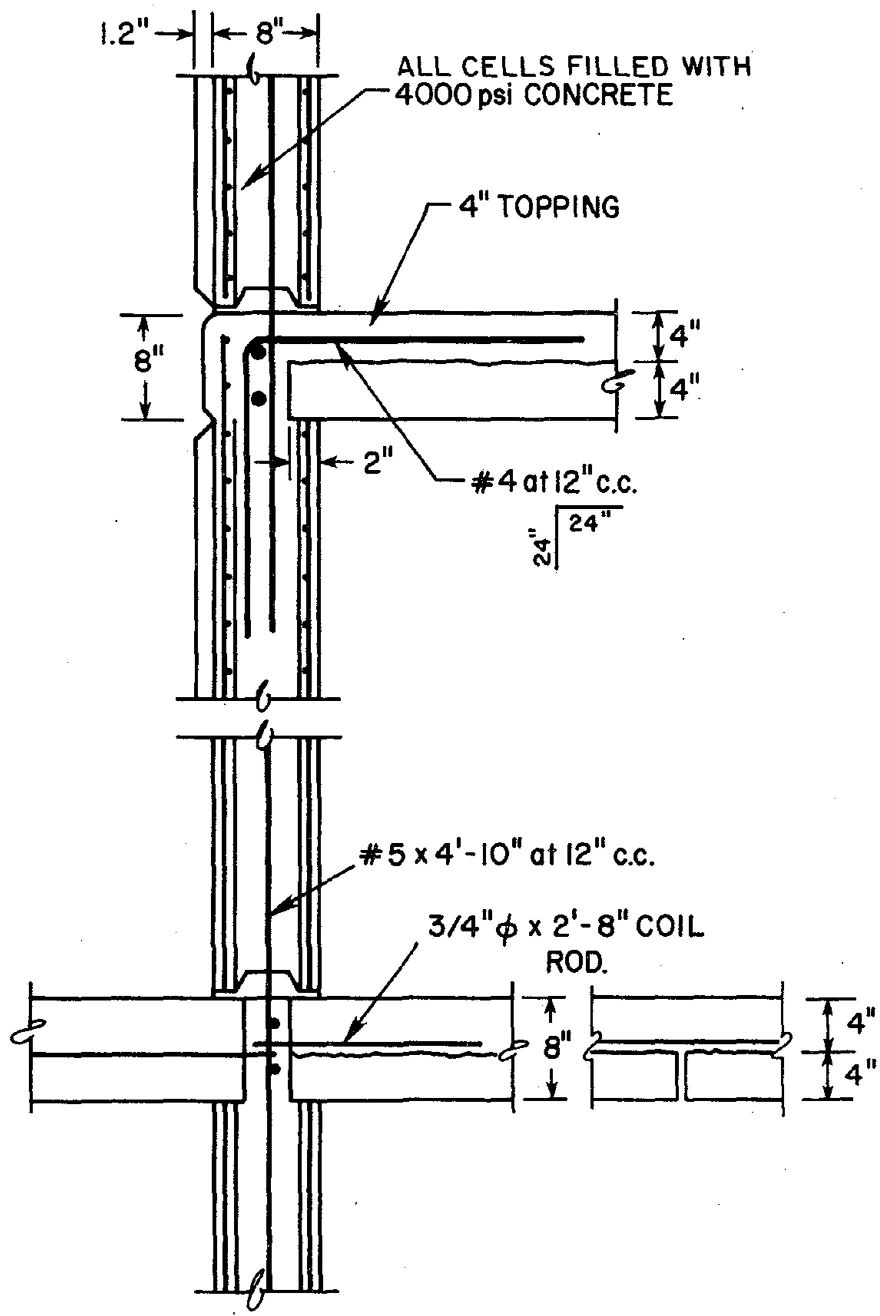


FIG. 2.4 TYPICAL WALL-FLOOR JOINT CONNECTION

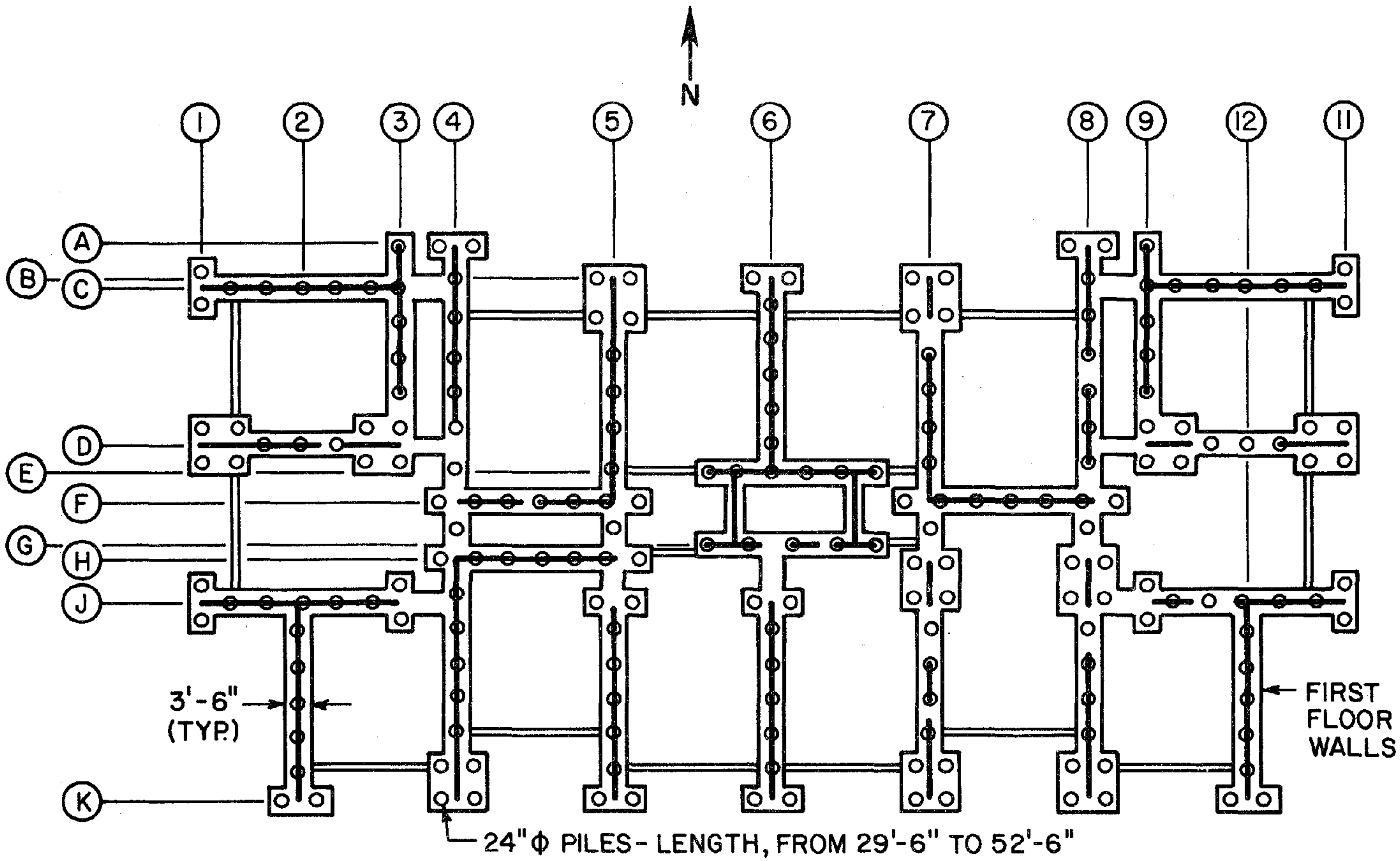


FIG. 2.5 TYPICAL FOUNDATION PLAN

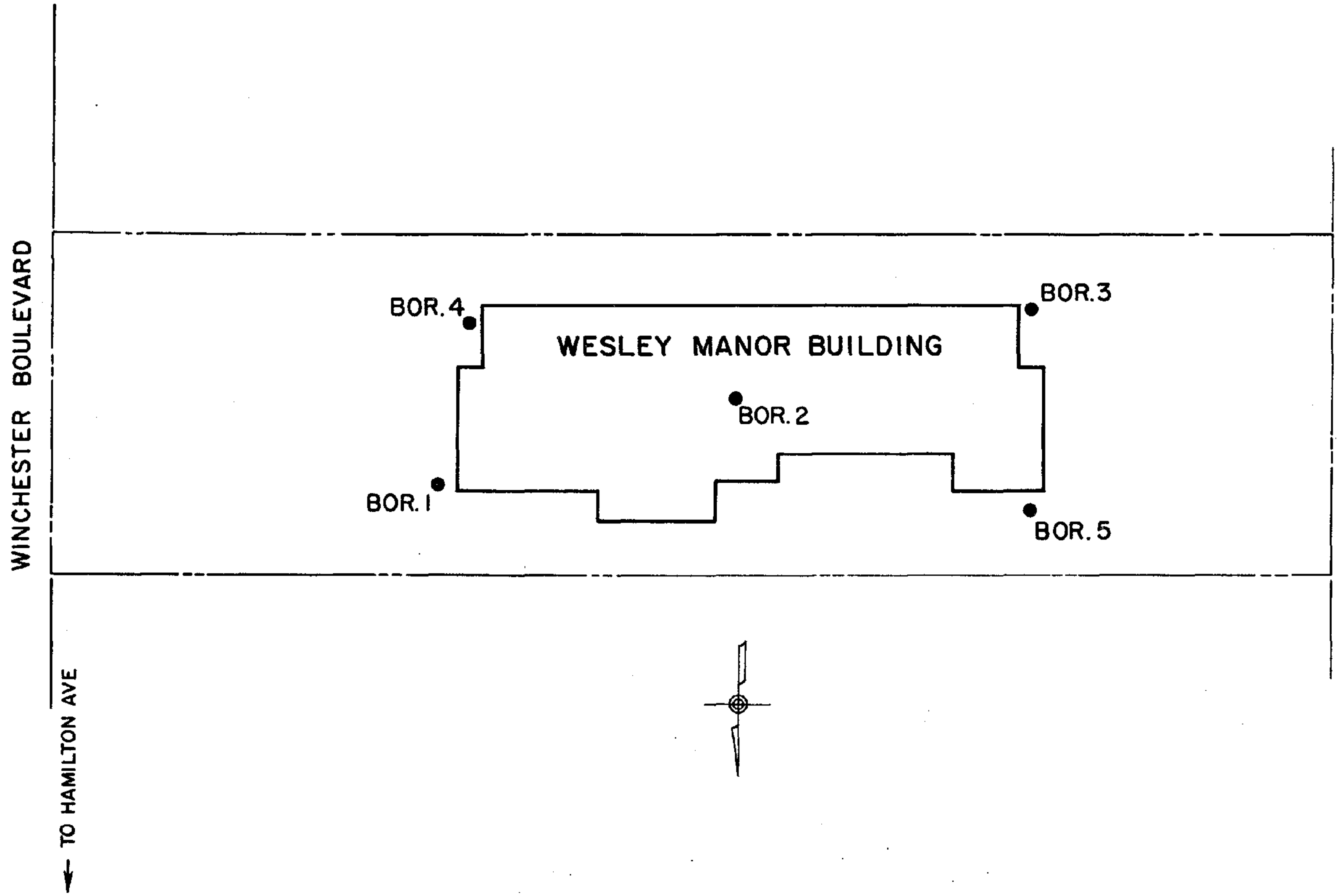


FIG. 2.6 LOCATION OF SOIL BORINGS

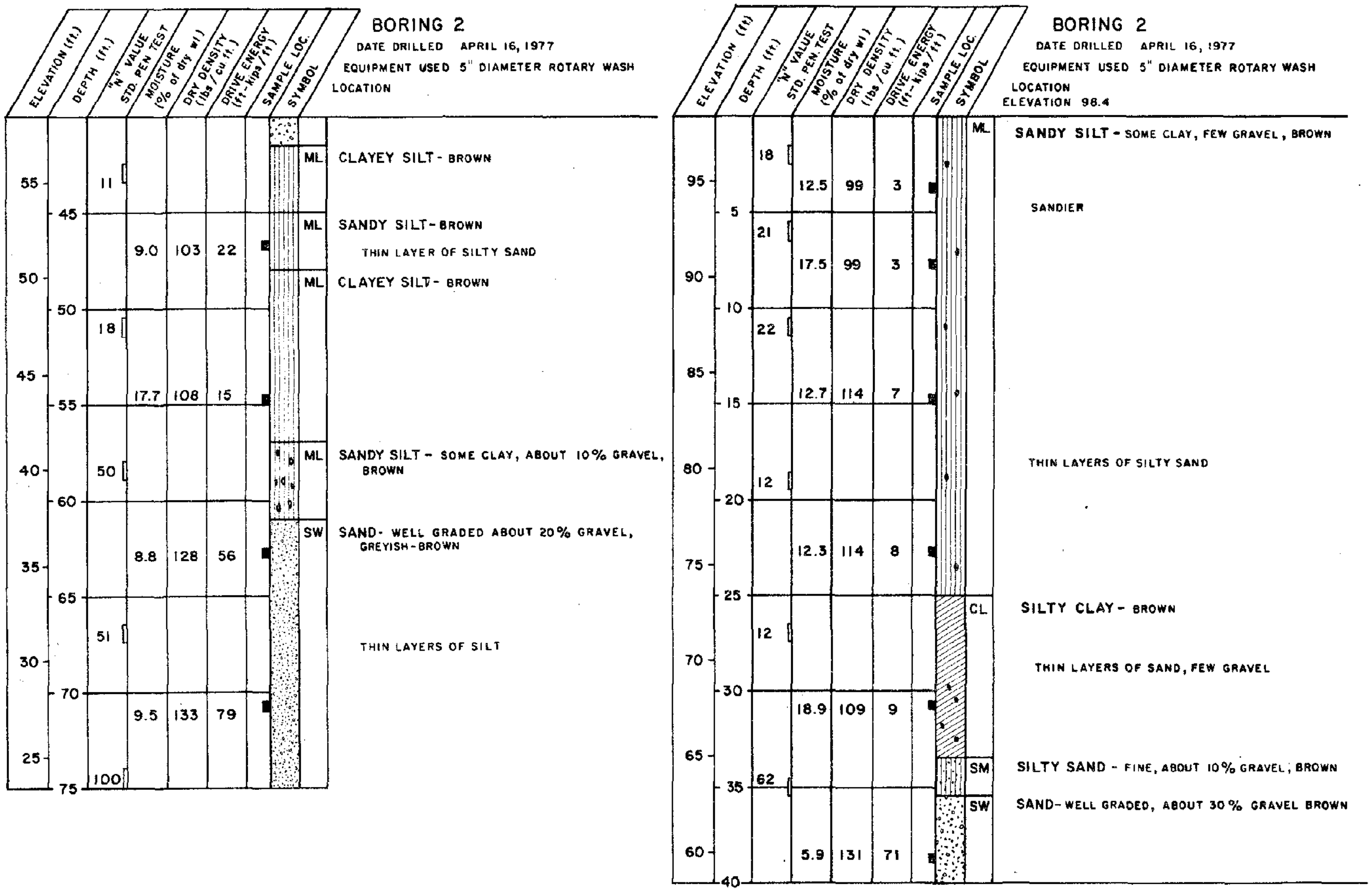


FIG. 2.7 LOG OF BORINGS

L. C. & A. SAMPLING: (Sampler Diameter - I.D. = 2.625". O.D. = 3.188")

5 Depth at which undisturbed sample taken

Energy required to drive L. C. & A. sampler 12", In ft.-kips per ft.:

Driving Weight = 350 lbs. Stroke = 1½'

STANDARD PENETRATION TEST:

22 Depth at which test performed

Number of blows required to drive Standard Penetration sampler 12":

Driving Weight = 140 lbs. Stroke = 2½'

DATUM:

Elevations refer to assumed datum; see Plate 1.

CLASSIFICATION SYSTEM:

Unified Classification System (Plate A-3).

FIG. 2.8 KEY TO LOG OF BORINGS

3. FORCED VIBRATION STUDY

3.1 General

The forced vibration study was carried out and completed during October 1978. 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 reduction applied, for forced vibration study conducted are also described. Finally, the experimental results are presented and discussed.

3.2 Experimental Equipment

The experimental apparatus employed in the tests were two vibration generators, twelve accelerameters 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 rotors, 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. 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 synchronized 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).

In the subject study the vibration generators were mounted on the 12th floor at the east and west sides of the building, as indicated in Fig. 3.3; namely, about 80 feet. Associated vibration control and recording equipment was also placed on the 12th floor.

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 of rotation of the baskets. Hence, the maximum accuracy of 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 and 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 rapidly, 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, 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 stiffness 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 study, with twelve accelerometers available, it was decided to develop the mode shapes by taking simultaneous measurements at each floor, with one accelerometers kept in reserve.

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 and all equipment associated with them in their respective recording channels are cross-calibrated simply by placing them all together and measuring the vibration amplitude of all 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 a 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 = damping factor,

f = resonant frequency,

Δf = differences 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 which an equivalent viscous damping coefficient lies known. Meaningful ranges might be defined as: under 1%, 1-2%, 2-5%, 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

3.4.1 General

The vibration equipment was bolted to the 12th 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 selection of the location of the two vibration genreators is typically guided by the structural layout of the building to be studied, and thus, the anticipated dynamic response. In that respect, ideally, the shakers should be placed along one of the center lines and as far apart as possible (the latter requirement to achieve a maximum torsional input under a 180^{o}-out-of-phase excitation). As it was not possible in this case to install the equipment along the EW center line, it was decided to place the shakers as close as practically possible near that center line. Admittedly, the center line itself is an arbitrary line as it is based in the anticipated center of stiffness.

In general the vibration equipment allows excitation of a structure in both the NS, EW and torsional modes. Even with a certain off-line position (or eccentricity) of the equipment separate excitation of the translational and torsional frequencies normally cause little trouble, provided the translational and torsional resonance frequencies are sufficiently separated and the structural damping is small (less than 2 to 3% of critical). However, the Wesley Manor Building was found to be highly susceptible to a trnaslational (EW) - torsional coupling. Hence, with the shakers operating in a translational longitudinal (EW) forcing manner, the structure failed to develop a clean translational excitation. Instead, at resonance, a combined translational-torsional motion developed. In fact, the frequency response behavior of the building under longitudinal translational excitation and torsional excitation was found to be virtually identical, indicating the absence of a true translational resonance condition.

3.4.2 Frequency Response Data

The frequency response curves for North-South, East-West and torsional forcing conditions are presented in Figures 3.4, 3.5 and 3.6. The first figure shows a clear N-S translational resonance frequencies at about 2.2 cps. However, Figures 3.5 and 3.6 both indicate a large response at about 1.76 cps and a considerably smaller at about 2.10 cps. The results seem to indicate that the translational and torsional resonance conditions coincide;

an unusual condition. The basic response signals for the EW excitation were EW accelerations, while for the torsional excitation (NS 180° out-of-phase) NS accelerations were recorded. In general, 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. Also presented in Figures 3.4 thru 3.6 are the actual exciting force (F_r) and displacement amplitude (u_r) for each of the resonance frequencies, as well as the calculated damping ratios.

The resonant frequencies and critical damping percentages derived from the frequency response curves are summarized in Tables 3.1 and 3.2, respectfully. Also shown in the last table are the damping values obtained from free-vibration decay data. In that case the free vibration of the building was recorded following resonance excitation and subsequent stoppage of the vibration generators. The pertinent vibration data are plotted in Figures 3.7 thru 3.9.

TABLE 3.1 RESONANT FREQUENCIES (cps)

Excitation	Frequency (cps)
NS	2.18
EW	1.76
EW	2.09
Torsional	1.75
Torsional	2.09

TABLE 3.2 DAMPING RATIOS

Excitation (cps)	From Resonance Curves	From Decay Curves
NS (2.18)	2.2%	1.2%
EW (1.76)	1.4%	---
EW (2.09)	1.4%	2.8%
Torsional (1.75)	1.1%	1.8%
Torsional (2.09)	1.3%	---

Finally, the exciting force (F_r) generated by both shaking machines and the corresponding displacement amplitude (U_r) at resonance are tabulated in Table 3.3

TABLE 3.3 SUMMARY OF THE BUILDING RESPONSE AT RESONANCE

Excitation	Force or Moment	Response at 12th Floor [in.]	
		Center	West End
N-S (2.18 cps)	5,905 LB	25.62×10^{-3}	-----
N-S (2.22 cps)	984 LB	4.09×10^{-3}	-----
E-W (1.76 cps)	2,992 LB	20.41×10^{-3}	-----
E-W (2.09 cps)	4,134 LB	11.09×10^{-3}	-----
Tors. (1.74 cps)	241,784 LBFT	-----	41.84×10^{-3}
Tors. (1.76 cps)	141,256 LBFT	-----	31.62×10^{-3}
Tors. (1.79 cps)	50,910 LBFT	-----	9.18×10^{-3}
Tors. (2.08 cps)	334,069 LBFT	-----	20.59×10^{-3}
Tors. (2.12 cps)	73,214 LBFT	-----	4.16×10^{-3}

3.4.3 Mode Shapes

The vertical mode shapes under resonance are presented in Figures 3.10 thru 3.14. These mode shapes are plotted along vertical lines at the center of the building and along a center line in the west side of the building.

The NS motions are typically plotted to the right and the EW motions to the left of the two vertical axes (center and west side). As shown in Fig. 3.10, a significant base rotation was recorded, indicating the need to consider soil-structure interaction in any analytical procedure.

The horizontal mode shapes for both the 6th and 12th floor levels at the different resonance frequencies are shown in Figures 3.15 thru 3.19. General observations based on acceleration data recorded at different slab locations, clearly indicated that the floor slabs acted as rigid diaphragms. It should be noted that the NS resonance mode shapes are NS-normalized at the center of the roof. However, all other mode shapes, EW and torsional, are EW normalized at the center of the roof. Closer observation of the floor mode shapes at 1.76 cps for EW excitation and 1.74 cps for torsion indicates the clear absence of NS translation at the center. However, for the 2.09 cps and 2.08 cps resonance conditions the NS translational amounts to 30% - 40% of the normalized EW translational modal component. The rotational contributions due to both EW and torsional excitations are virtually the same, as shown in Figs. 3.16 and 3.17 (for EW excitation at 1.76 and 2.09 cps, respectively) and in Figs. 3.18 and 3.19 (for torsional excitation at 1.74 and 2.08 cps, respectively).

3.5 Discussion of Experimental Results

Only the first transaverse (N-S), longitudinal (E-W) and torsional resonance frequencies could be excited. Equipment limitations prevented the search for frequencies higher than 6.75 cps. Whereas the N-S mode has only small contributions from E-W and torsional motion, the E-W and torsional modes are highly coupled. This phenomenon is illustrated by the frequency response curves of Figs. 3.4 and 3.5 with resonant frequencies under both E-W and torsional excitation occurring at the same frequencies, namely 1.76 cps and 2.09 cps.

In case significant translational and torsional modal coupling occurs at two resonant frequecnies, one could term the resonance condition with the largest torsional components as the "torsional" resonant frequency and the other as the "translational" resonant frequency. However, as in this case the torsional components are virtually identical, such identification is impossible. This rare condition makes modal identification of the experimental data and a comparison with analytical results difficult.

The floor modes at the resonant frequencies were observed for the 12th and 6th floor. The floor slab behaved like a rigid diaphragm; a significant result in the subsequent development of the analytical model of this structure.

The damping factors were calculated from resonance curves (Figs. 3.4 through 3.6) and from force-vibration decay data (Figs. 3.7 through 3.9). The results from the two methods, compared in Table 3.2, are reasonably close and define a distinct damping ratio range of between 1 and 3%.

Finally, a comparison of the experimental resonance data with the results of a standard UBC analysis of the fundamental period, as $T = 0.05\ H/\sqrt{D}$ is of interest. The two different resonance data are presented in Table 3.4.

TABLE 3.4 COMPARISON OF FUNDAMENTAL EXPERIMENTAL AND UBC PERIODS

Excitation	Exp. Resonance (sec.)	UBC (sec.)	Difference
NS	0.46	0.59	- 22%
EW	0.57 0.48	0.41	+ 39% +17%

As the experimental EW resonance conditions did not permit identifying the fundamental EW resonance frequency, both resonance values are presented in the table. Despite the uncertainty, the results indicate that for the short NS (or, according to the Code, the "weak" direction) the UBC underestimates the effective stiffness (0.59 sec. versus 0.46 sec.). This situation would effectively become worse if, according to the Code, the foundation flexibility should be brought into account; thus increasing the period to about 0.7 sec. On the other hand, a similar correction would bring the Code and experimental EW resonance data in closer agreement. However, most likely, the experimental data would still reflect a lower stiffness condition in the longitudinal direction, than the Code would imply.

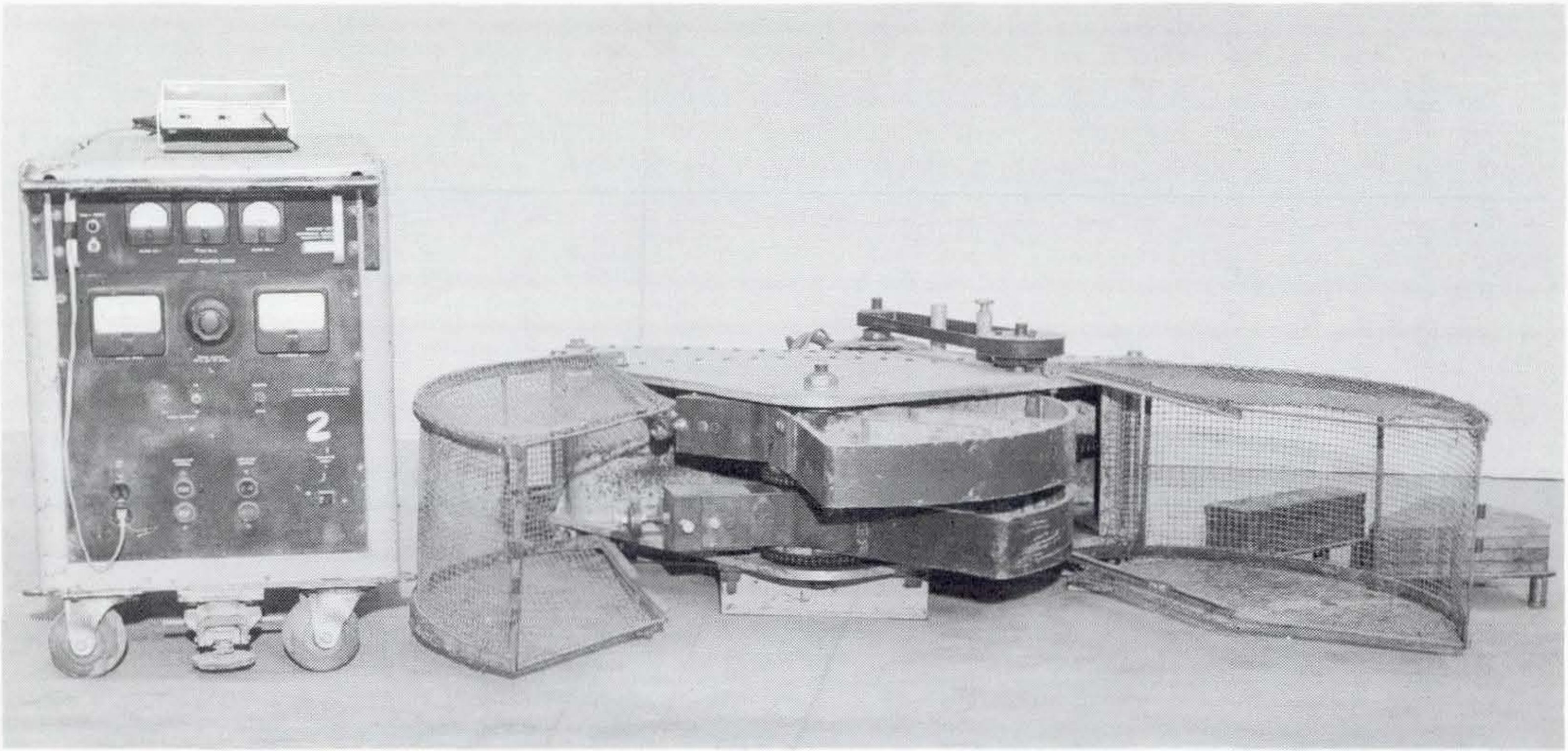


Fig. 3.1 Vibration Generator

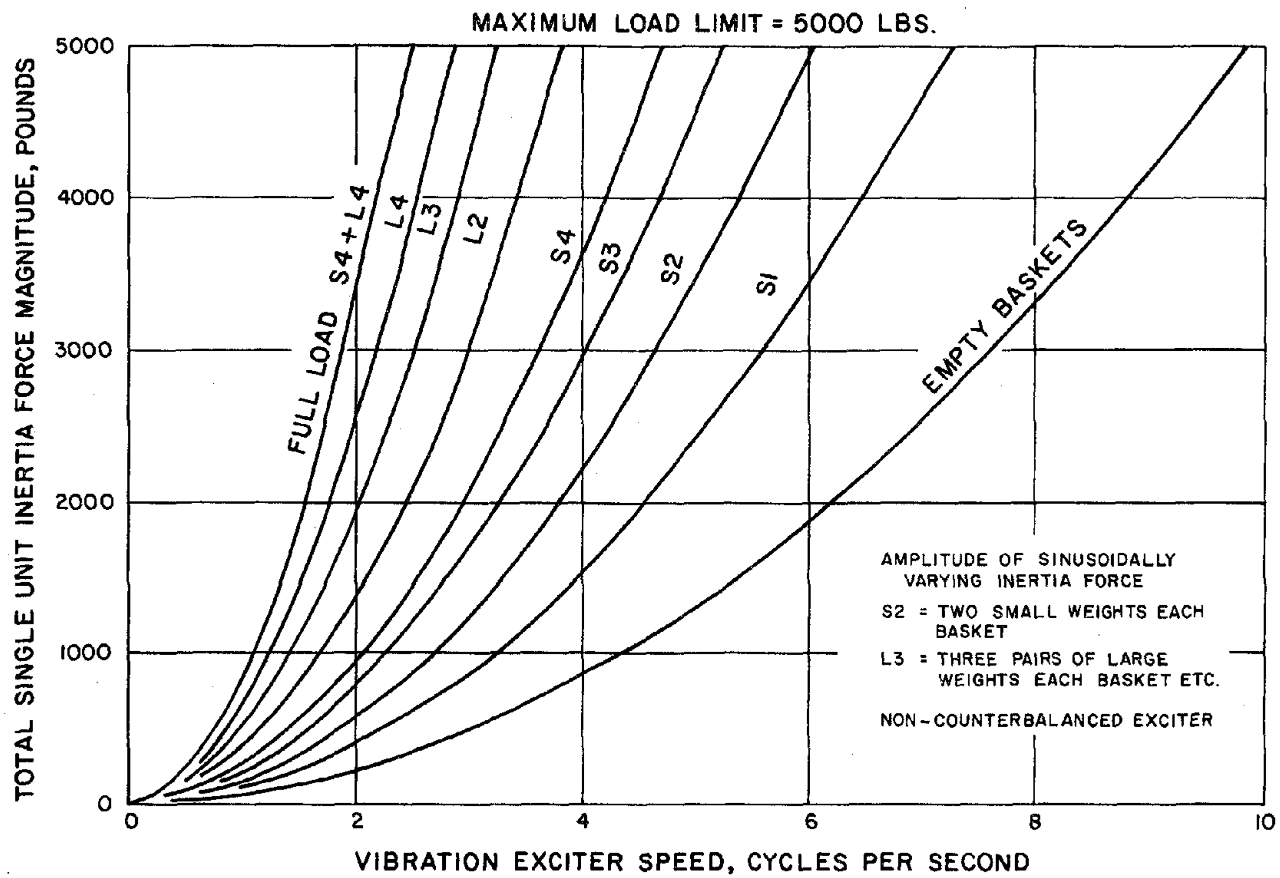


FIG. 3.2 VIBRATION FORCE vs. SPEED NON-COUNTERBALANCED

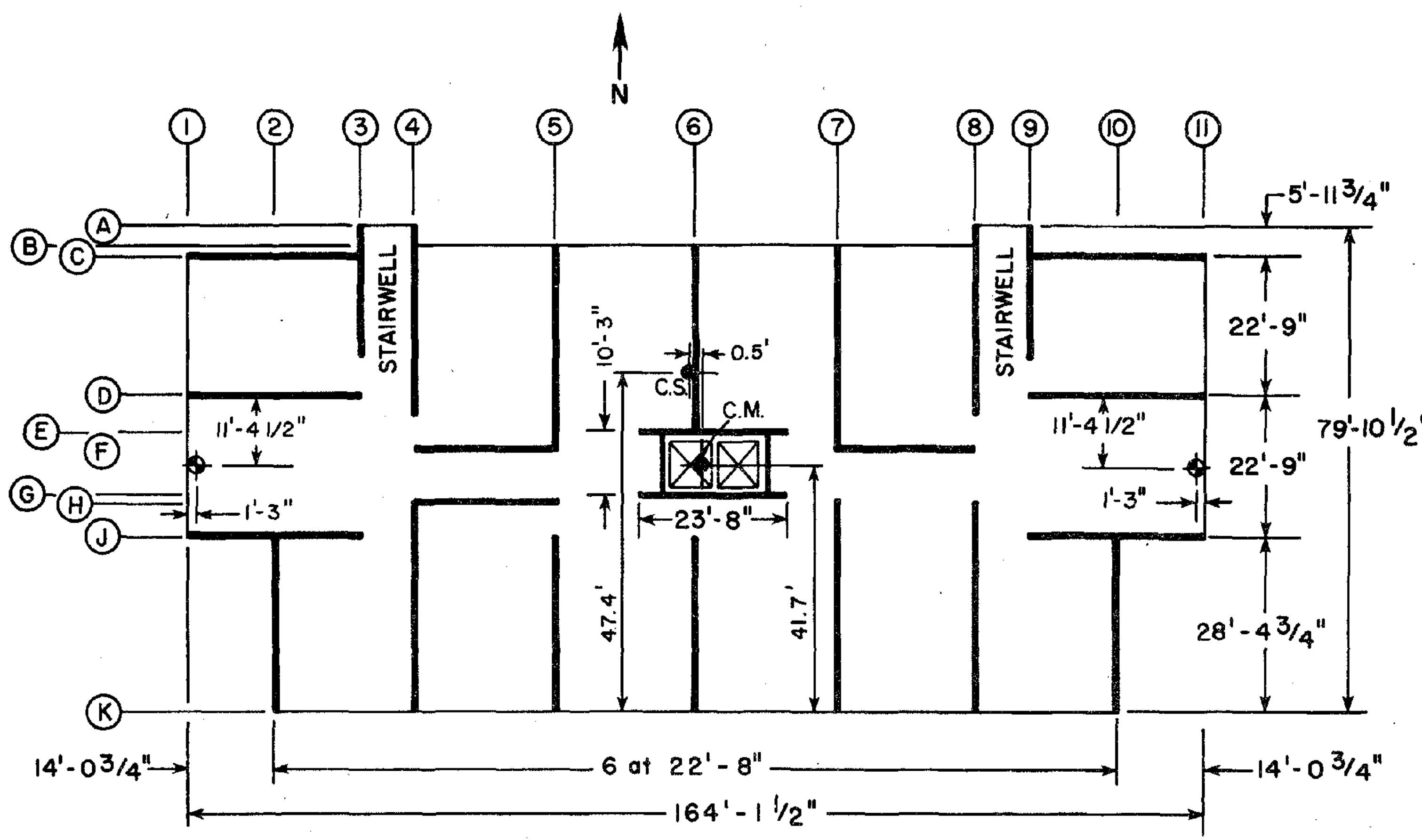


FIG. 3.3 LOCATION OF VIBRATION GENERATORS, CENTER OF STIFFNESS (C.S.) AND CENTER OF MASS (C.M.)

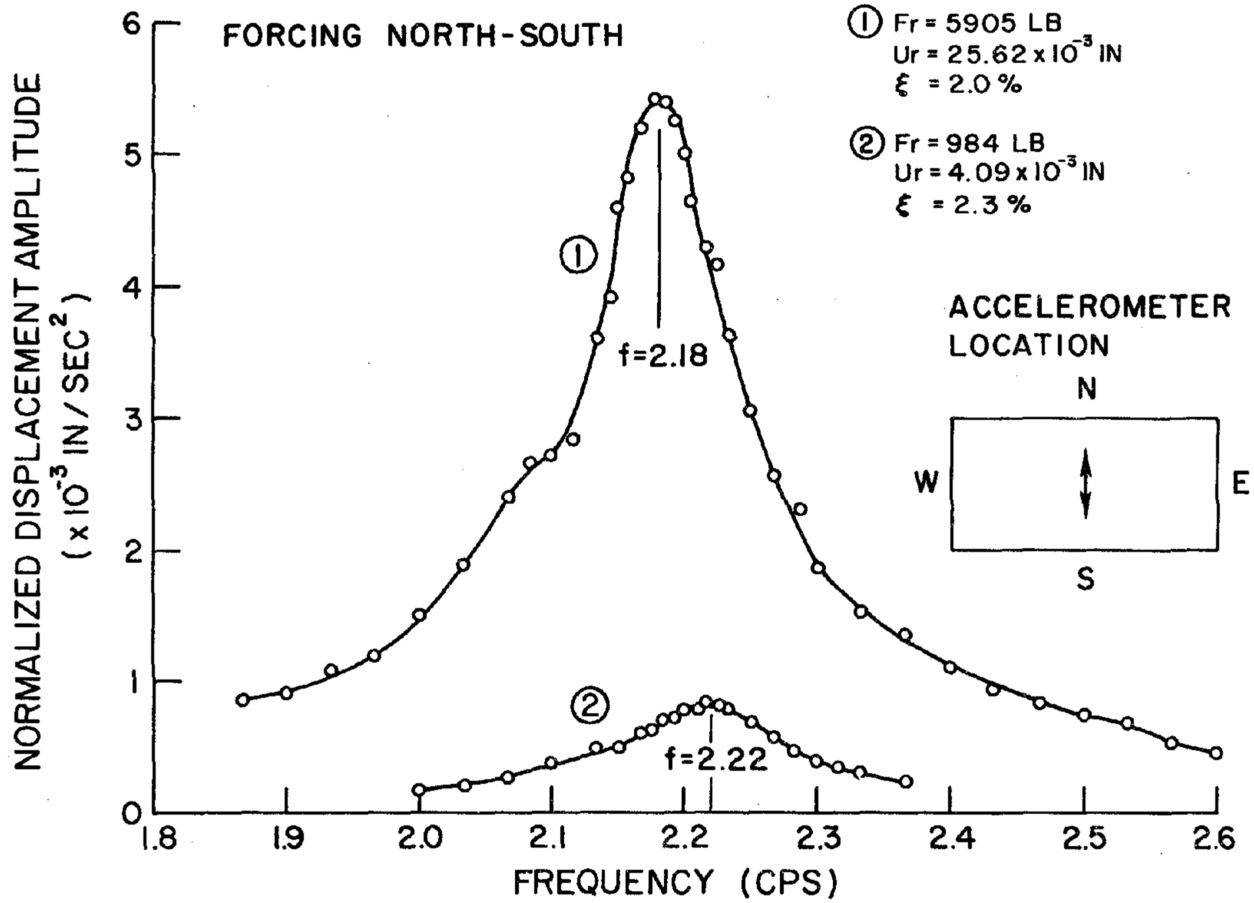


FIG. 3.4 FREQUENCY RESPONSE FOR NS FORCING

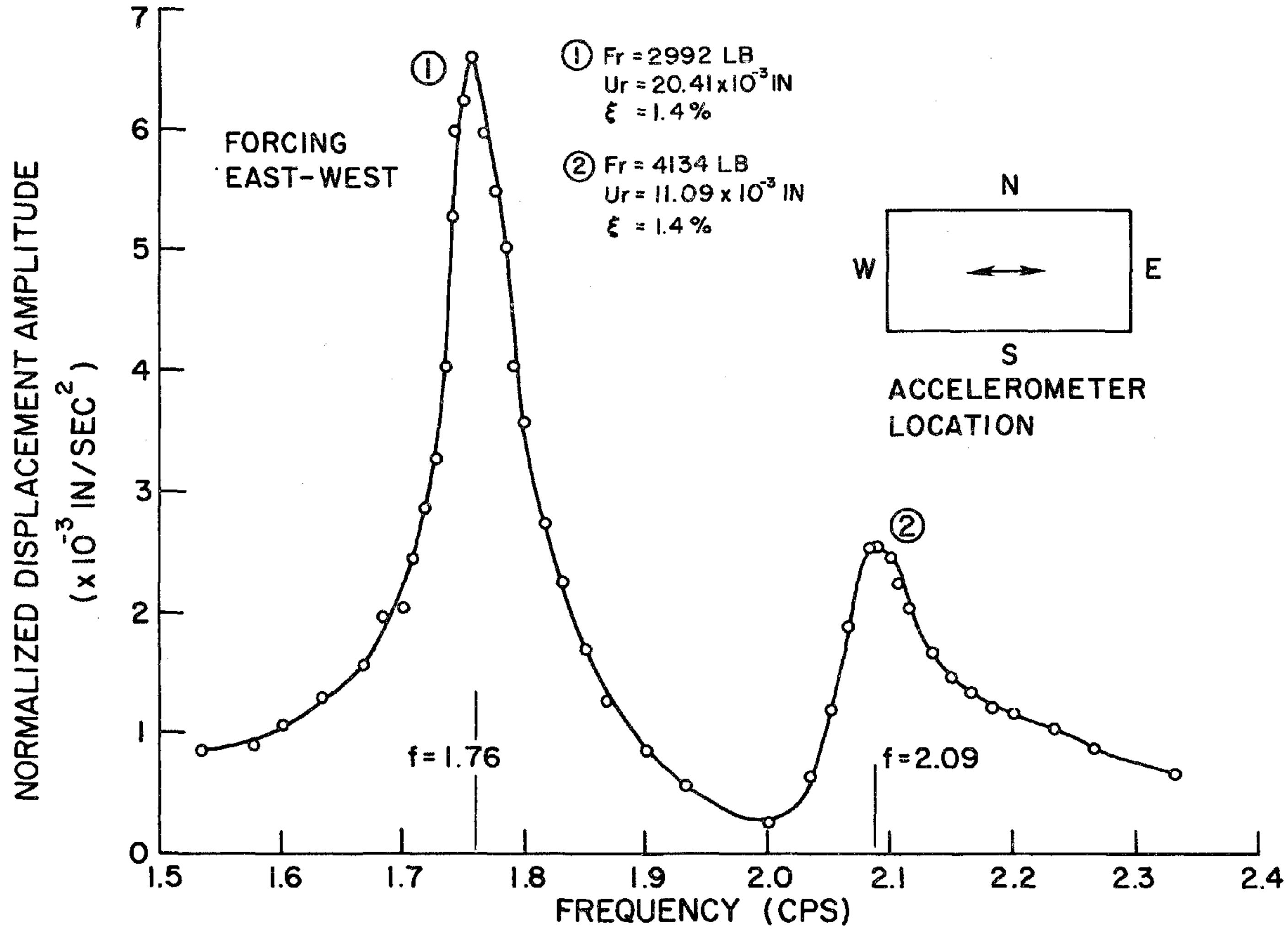


FIG. 3.5 FREQUENCY RESPONSE FOR EW FORCING

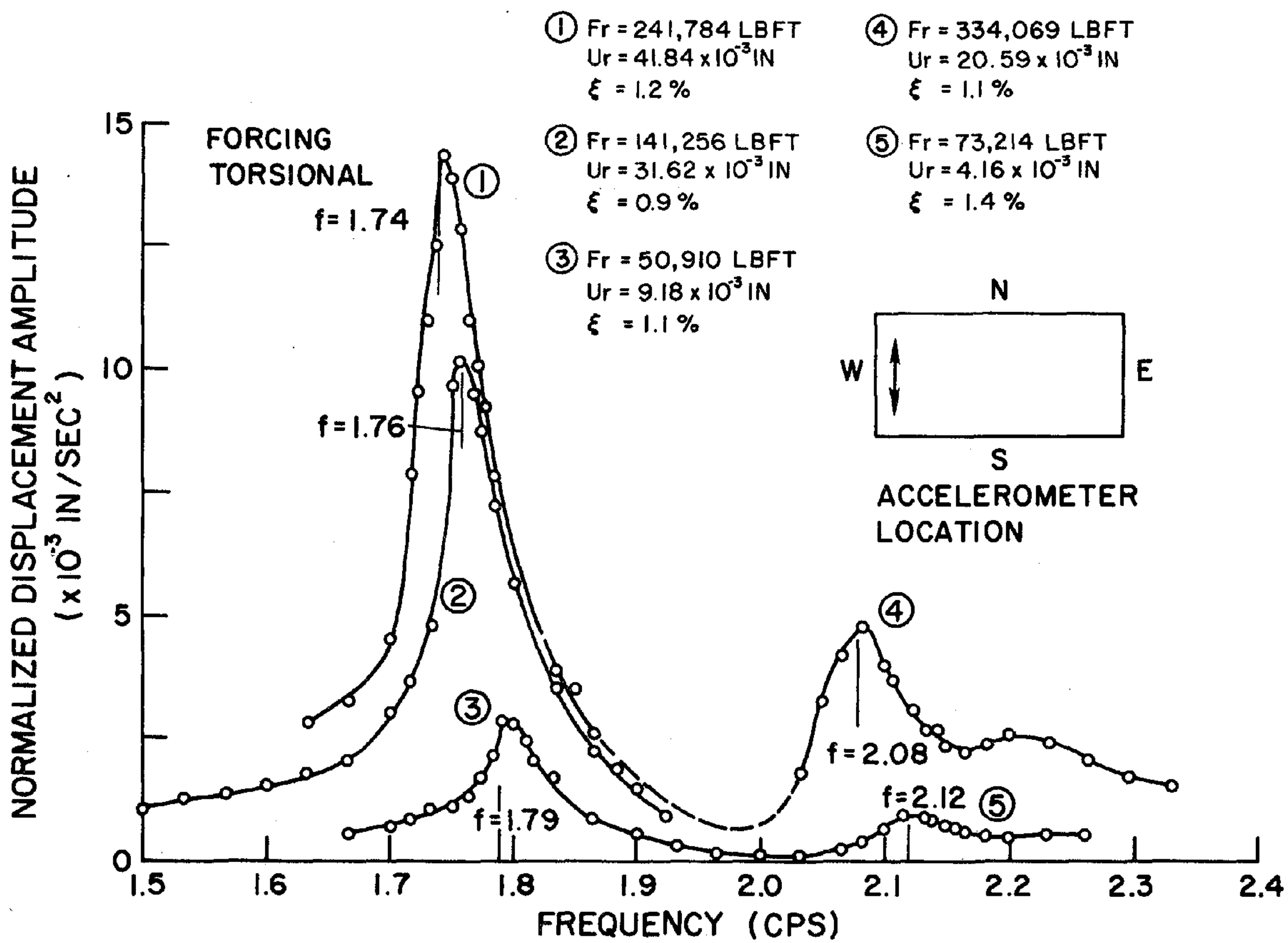


FIG. 3.6 FREQUENCY RESPONSE FOR TORSIONAL FORCING

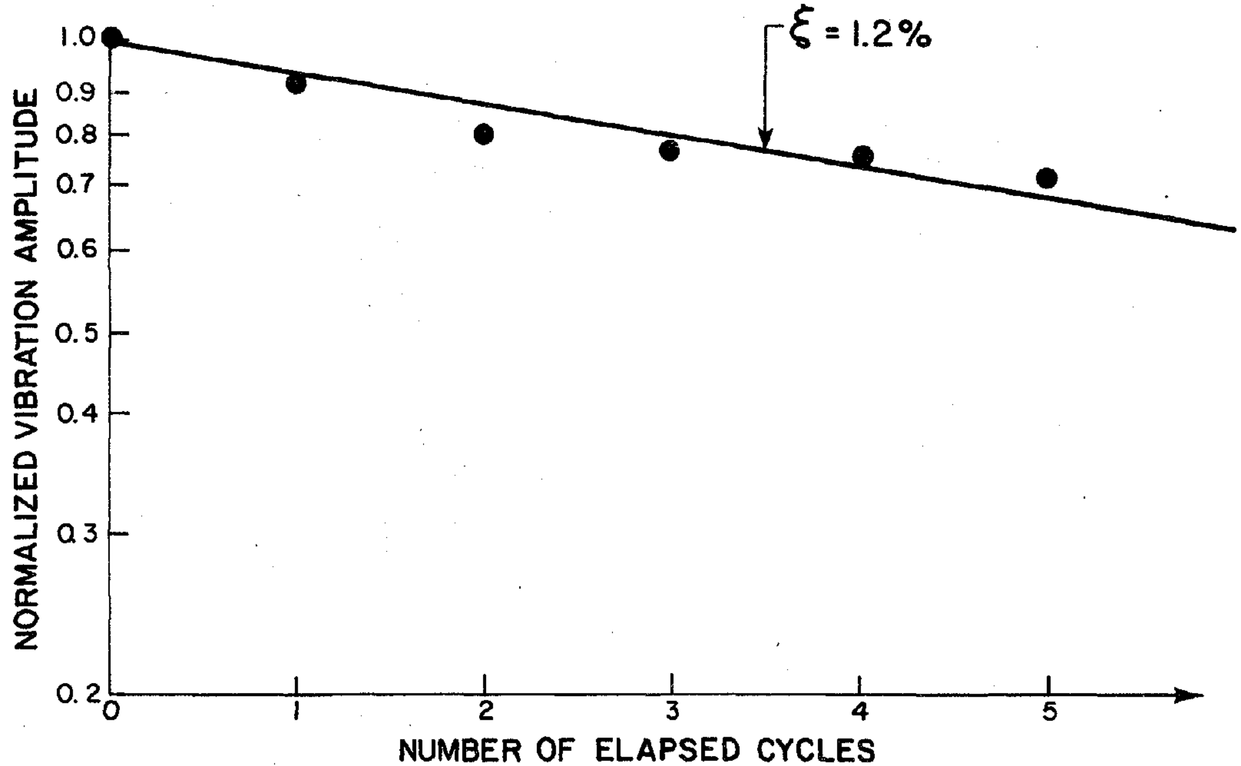


FIG. 3.7 LOGARITHMIC DECAY DATA: 1ST MODE NS (2.18 CPS)

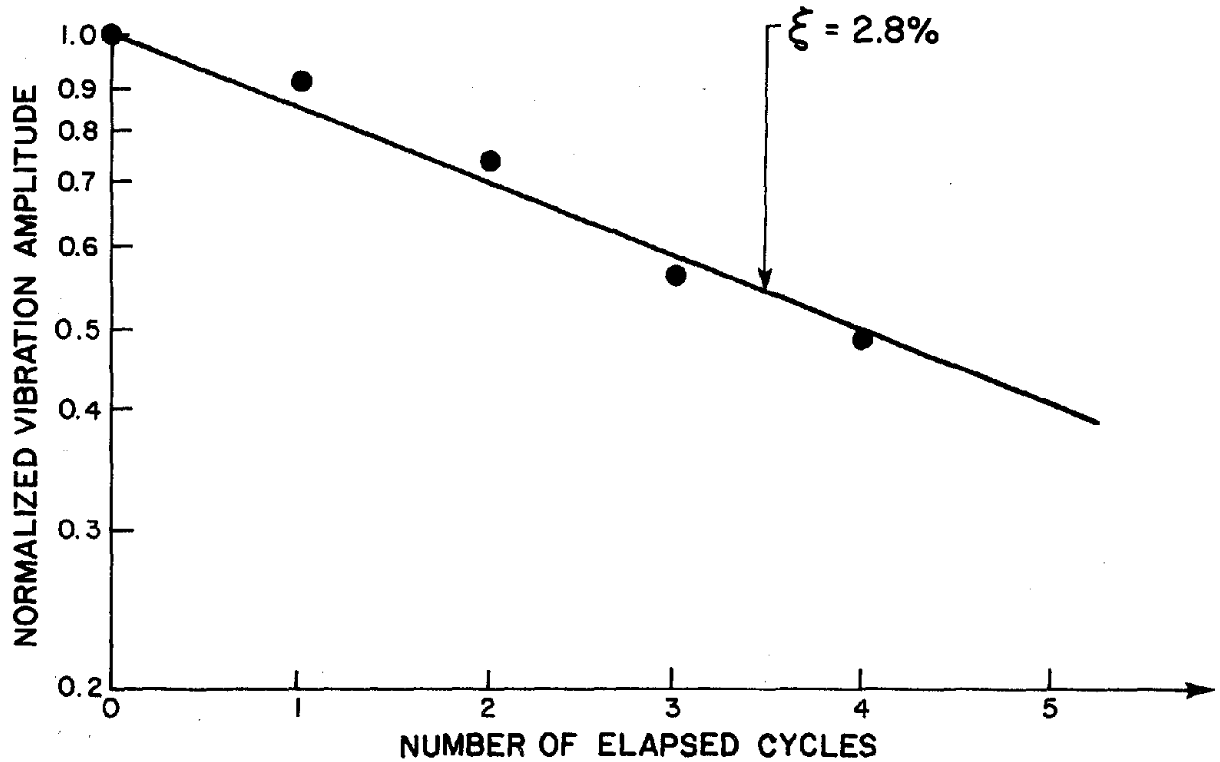


FIG. 3.8 LOGARITHMIC DECAY DATA: 1ST MODE EW (2.09 CPS)

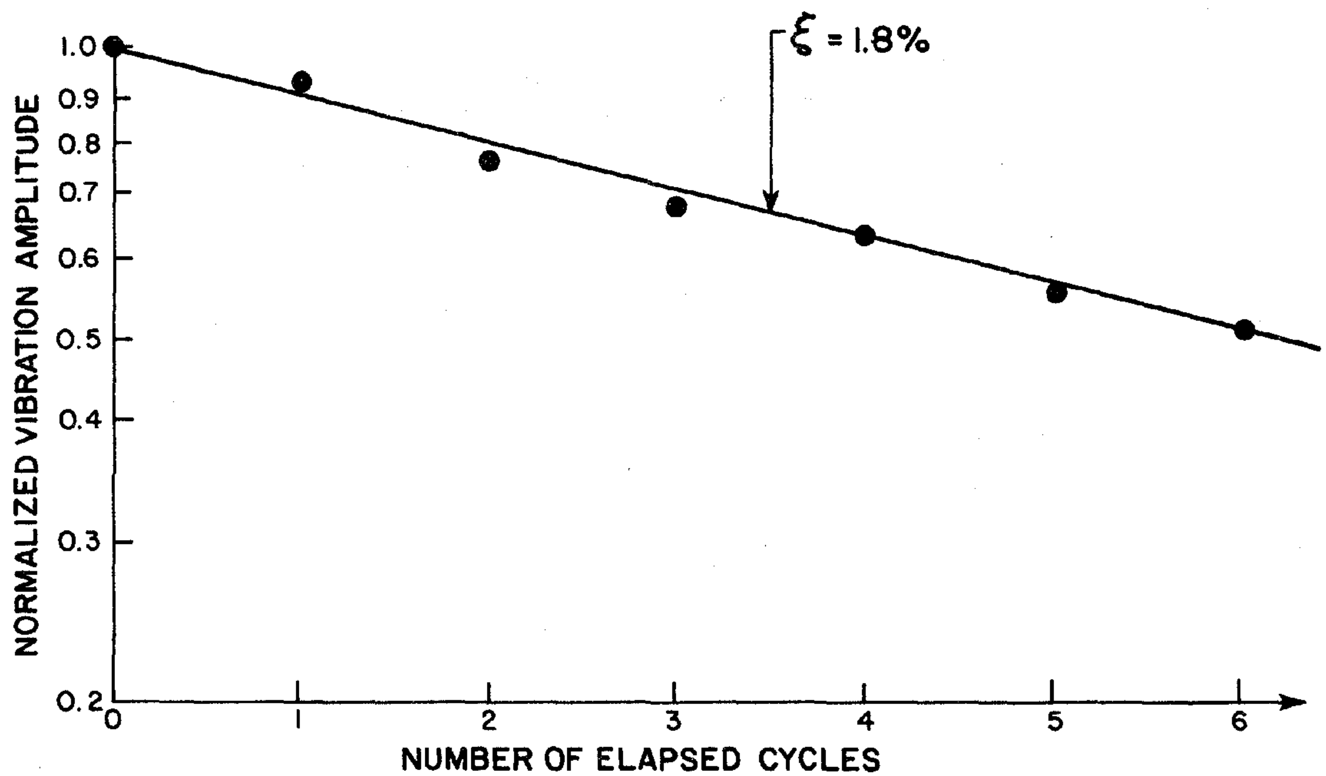


FIG. 3.9 LOGARITHMIC DECAY DATA: 1ST MODE TORSION (1.76 CPS)

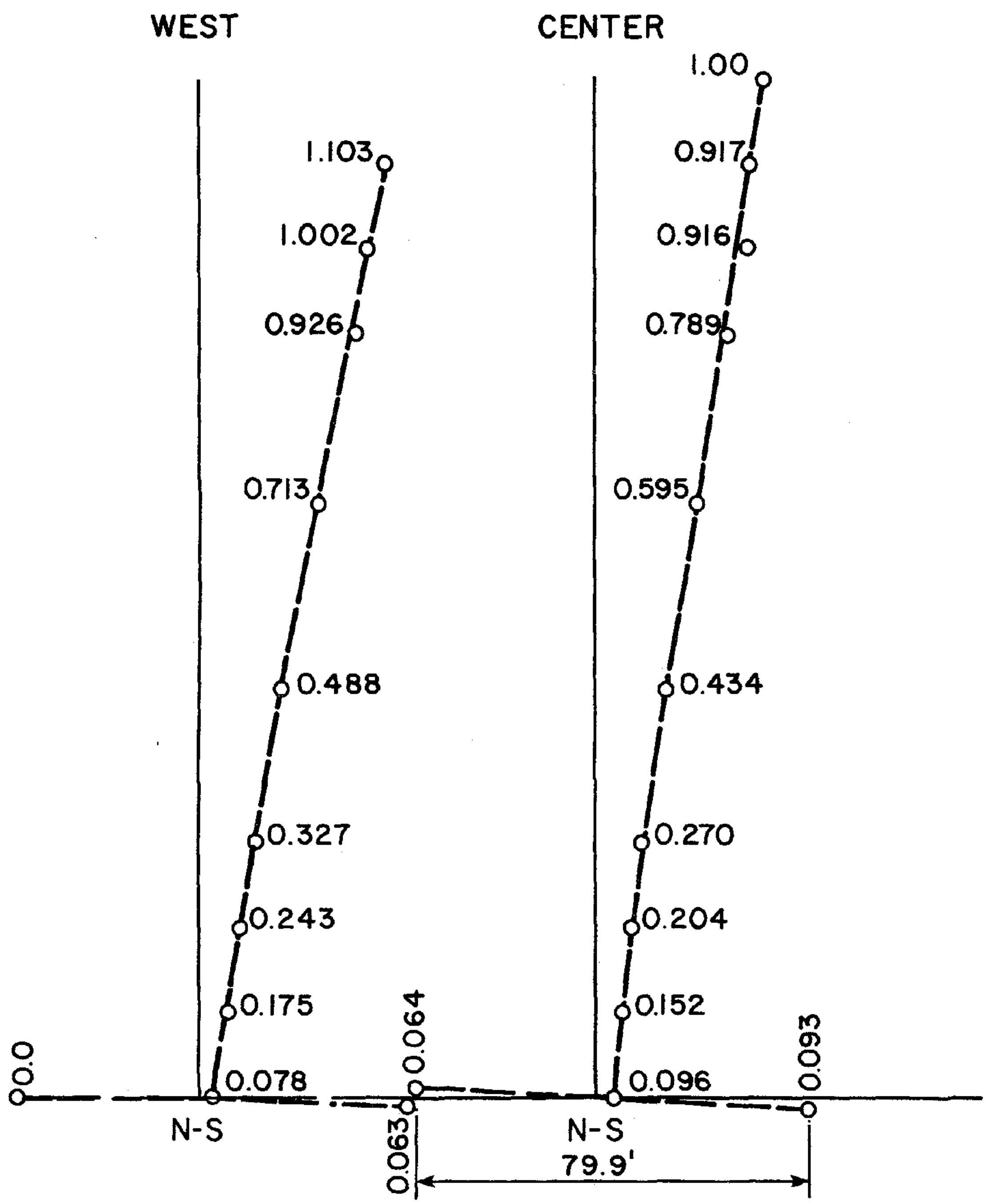


FIG. 3.10 FUNDAMENTAL NS MODE SHAPE (2.18 CPS)

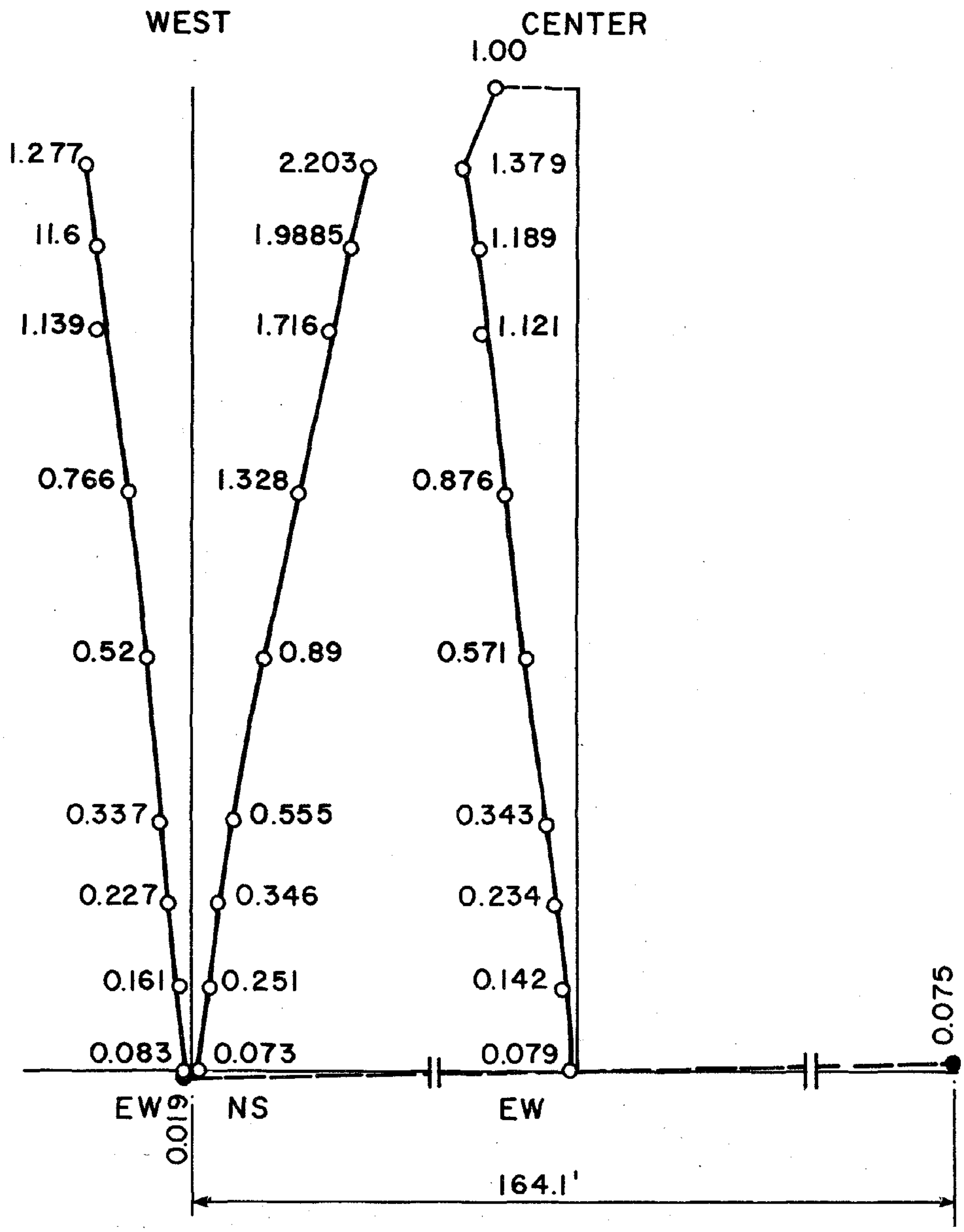


FIG. 3.11 VERTICAL MODE SHAPE EW FORCING (1.76 CPS)

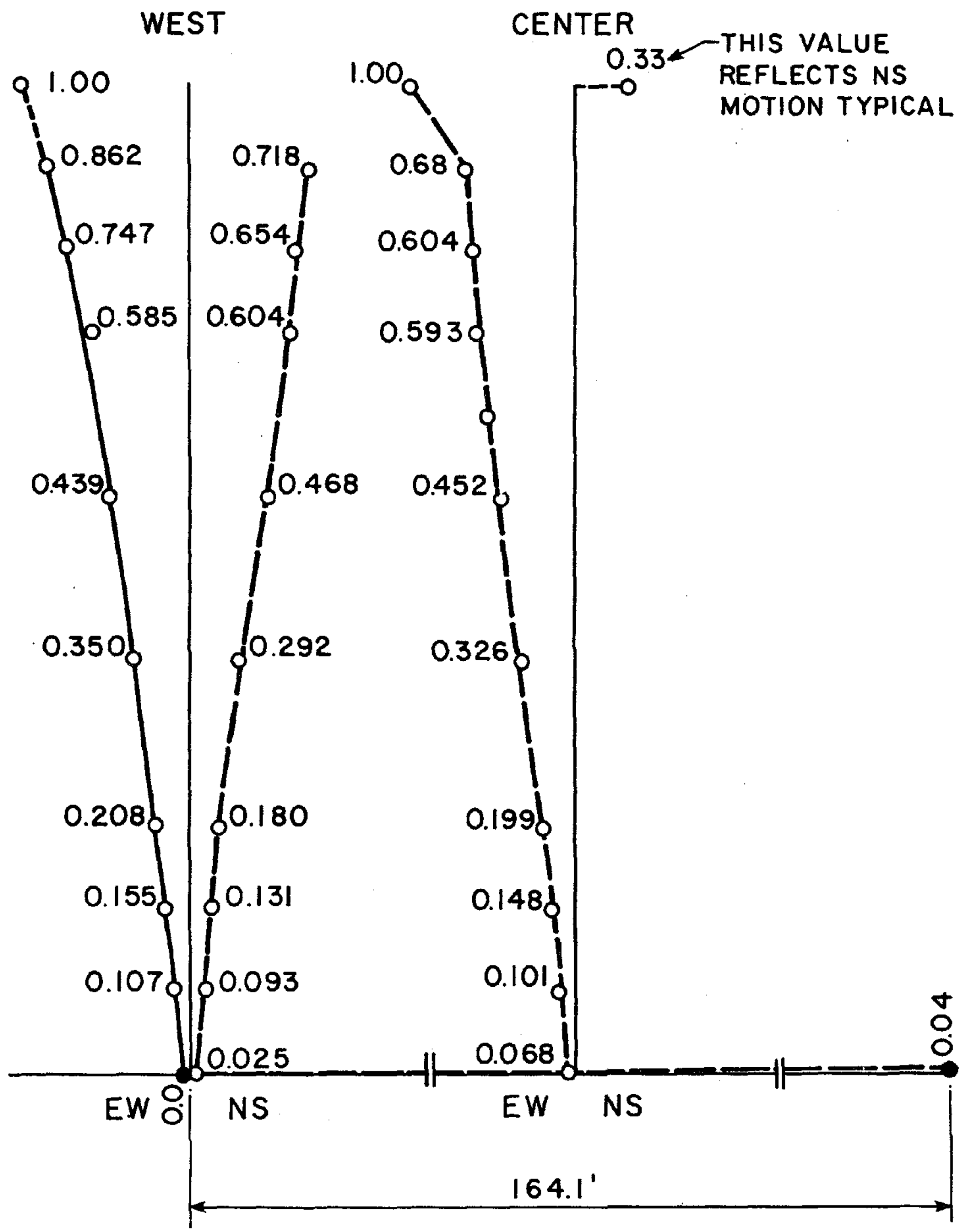


FIG. 3.12 VERTICAL MODE SHAPE EW FORCING (2.09 CPS)

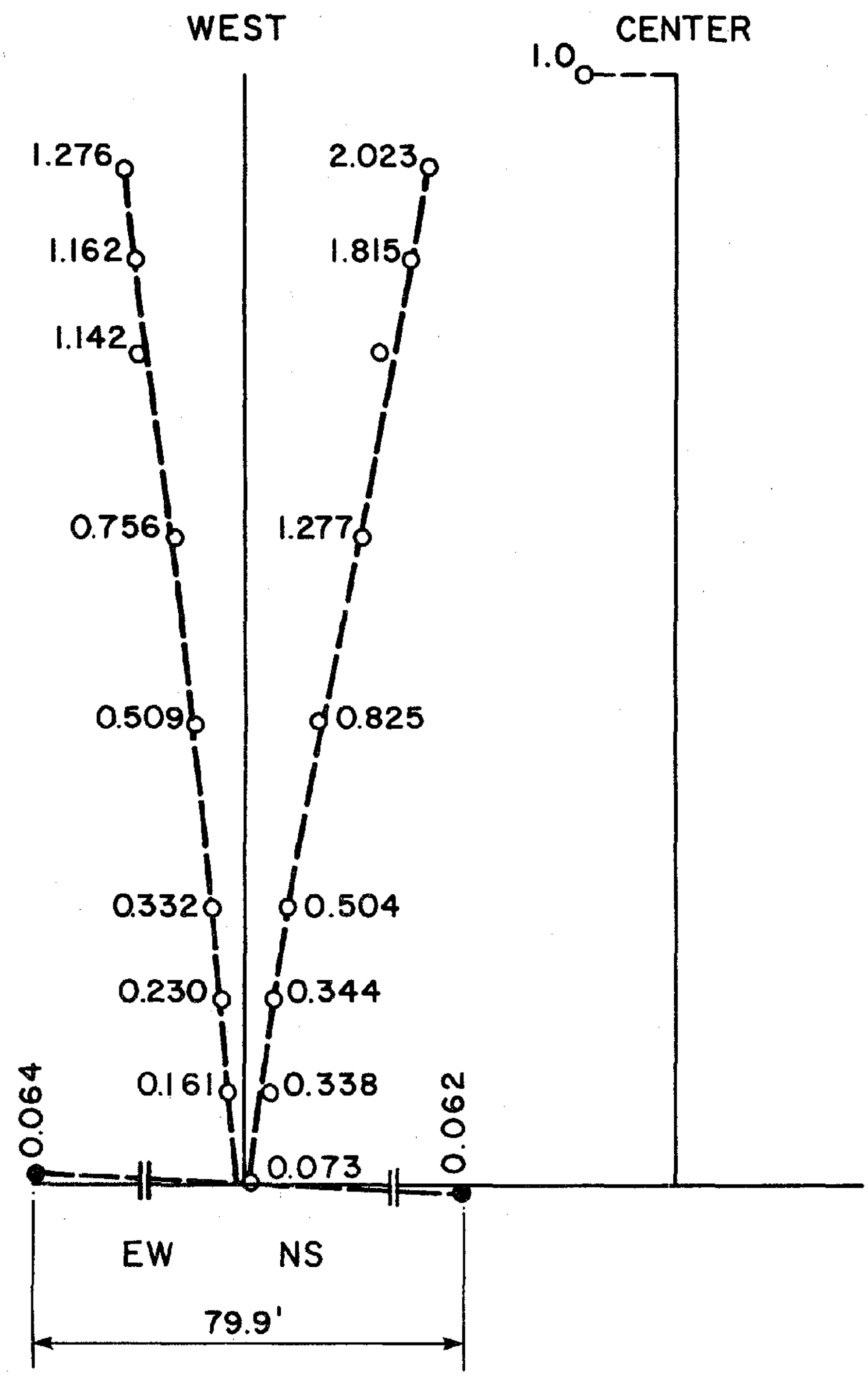


FIG. 3.13 VERTICAL MODE SHAPE TORSIONAL FORCING (1.76 CPS)

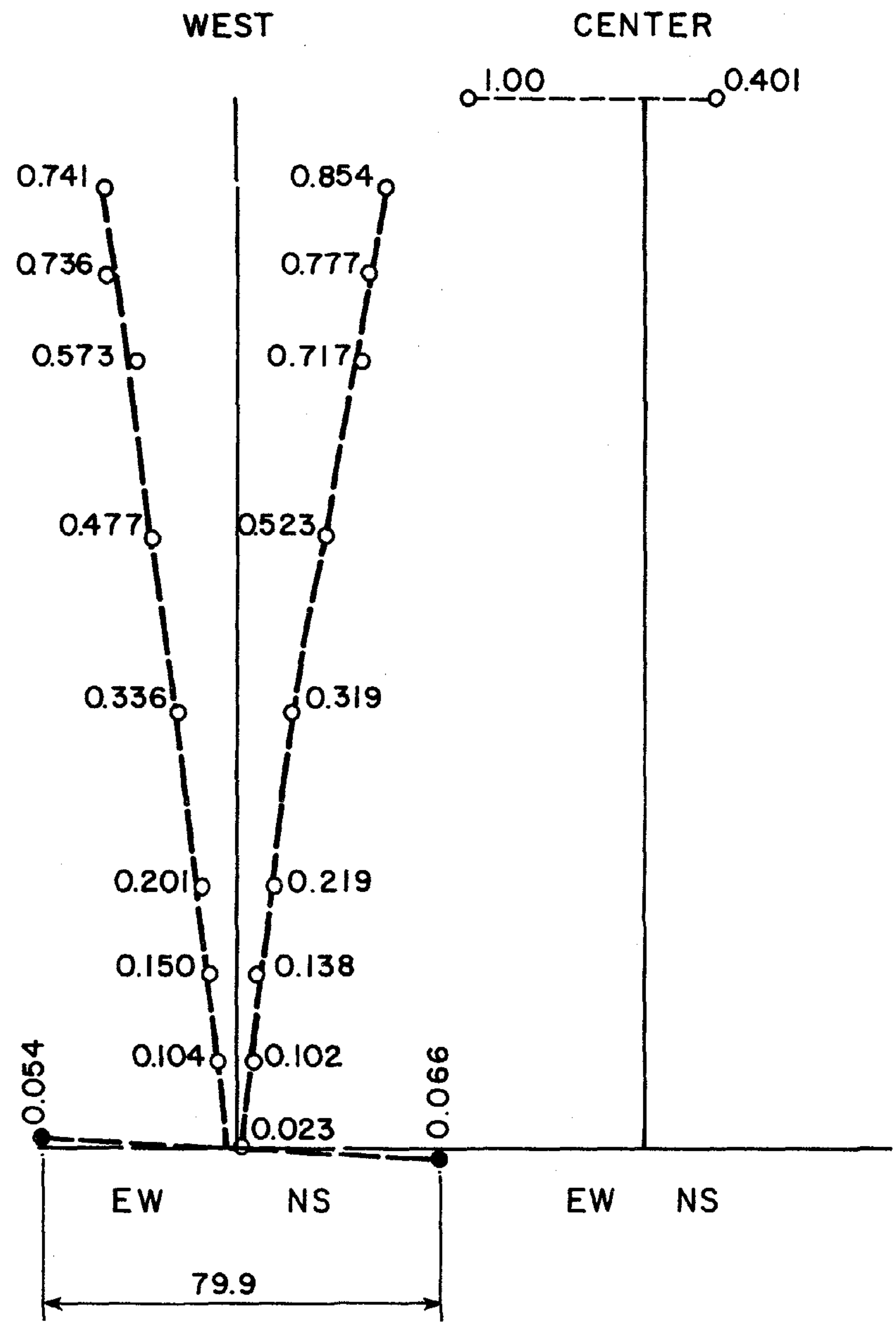
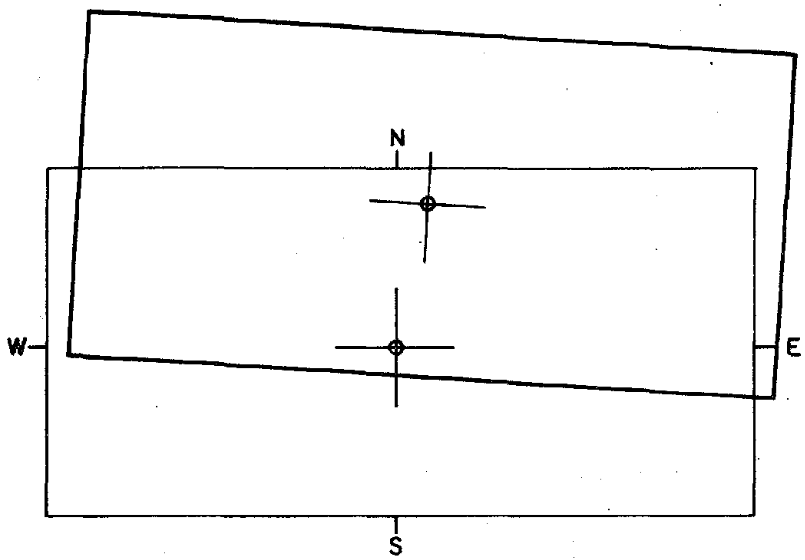
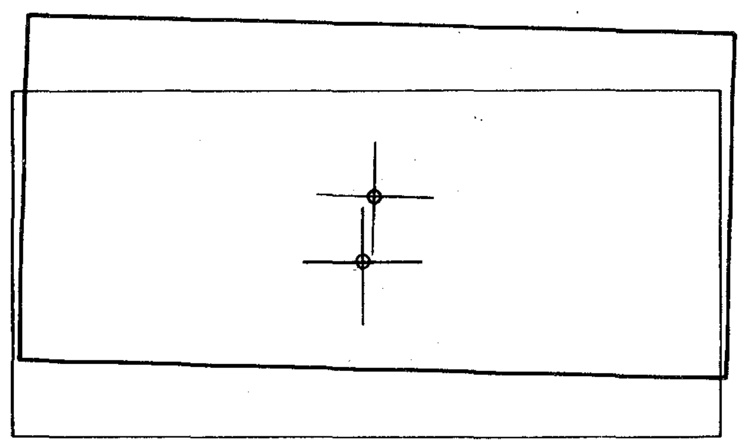


FIG. 3.14 VERTICAL MODE SHAPE TORSIONAL FORCING (2.09 CPS)

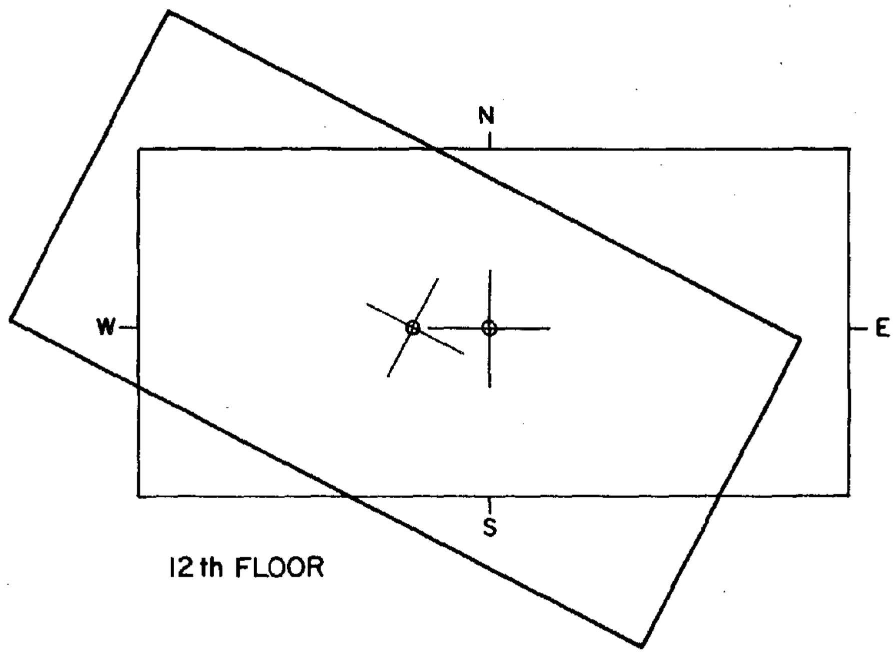


12 th FLOOR



6 th FLOOR

FIG. 3.15 NS FLOOR MODE SHAPES (2.18 CPS)



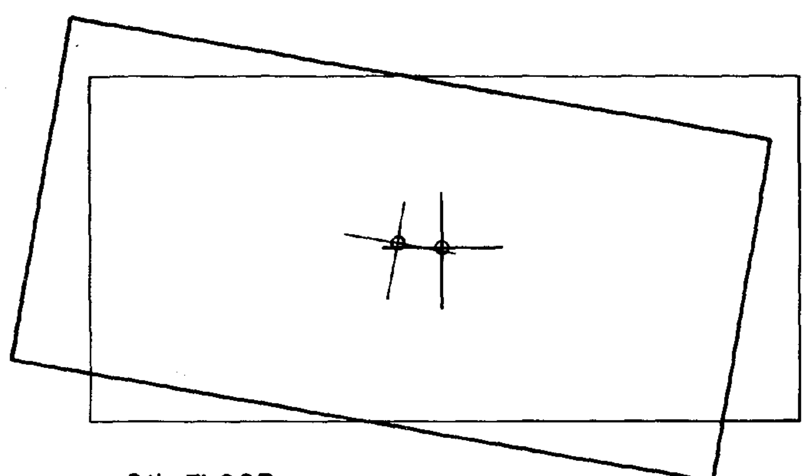


FIG. 3.16 FLOOR MODE SHAPES EW FORCING (1.76 CPS)

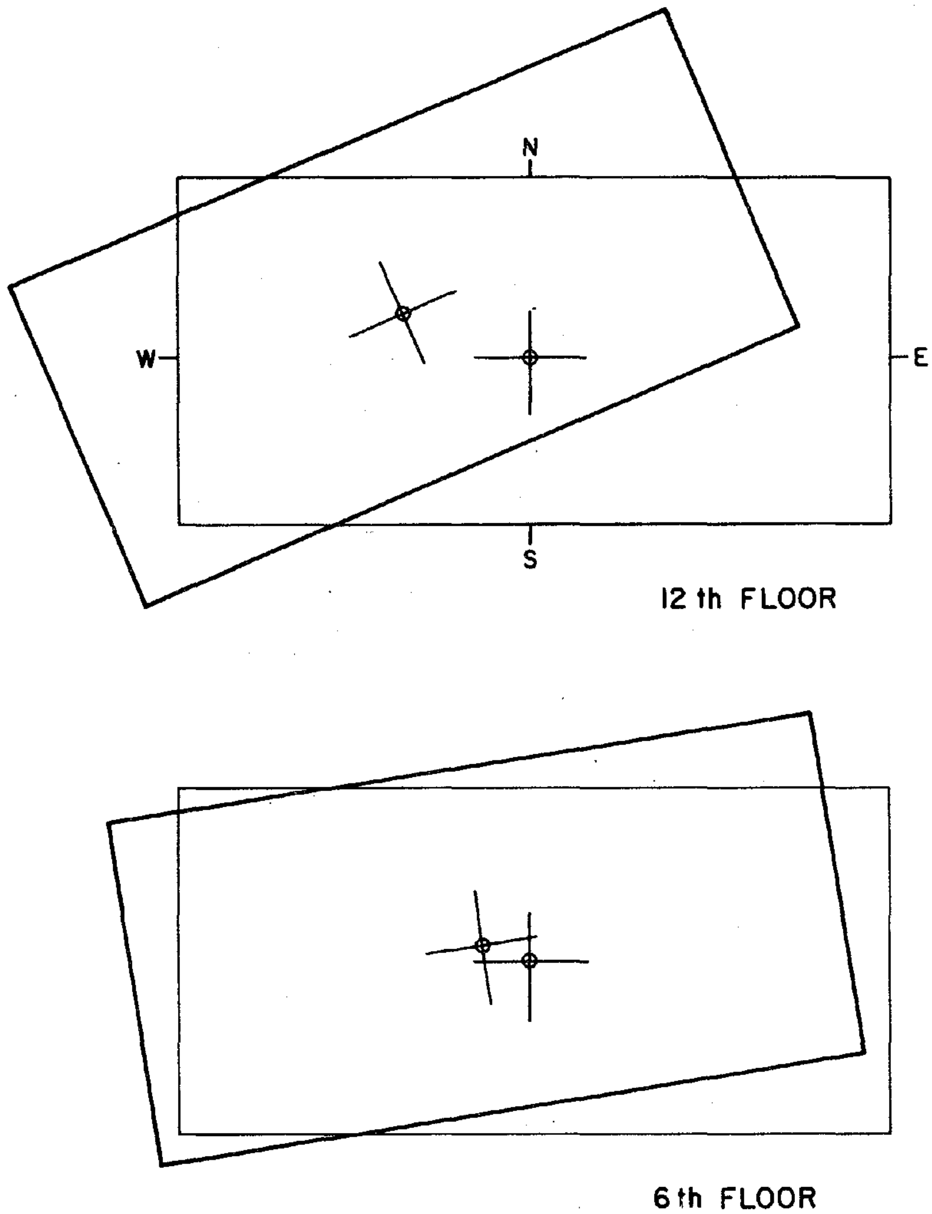


FIG. 3.17 FLOOR MODE SHAPES EW FORCING (2.09 CPS)

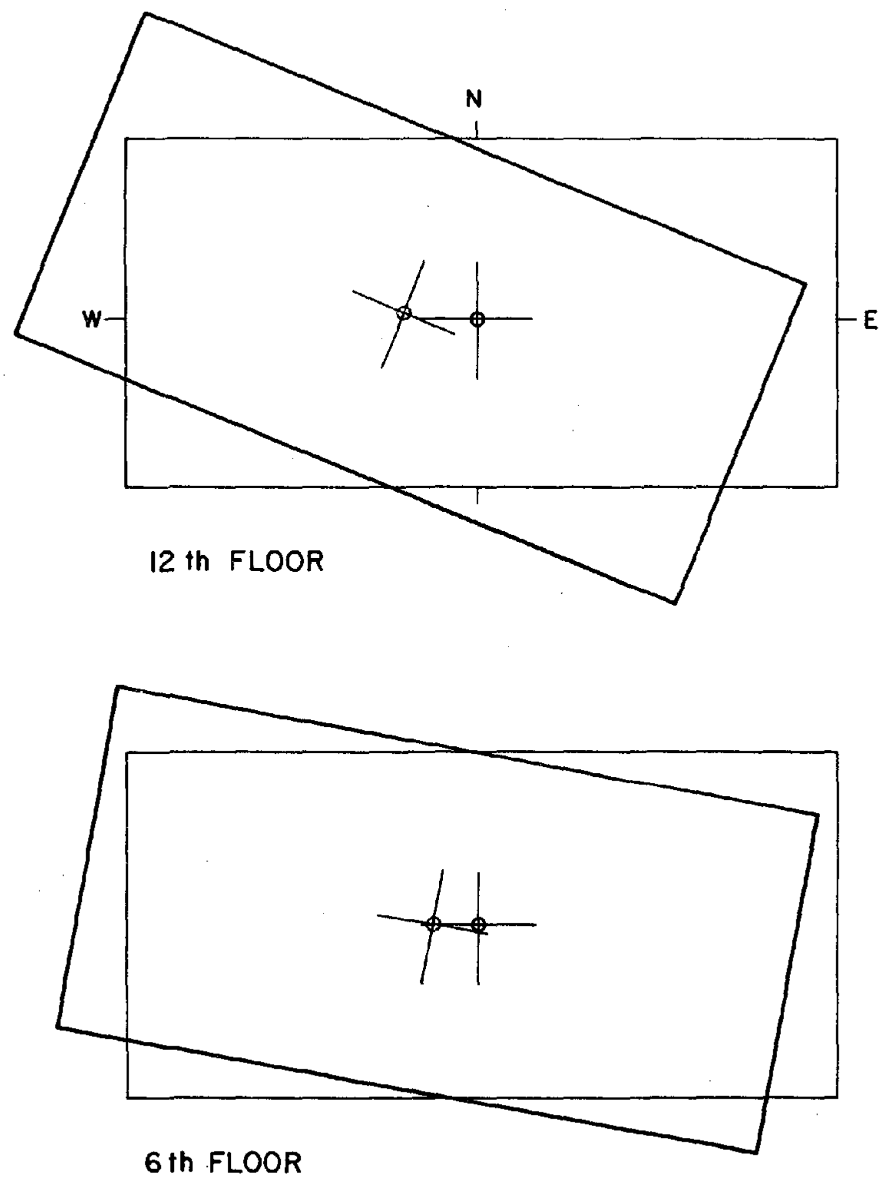


FIG. 3.18 FLOOR MODE SHAPES TORSIONAL FORCING (1.74 CPS)

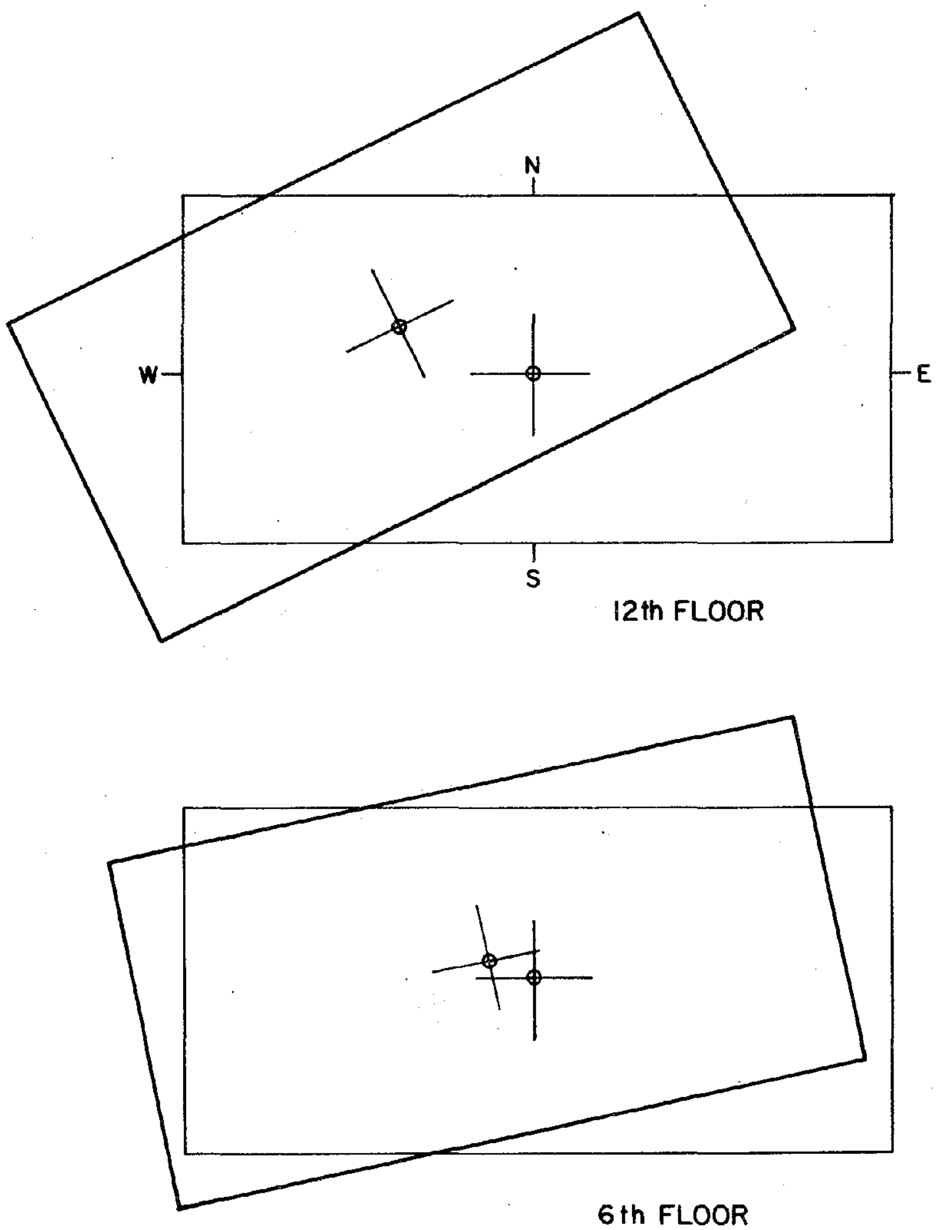


FIG. 3.19 FLOOR MODE SHAPES TORSIONAL FORCING (2.08 CPS)

4. AMBIENT VIBRATION STUDY

4.1 General

Ambient vibration studies use field measurements of wind and micro-tremor 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 simply method of field measurements. 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 amibent 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 Kinemetric Ranger Seismomenters, 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 force to extend the natural period of the mass and its suspension.

The resulting seismometer frequency was 1 Hz. Damping was set at 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 12th floor of the building (see Fig. 4.2). They were oriented so that the signals from the seismometers on the north and south sides could be used to detect the east-west frequencies. Similarly, the signals from those on the east and west sides were used to determine 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 seismoeter readings and the torsional frequencies from the average of the differences of the seismoeters 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 on the roof, as a reference 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 relocated successively on each floor to allow the evaluation of the model response over the height of the building (Figs. 4.4 through 4.8). As in the case for determining the modal frequencies, 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 roof motion. Torsional modal information was obtained in a similar manner, except that the difference, rather than the sum of the seismometer signals 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 seiemometers used was approximately 4.32 volts/in/sec. Corresponding first mode acceleration and displacement were about $\pm$ 0.03 x 10^{-5} g and $\pm$ 5.5 x 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 rate 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 resonant frequency runs, 4096 data points were selected for the translational 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 19 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 uniformly distributed between 0 and 40 sample/sec. 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 under both N-S and E-W forcing conditions as well as under torsion. The results, together with modal data from forced vibrations are presented in Figs. 4.4 through 4.8.

TABLE 4.1 - RESONANT FREQUENCIES (cps)

Excitation	Frequency
N-S	2.24
E-W	1.82
E-W	2.14
Torsional	1.82
Torsional	2.14

4.3.4 Damping

In the case of forced vibration study, damping in the structure can be determined by the bandwidth method or by measuring a free vibration decay response. In ambient vibration studies only the first method can be used, provided that wind excitations are random and stationary in time (12). Using the bandwidth method provided the dampings ratios as presented in Table 4.2.

TABLE 4.2 - DAMPING RATIOS

Excitations	Damping Factors
N-S (2.24 cps)	0.7 %
E-W (1.82 cps)	0.9 %
E-W (2.14 cps)	0.7 %
Torsional (1.82 cps)	1.2 %
Torsional (1.82 cps)	0.8 %

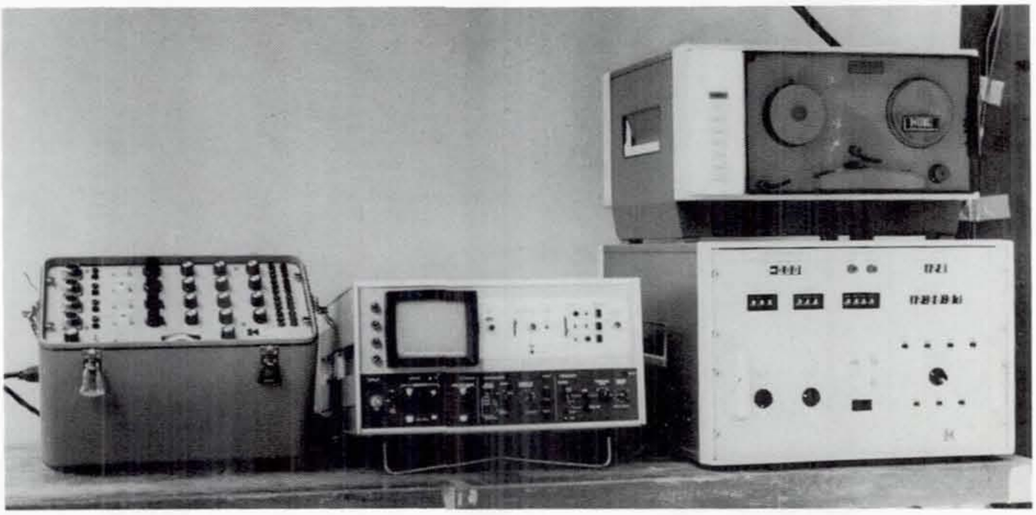


FIG. 4.1 AMBIENT VIBRATION EQUIPMENT

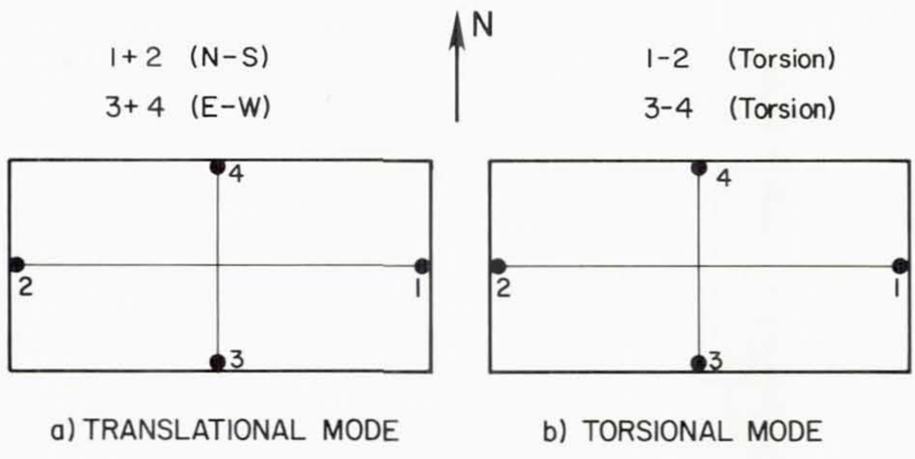


FIG. 4.2 LOCATION OF RANGER SEISMOMETERS ON THE 12TH FLOOR FOR RESONANT FREQUENCY RESPONSE

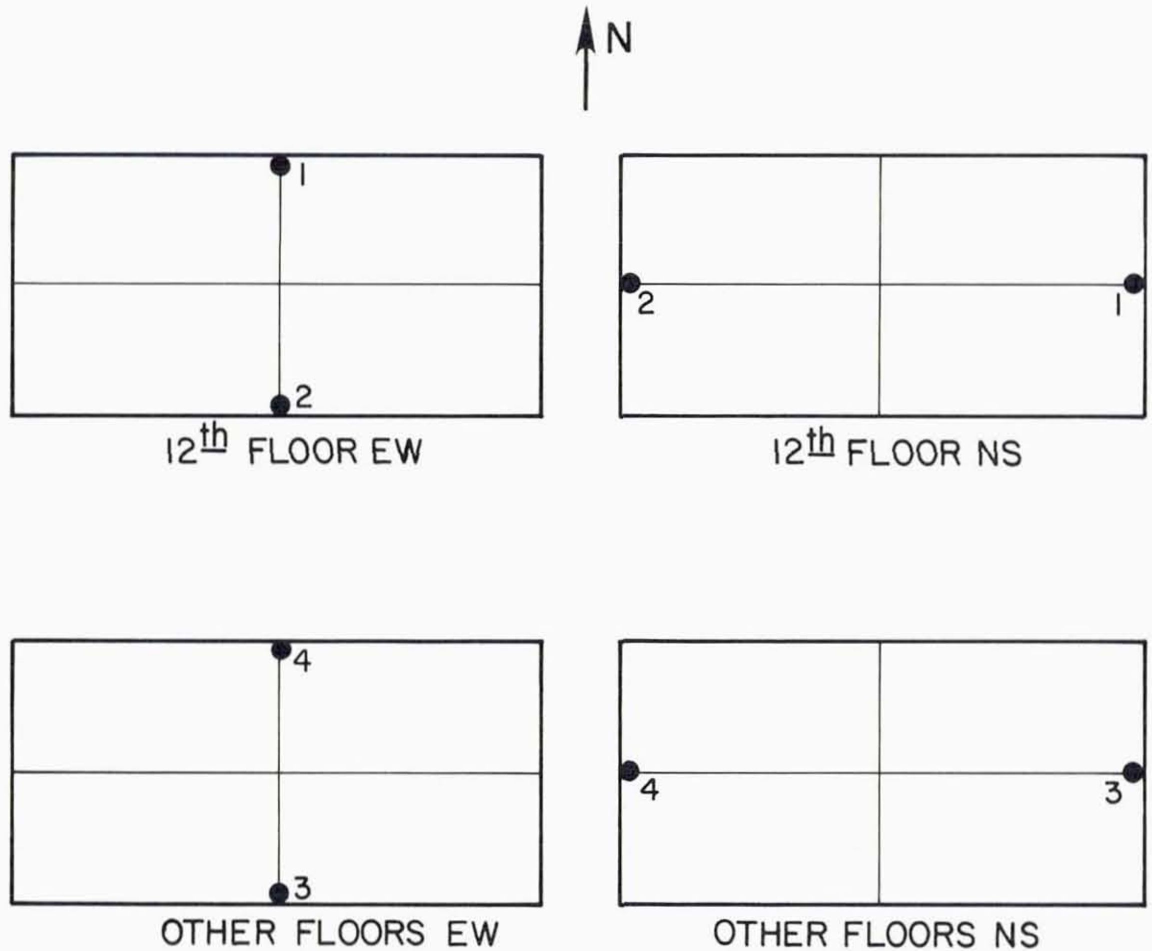


FIG. 4.3 LOCATION OF RANGER SEISMOMETERS FOR THE MODE SHAPES

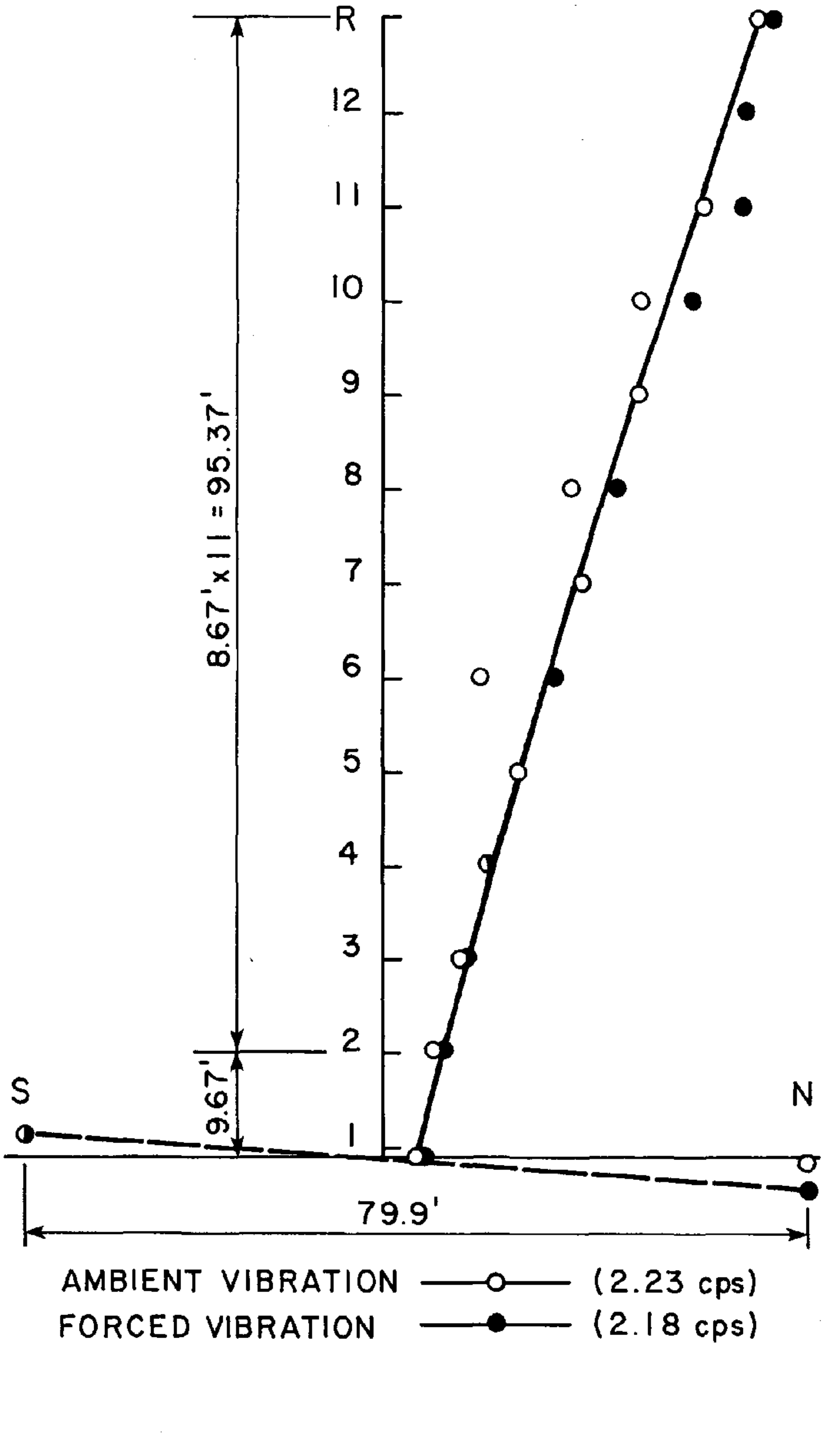


FIG. 4.4 NS VERTICAL MODE SHAPE (2.23 CPS)

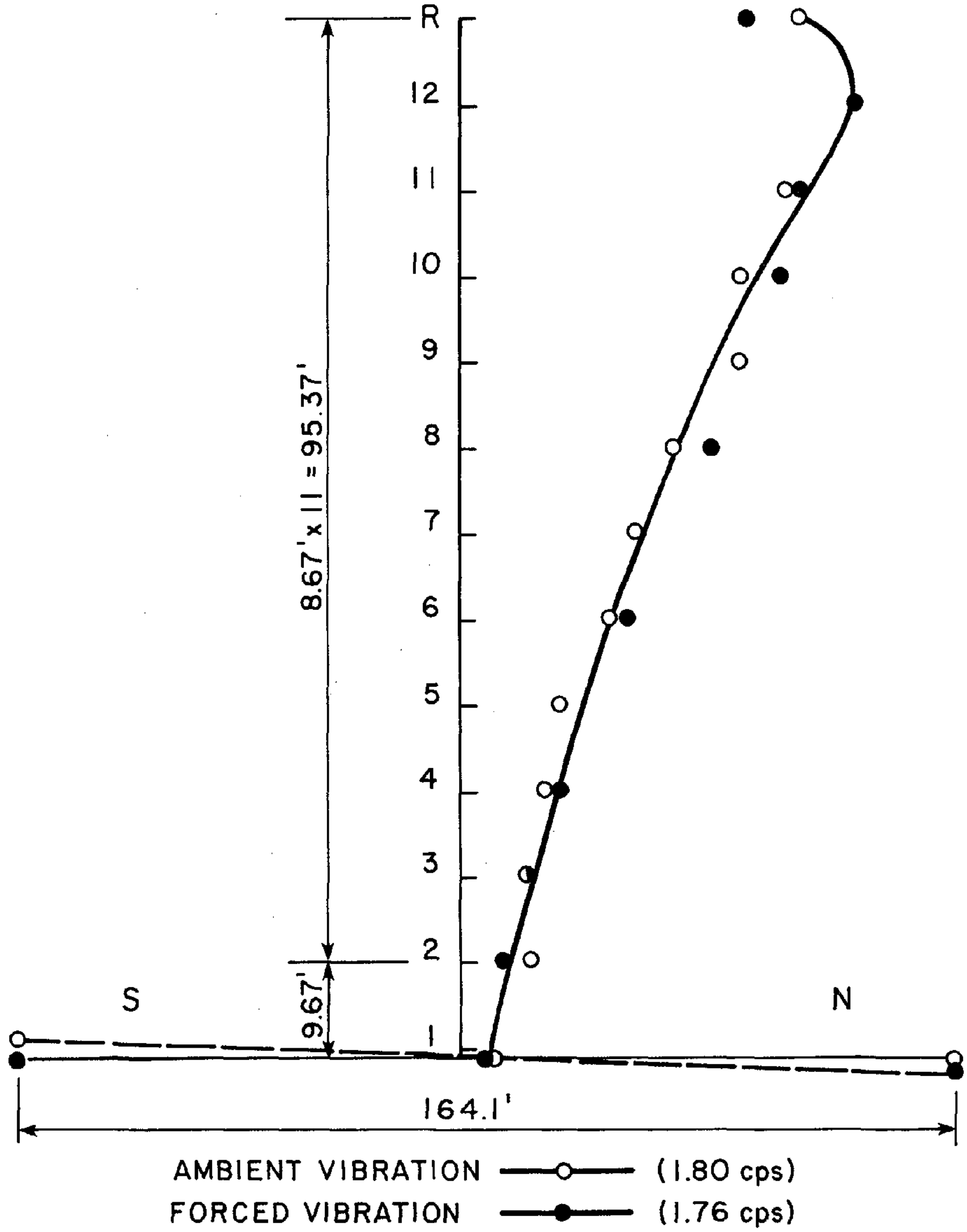


FIG. 4.5 EW VERTICAL MODE SHAPE (1.80 CPS)

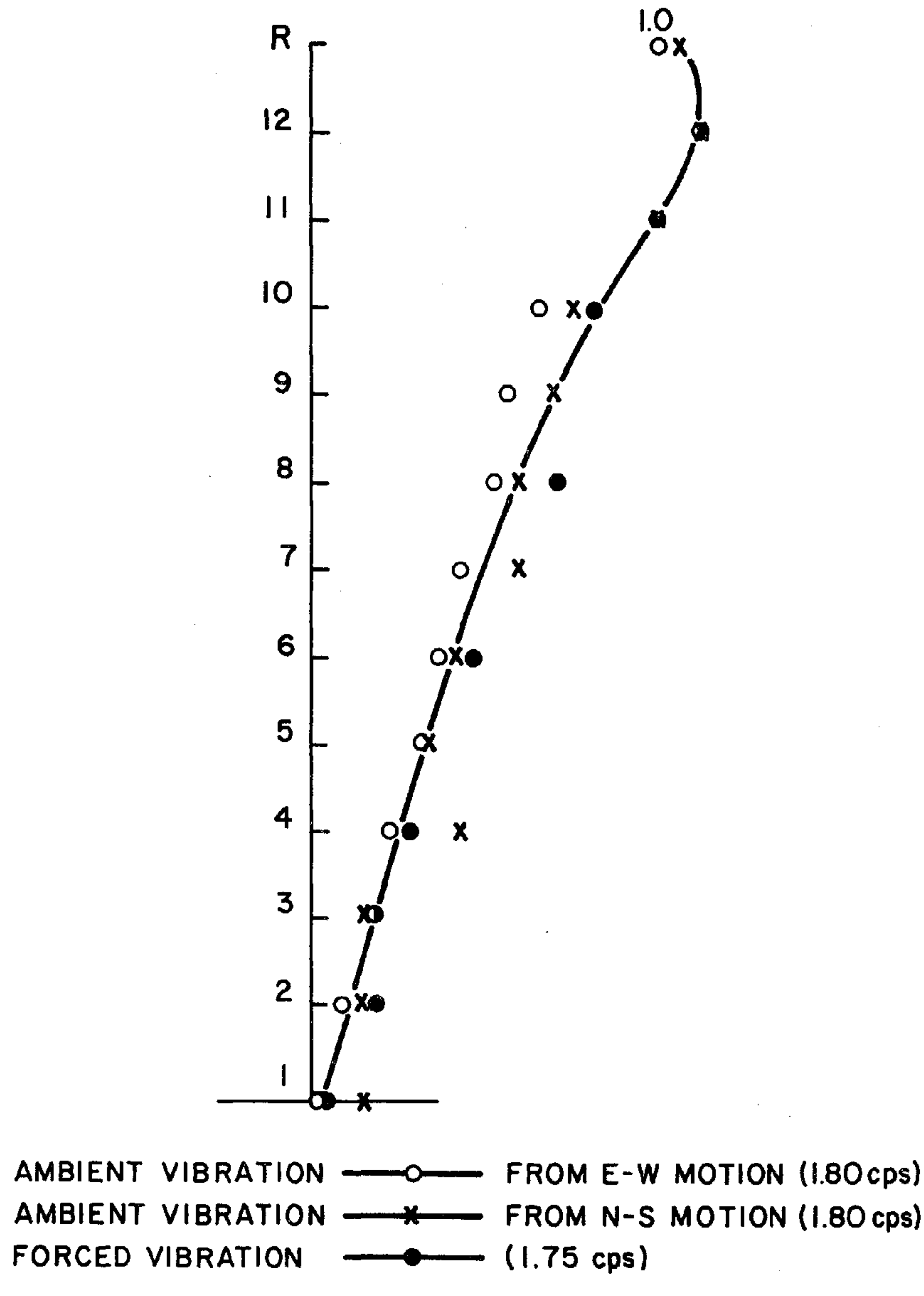


FIG. 4.6 TORSIONAL VERTICAL MODE SHAPE (1.80 CPS)

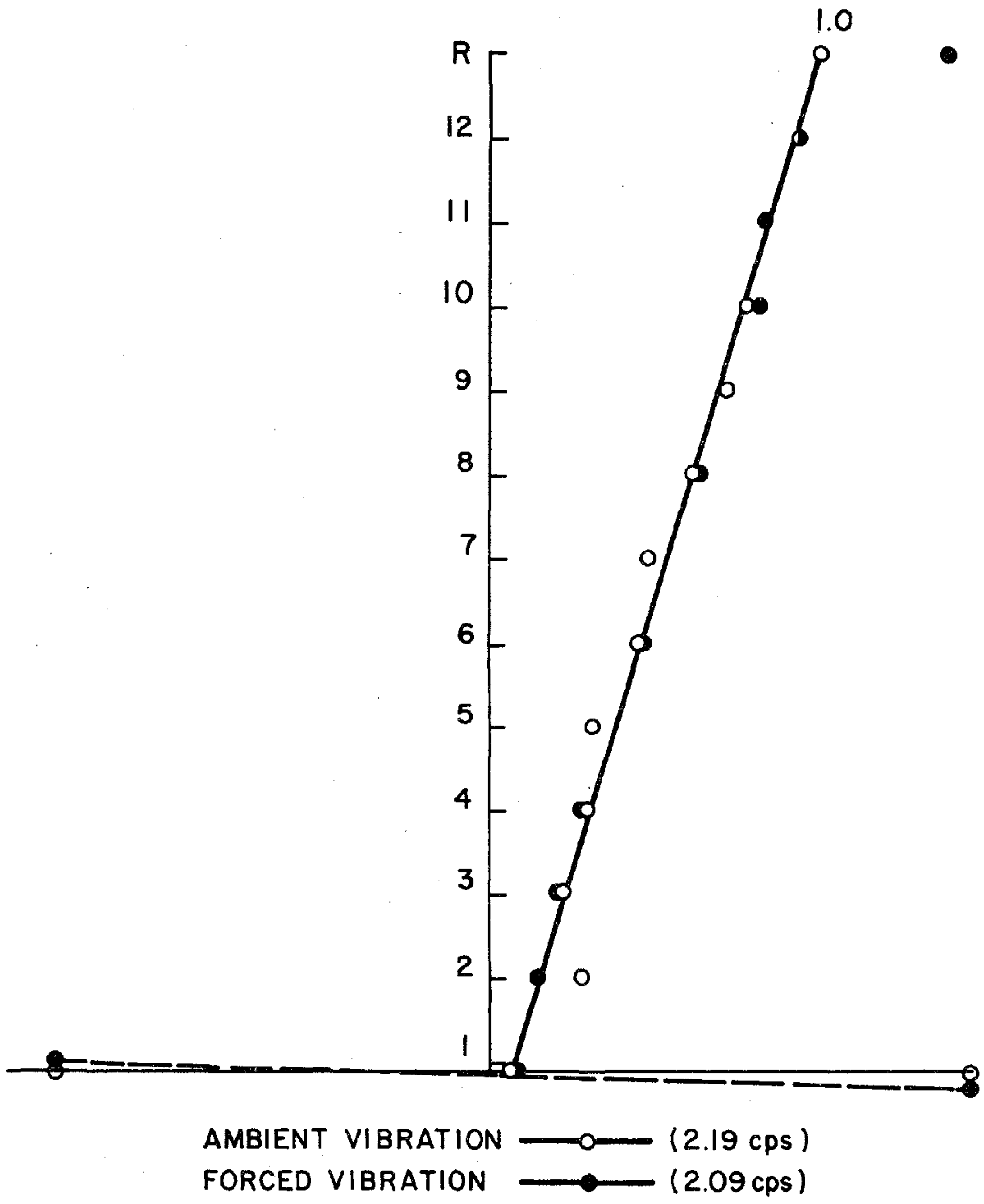


FIG. 4.7 TORSIONAL VERTICAL MODE SHAPE (2.19 CPS)

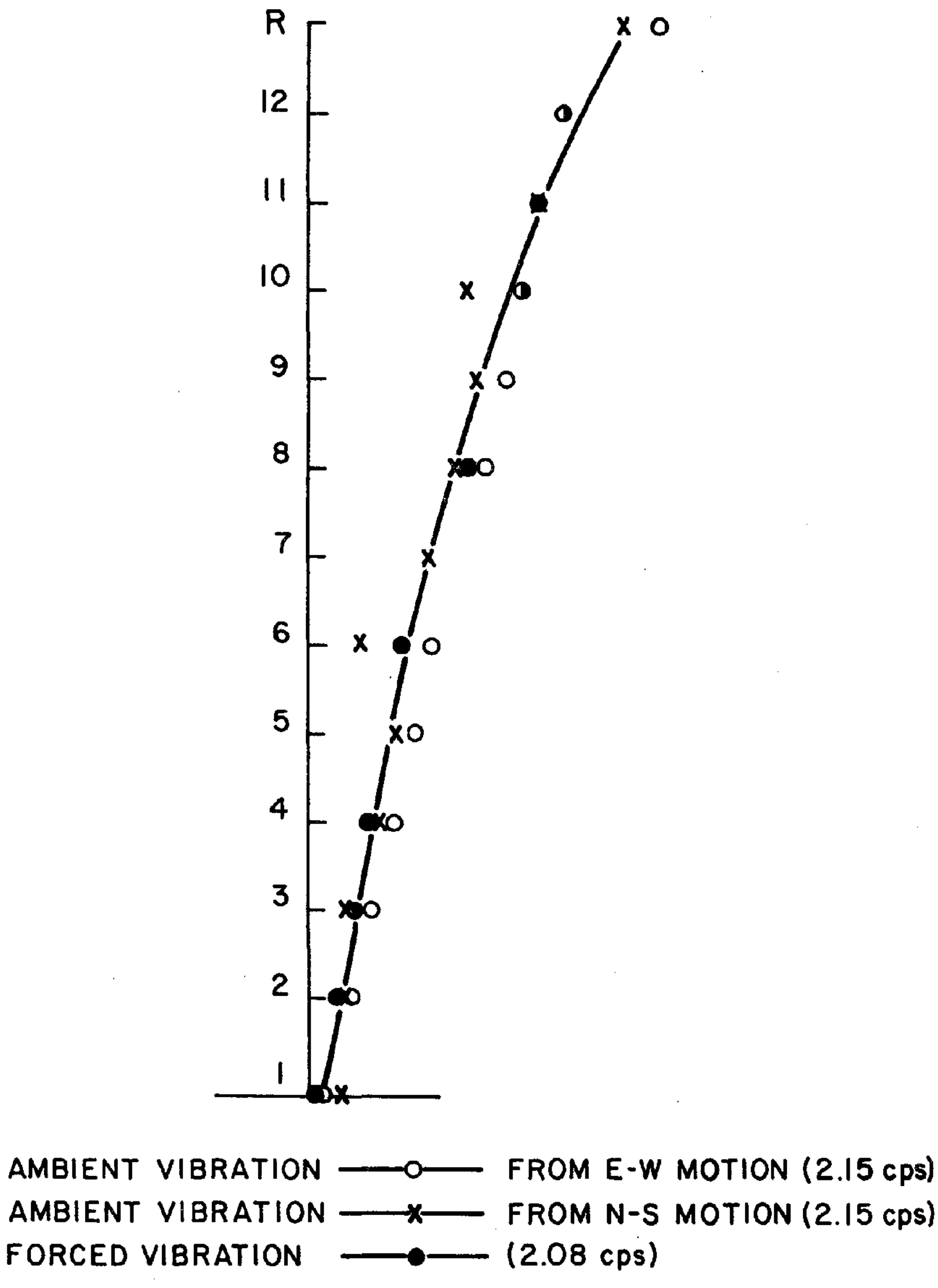


FIG. 4.8 TORSIONAL VERTICAL MODE SHAPE (2.15 CPS)

5. COMPARISON OF FORCED AND AMBIENT VIBRATION STUDIES

The dynamic properties (resonant frequencies, modes of vibration and damping values) of the Wesley Manor Building were determined by a full-scale dynamic test 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 test are 2 to 4% smaller than the resonant frequencies from the ambient vibration test. This nonlinear effect reflects the greater effective stiffness under small displacements. The same basic effects can be noted from the frequency shifts under low levels of excitation, as illustrated in Figs. 3.4 and 3.6.

Equivalent viscous damping factors obtained from the two vibration studies show some difference, indicating basically larger damping values under increasing loads (or displacements). However, the differences are not pronounced. In general, the results can be viewed more appropriately as an indication of the range of damping, rather than as specific damping values associated with each mode.

Mode shapes associated with translational (E-W and N-S) and torsional excitations are compared in Figs. 4.4 through 4.8. The results from both studies show good agreement.

TABLE 5.1 COMPARISON OF RESONANT FREQUENCIES AND DAMPING RATIOS

Mode	Forced Vibration			Ambient Vibration	
	Frequency (cps)	Damping Ratios from		Frequency (cps)	Damping Ratios
		decay curve	freq. response curve		
NS	2.18	1.2 %	2.2 %	2.24	0.7 %
EW	1.76	- - -	1.4 %	1.82	0.9 %
EW	2.09	1.8 %	1.1 %	2.14	0.7 %
Tors.	1.75	2.8 %	1.4 %	1.82	1.2 %
Tors.	2.09	- - -	1.3 %	2.14	0.8 %

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 frequence response data can not be generated under ambient conditions, fewer measurements are required in such studies. Furthermore, each measurement requires less time. On the other hand, data analysis is slightly more complicate because of computer use for Fourier analysis.

6. FORMULATION OF MATHEMATICAL MODEL

6.1 General

A mathematical computer model of the Wesley Manor 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 analysis 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 Modeling of Structure

The basic model of the building was formulated as a system of frames and shear wall elements interconnected by floor diaphagms 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 Fig. 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 5/6A, 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 distances 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 the 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.

6.4 Foundation Modeling

Initial analytical studies, assuming a rigid base, showed 30 to 50% larger resonance frequencies than those obtained experimentally. Hence, it was essential to account for the actual foundation stiffness. Two different ways of obtaining stiffness values for the foundation, one using vibration test data and the other using available soil data are described in the following sections.

6.4.1 Vibration Test Data Process

The computer program TABS, used to compute the dynamic properties of the structure, does not permit the input of 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 rotation

r_v = base displacement,

EI = flexural rigidity of dummy story, and

L = height of dummy story.

With estimated constant floor masses of 53 kips $sec^2/ft.$ for every story and the measured floor accelerations for the two fundamental translational 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 NS and coupled EW/torsional modes are presented in Table 6.1.

TABLE 6.1 DUMMY STORY PROPERTIES

Mode	Frequency (cps)	L(ft)	EI(kp.ft^2)
N-S	2.18	32.27	$3.662*10^{10}$
E-W/Torsion	1.76	23.86	$1.962*10^{10}$
E-W/Torsion	2.09	37.95	$5.356*10^{10}$

As the dummy story height for both NS and EW directions has to be the same, an optimum dummy story element had to be developed. Following several alternatives, the length of the dummy story obtained from the uncoupled NS mode was chosen for the analysis. Consequently the stiffness values for the other modes had to be modified. The EI values were scaled by (L_0/L), where L_0 = 32.27 ft., thus setting the lateral stiffness right but changing the rotational stiffness. The stiffnesses for the dummy as computed and actually used in the model are summarized in Table 6.2. Using the dummy story properties very good agreement between experimentally measured and analytically computer frequencies and mode shapes could be achieved. But since the prediction of the dynamic properties of a structure is essential for a good design against earthquake loading, an attempt was made to model the foundation using available information about the soil.

TABLE 6.2 LATERAL AND ROTATIONAL SOIL STIFFNESS

	From Experimental Data	From Soil Data
N-S (2.18 cps)		
$K_{ROT} = \frac{4EI}{L}$	4.539×10^{12} $\left[\frac{lb\ ft}{rad}\right]$	4.438×10^{12} $\left[\frac{lb\ ft}{rad}\right]$
$K_{LAT} = \frac{12EI}{L^3}$	1.308×10^{10} $\left[\frac{lb}{ft}\right]$	0.401×10^{10} $\left[\frac{lb}{ft}\right]$ *Soil Resistance
E-W (1.76 cps)		
$K_{ROT} = \frac{4EI}{L}$	3.291×10^{12} $\left[\frac{lb\ ft}{rad}\right]$	19.277×10^{12} - rigid
E-W (2.09 cps)		2.882×10^{12} - pile groups I,II,III
$K_{ROT} = \frac{4EI}{L}$	5.645×10^{12} $\left[\frac{lb\ ft}{rad}\right]$	12.7938×10^{12} - flexible structure
Actually used in the Model	6.020×10^{12} $\left[\frac{lb\ ft}{rad}\right]$	
E-W (1.76 cps)		
$K_{LAT} = \frac{12EI}{L^3}$	1.734×10^{10} $\left[\frac{lb}{ft}\right]$	0.401×10^{10} $\left[\frac{lb}{ft}\right]$ *Soil Resistance
E-W (2.09 cps)		
$K_{LAT} = \frac{12EI}{L^3}$	1.176×10^{10} $\left[\frac{lb}{ft}\right]$	
Actually used in the Model	1.734×10^{10} $\left[\frac{lb}{ft}\right]$	

6.4.2 Soil Data Process

The dynamic modulus of elasticity of the soil was determined by a seismic downhole survey in boring 2 at the site (Fig. 6.4). The propagation velocity of the shear waves was measured and together with the information about the dry density used in the formula $v = \sqrt{G*g/\gamma}$ (18), where

v = velocity of the shear waves (ft/sec)

γ = dry density (lbs/ft^3)

g = constant of gravity (ft/sec^2)

G = shear modulus of the soil (lbs/ft^2)

The increase of G with depth is shown in Fig. 6.5. A Poisson's ratio of 0.3 was selected for the soil. It has to be noted that the shear modulus decreases with higher shearing deformation and also depends on the frequency of the loading. In this study constant values for G and v are used. The structure is founded on piles as shown in detail in Fig. 6.6.

6.4.2a Vertical Stiffness

Since the piles are not resting on bedrock and the displacements necessary to obtain point resistance are much larger than the displacements for the development of skin resistance the point resistance was assumed to be zero. A second assumption had to be made about the variation of the skin friction along the pile. A triangular variation of skin friction with the maximum at the top and zero at the pile point, yielding a parabolic axial force distribution in the pile (see Fig. 6.7), was found to be in agreement with experimental data (19) and previous modeling experience (20). With these assumptions the stiffness K_p for each pile type could be calculated.

Because the pile transmits loads to the soil through friction, the skin forces of the pile were applied to a finite element model of the soil directly, rather than using a model of a pile-soil system and enforcing displacement continuity at the nodes. The resulting axisymmetric 72 element model is shown in Fig. 6.8, with the element G values based on the shear modulus data presented in Fig. 6.5. The computer program FEAP (21) was used in this analysis. The calculated displacement of the soil at the pile top permitted an estimate of the soil stiffness K_s. The soil and pile stiffness were combined to give the stiffness of the pile-soil system for one pile in the vertical direction by adding the flexibilities, $1/K_v = 1/K_p + 1/K_s$. Table 6.3 summarizes the obtained values for the vertical stiffness (K_v). The total vertical stiffness of the foundation is the sum of the individual pile stiffnesses.

TABLE 6.3 VERTICAL PILE STIFFNESSES

Pile Length (ft)	K_p (lbs/ft)	K_s (lbs/ft)	K_v (lbs/ft)
29.5	19.56×10^7	4.35×10^7	3.56×10^7
36.0	16.02×10^7	5.06×10^7	3.84×10^7
41.5	13.24×10^7	5.66×10^7	3.97×10^7
47.0	11.92×10^7	6.25×10^7	4.10×10^7
52.5	10.76×10^7	6.85×10^7	4.19×10^7

6.4.2b Rotational Stiffness

The coordinates K_o, Y_o of the resulting vertical stiffness of the entire 204 pile foundation were calculated using standard procedures (i.e., $Y_o = (\Sigma K_{vi} \times Y_i)\ (\ \Sigma\ K_{vi})$. Assuming that the pile-cap foundation plane remains plane, or effectively the building acts as a rigid system, the rotational stiffnesses of the entire foundation with respect to the NS and EW directions can be determined (i.e., $K_{ROT-NS} = \Sigma\ K_{vi}\ \overline{X}_i^2$, where $\overline{X}_i$ is the distance in the EW direction between pile i and the center of stiffness), Table 6.2 shows the results of these analyses and indicates for the NS direction an excellent agreement between the calculated rotational stiffness and the experimentally determined value. However, the calculated rotational stiffness for the longer side of the building, in E-W direction, differs by a factor of about 6 from the experimental value. This result indicates that the assumption of a rigid system is not true for the longer and more flexible direction of the structure.

In order to attempt to capture the effective foundation stiffness several different stiffness assumptions were considered. For instance, rather than considering the foundation as one single pile system, three separate totally uncoupled rigid pile groups were considered as an alternative system (see Fig. 6.6). In that case the combined EW rotational stiffness was found to be in relatively close agreement with the experimental value; namely, 2.88×10^{12} lb.ft/rad versus 3.30×10^{12} lb.ft/rad, respectively. Of course using the separate three pile group approach for calculating the NS rotational stiffness, the difference with the rigid approach is minimal, and the agreement with the experimental results excellent.

A second alternative considered the entire building as a beam on an elastic foundation. Without pursuing a detailed analysis, the building was modeled as a simple beam, supported at the center of each of the earlier noted three pile groups (see Fig. 6.9). The rotational stiffness in this case, was found to be a factor of 4 too large. A more refined model, considering complete vertical shear deformation of the building and discrete vertical pile stiffnesses may lead to a better correlation. However, such an approach was considered beyond the basic scope of this investigation.

6.4.2c Lateral Stiffness

The later stiffness of a single pile was obtained by solving the equation $E_p I_p \frac{d^4u}{dz^4} = -p(z)$, where

$E_p I_p$ = flexural regidity of the pile,

u = lateral deflection,

$p(z)$ = reactive lateral pressure of the soil against the pile, and

z = depth.

This equation was solved numerically assuming that the ratio of pressure ($p(z)$) to deflection (u) fulfills: $p(z)/u = K_h = n_h \times z$ (22), where

K = soil reaction modulus, and

n = coefficient of soil reaction (empirical quantity).

The coefficient of soil reaction was determined using a formula given by Vesic (23),

$$n_h = \frac{1.3}{B} \sqrt[12]{\frac{E_s B^4}{E_p I_p}} \quad \frac{E_s}{1-v^2}$$

B = diameter of the pile

E_s = modulus of elasticity of the soil

v = Poisson's ratio for the soil (=0.3).

According to this assumption the soil reaction modulus K_h increases with depth as shown in Fig. 6.10. The pile deflections for both a rotationally free and rotationally fixed pile head are shown in Fig. 6.11 and indicate that the lateral deflections are almost zero below the 8 foot level. Thus, it can be concluded that, in this case, the lateral pile stiffness only depends on the soil properties of the uppermost layer.

Summing the lateral stiffness of all piles results in a total lateral stiffness K_{LAT} of 0.401 x 10^{10} lb/ft, as noted in Table 6.2. This stiffness is only about 25 to 30% of the experimentally derived values. This result is not surprising as the total lateral stiffness is actually a result of not only the lateral pile resistance but also, and seemingly more important, of the resistance provided by the soil against the pile-cap beams and by the possible friction between soil and floor slab.

6.5 Analytical Results

The final analytical model used in the correlative analyses had a simple 32'-4" high dummy story as discussed previously. The analytical results and the experimental data for both forced and ambient vibration tests are presented in Table 6.4. Also presented are the analytical results assuming a rigid base.

TABLE 6.4 EXPERIMENTAL AND ANALYTICAL FREQUENCIES (cps)

Forcing Direction	Experiment		Analysis		Code
	Forced Vibration	Ambient Vibration	Rigid Base	Flexible Foundation	
E-W/Torsion	1.76	1.82	2.27	1.71	- - -
E-W/Torsion	2.09	2.14	2.72	2.08	2.44
N-S	2.18	2.24	3.14	2.19	1.69

The results for the flexible base condition agree very well with the experimental forced-vibration data. The slightly higher frequencies for the ambient vibration results clearly indicates a foundation non-linearity. In fact a softening of the foumdation modes increased forcing and displacements levels as noted, could be expected.

For each of the three basic resonance frequencies the vertical center-line mode shapes for both the experimental and analytical results are shown in Figs. 6.12 through 6.14. The horizontal mode shapes for the 6th and 12th floors for the same resonance conditions are presented in Figs. 6.15 through 6.17. The results show in general an excellent agreement between experimental and analytical data.

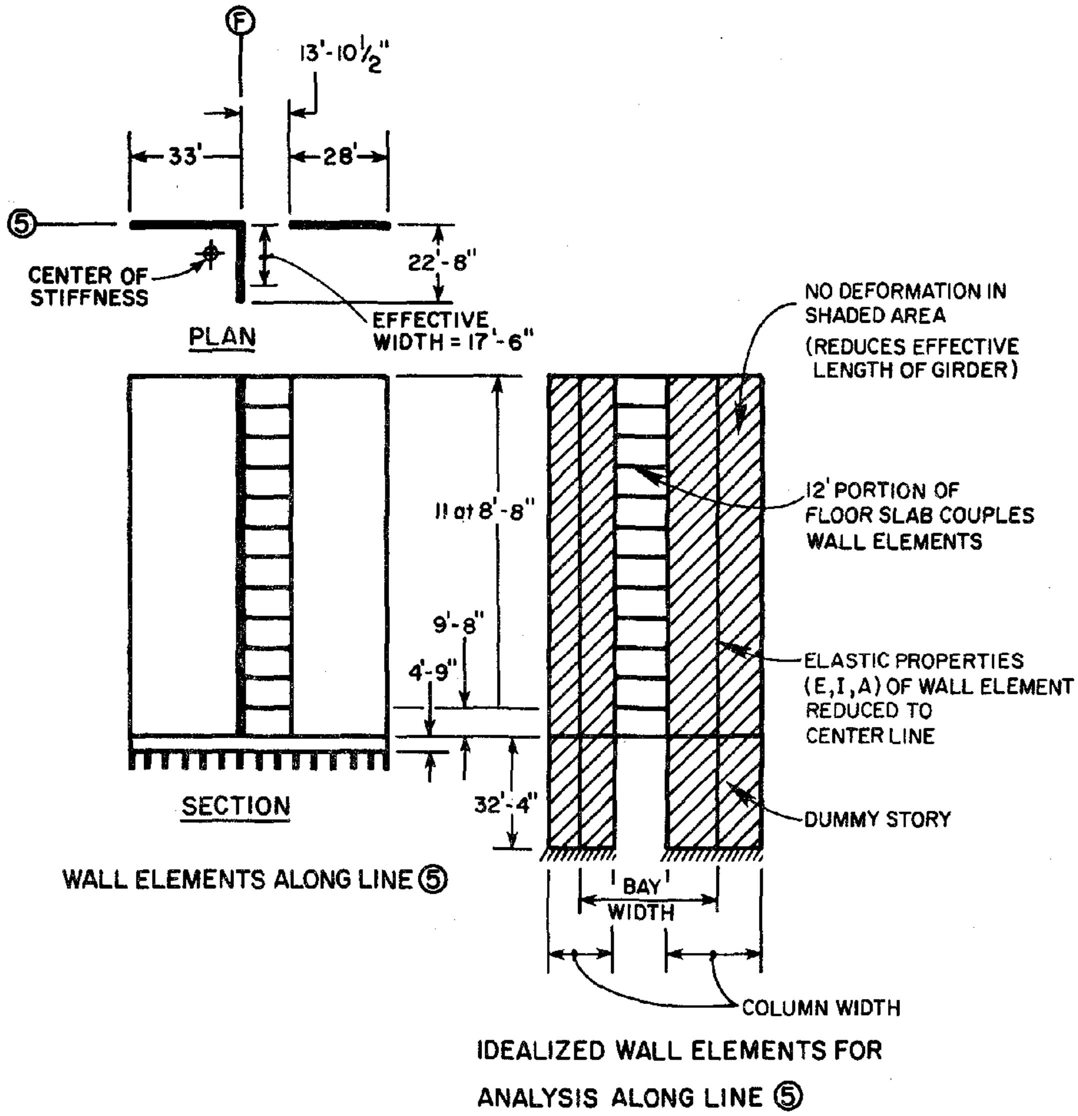


FIG. 6.1 TYPICAL WALL ELEMENT FORMULATION

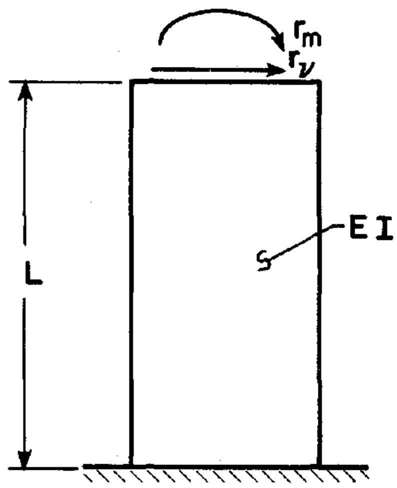


FIG. 6.2 2 DEGREE OF FREEDOM MODEL FOR THE DUMMY STORY

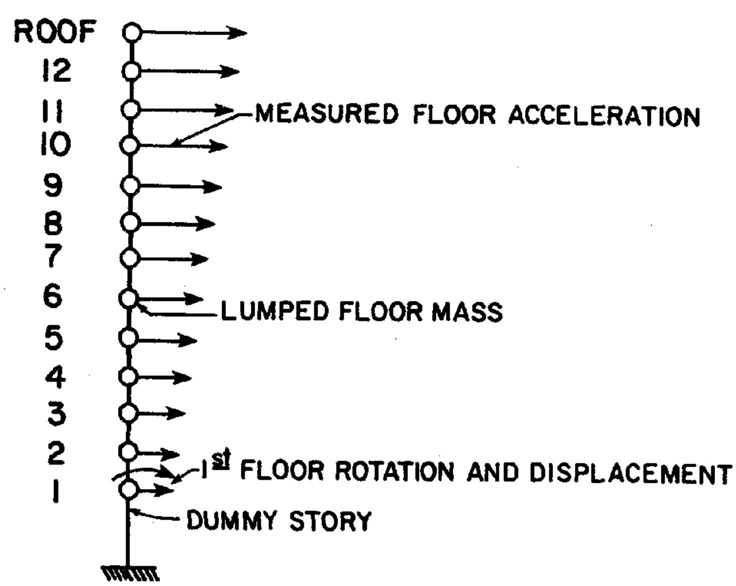


FIG. 6.3 EXPERIMENTAL DATA

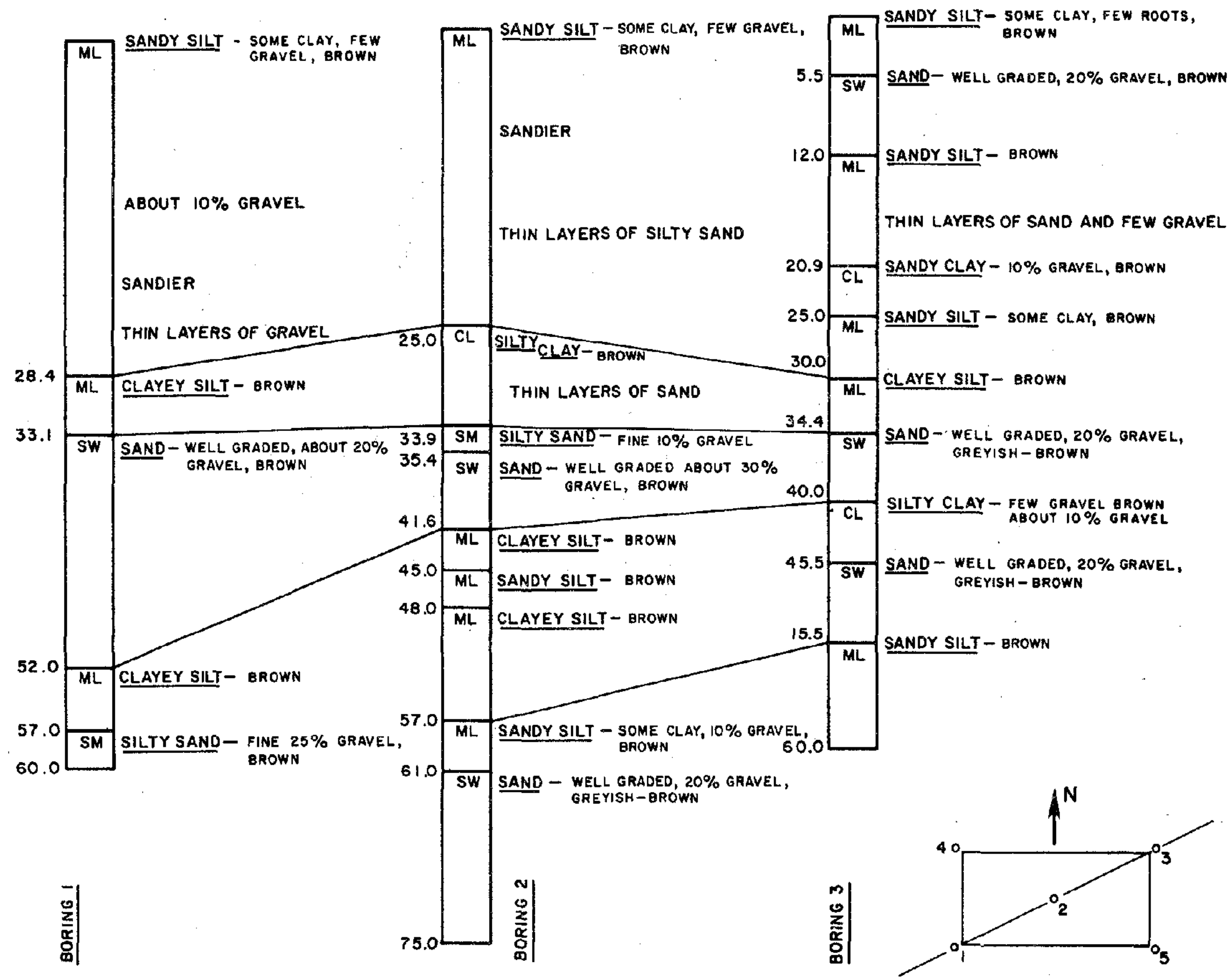
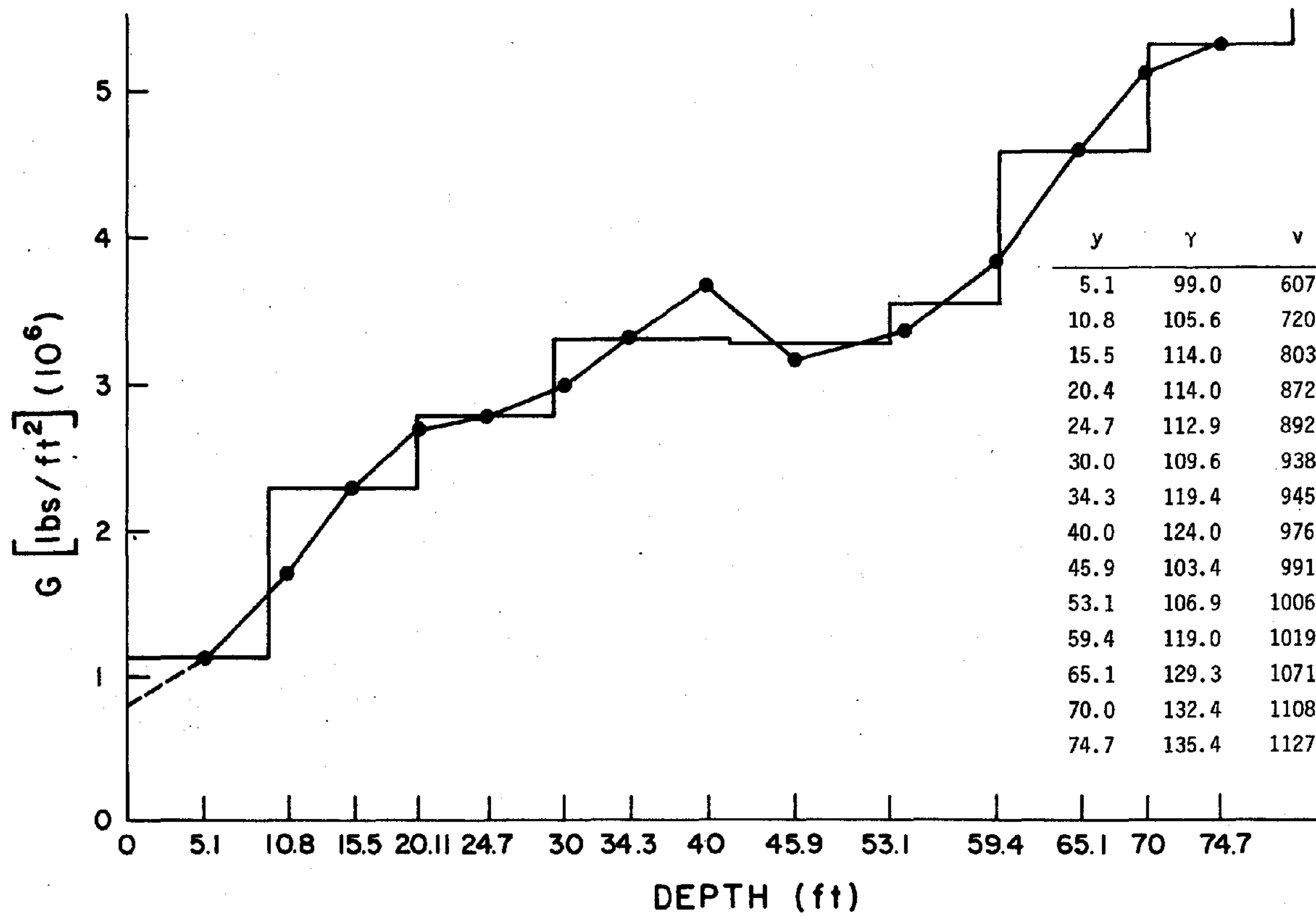


FIG. 6.4 SOIL SECTION



y	γ	v	$G \cdot v_t^2 \cdot \frac{\gamma}{g}$
5.1	99.0	607	1.134×10^6
10.8	105.6	720	1.702×10^6
15.5	114.0	803	2.285×10^6
20.4	114.0	872	2.695×10^6
24.7	112.9	892	2.792×10^6
30.0	109.6	938	2.998×10^6
34.3	119.4	945	3.314×10^6
40.0	124.0	976	3.672×10^6
45.9	103.4	991	3.157×10^6
53.1	106.9	1006	3.363×10^6
59.4	119.0	1019	3.841×10^6
65.1	129.3	1071	4.610×10^6
70.0	132.4	1108	5.053×10^6
74.7	135.4	1127	5.346×10^6

FIG. 6.5 SHEAR MODULUS OF THE SOIL

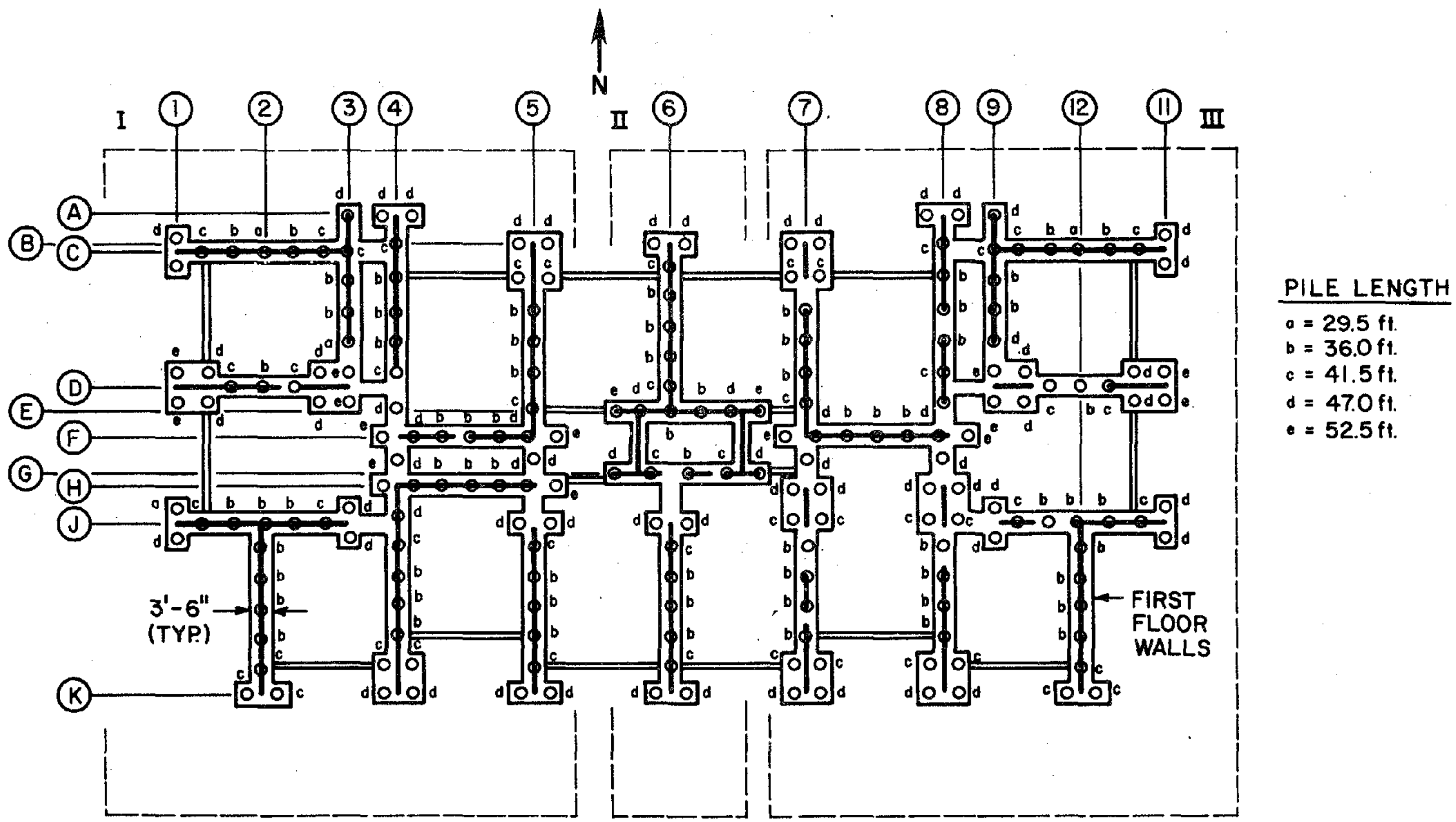


FIG. 6.6 FOUNDATION PLAN

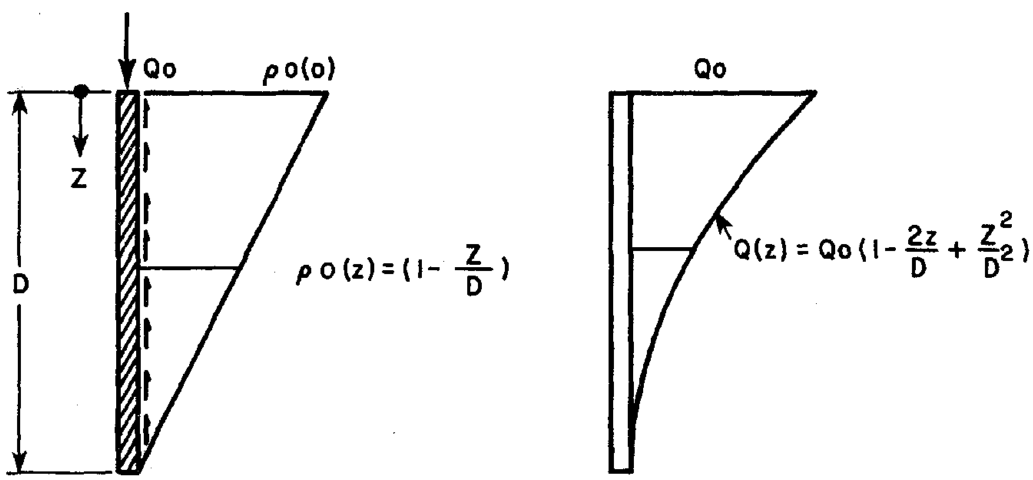


FIG. 6.7 VARIATION OF SKIN FRICTION AND AXIAL FORCE IN THE PILE

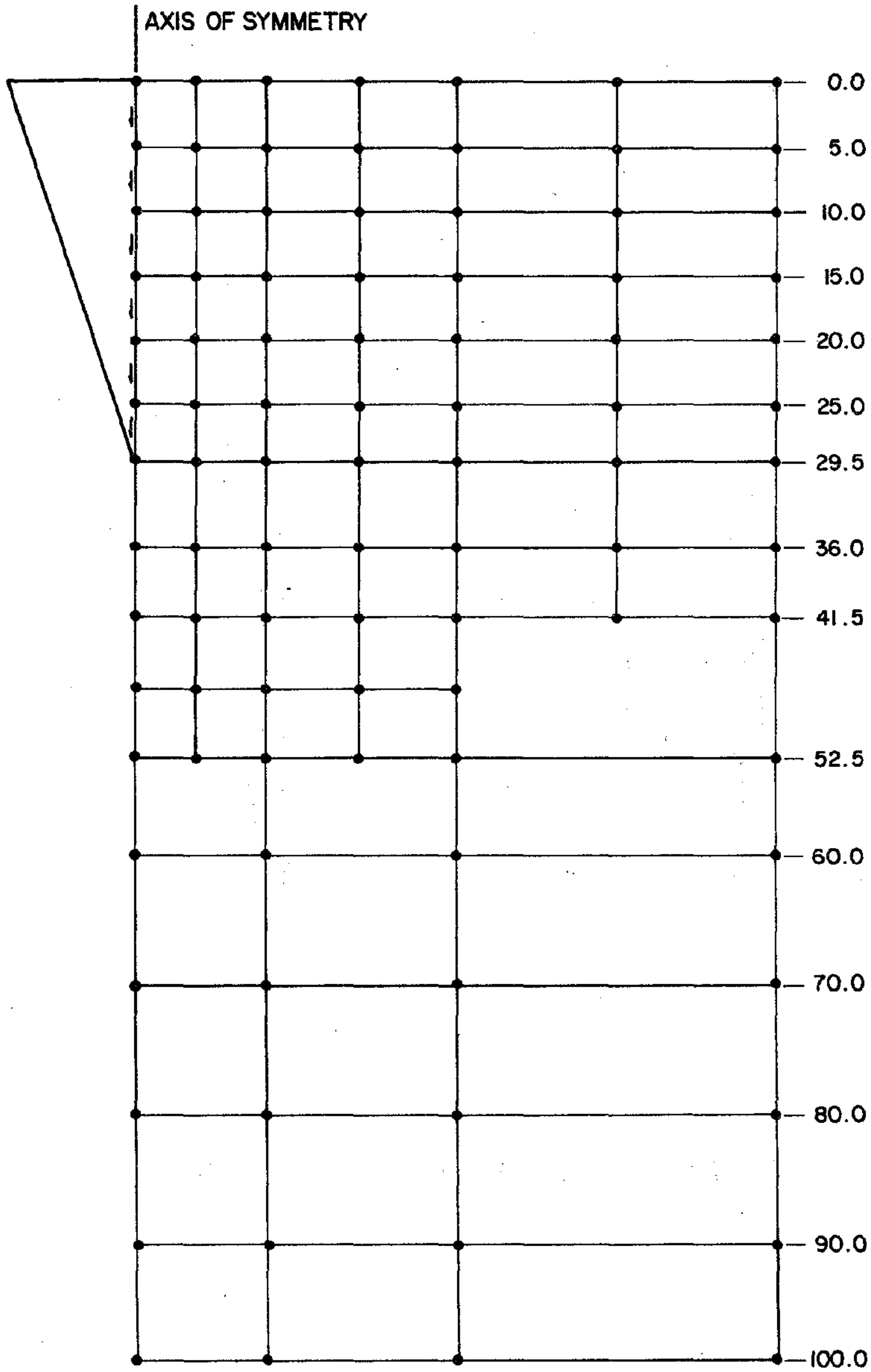


FIG. 6.8 FINITE ELEMENT MODEL OF THE SOIL

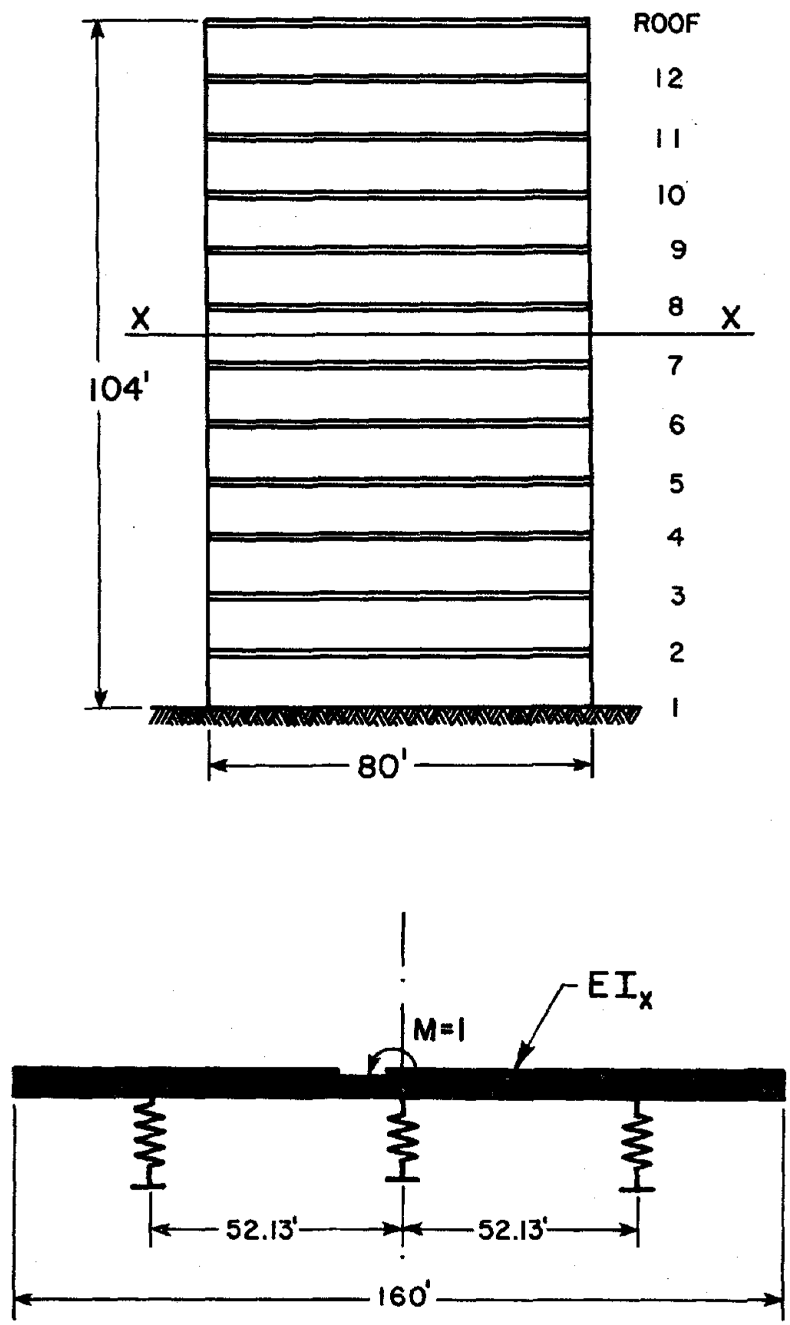


FIG. 6.9 SIMPLIFIED BEAM FOUNDATION MODEL FOR THE ROTATIONAL STIFFNESS

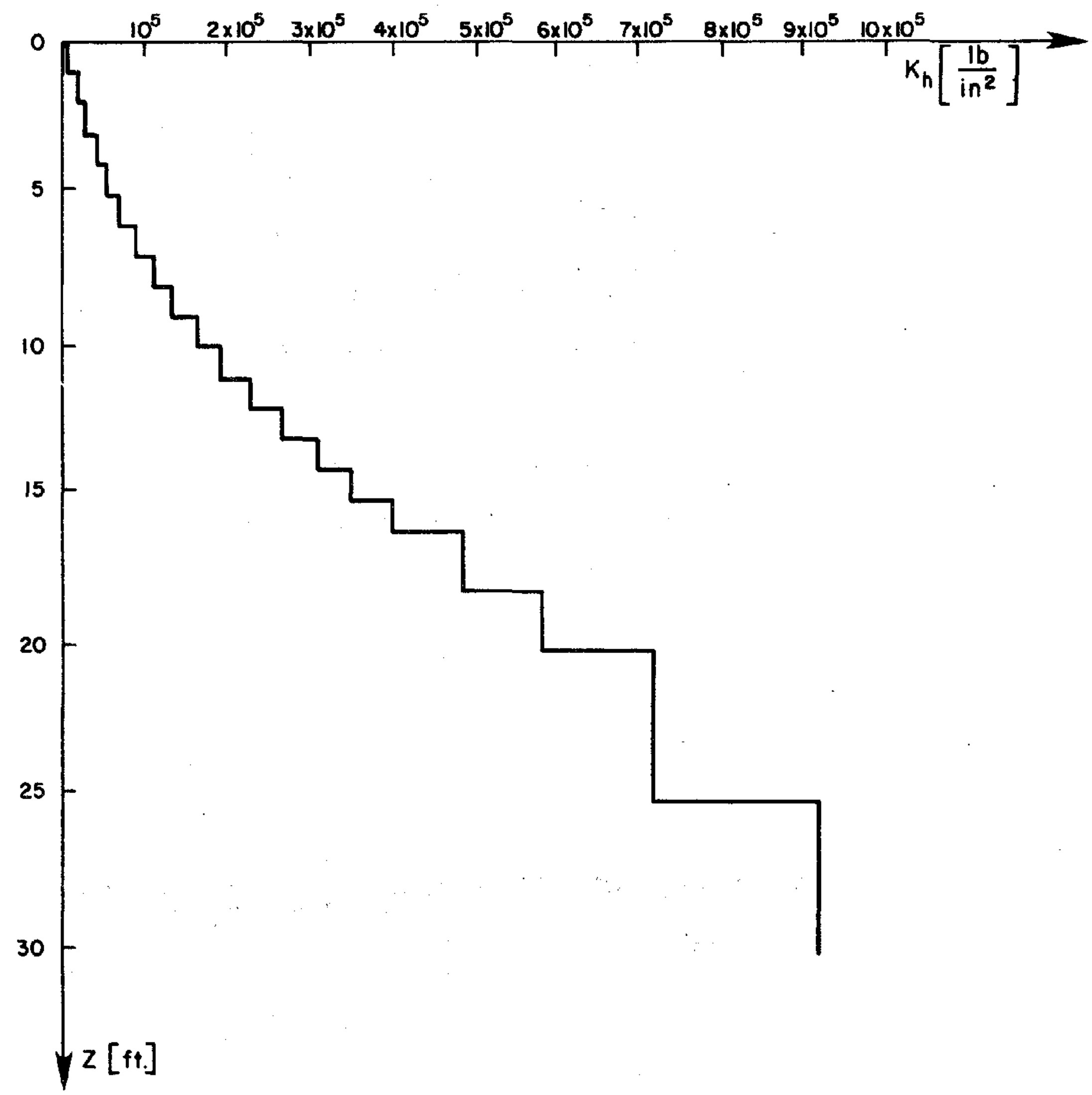
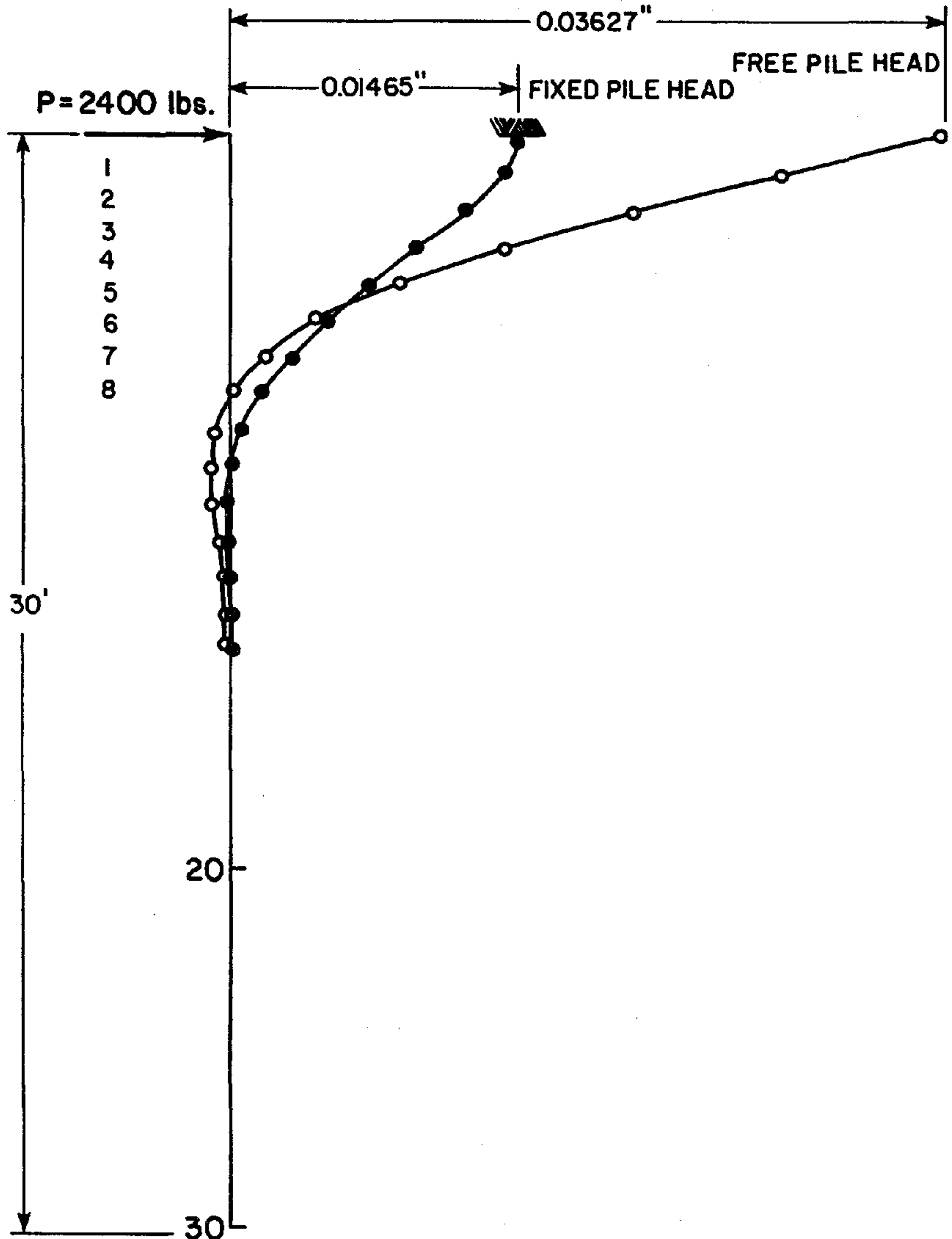


FIG. 6.10 VARIATION OF THE MODULUS OF SOIL REACTION k_n



PILE – DEFLECTIONS

FIXED PILE HEAD:

$$K_{LATERAL} = \frac{24000}{0.01465} = 1.6387 \times 10^6 \left[\frac{lb.}{in.}\right] = 1.966 \times 10^7 \left[\frac{lb.}{ft.}\right]$$

FREE PILE HEAD:

$$K_{LATERAL} = \frac{24000}{0.03626} = 0.6617 \times 10^6 \left[\frac{lb.}{in.}\right] = 0.794 \times 10^7 \left[\frac{lb.}{ft.}\right]$$

FIG. 6.11 PILE DEFLECTIONS

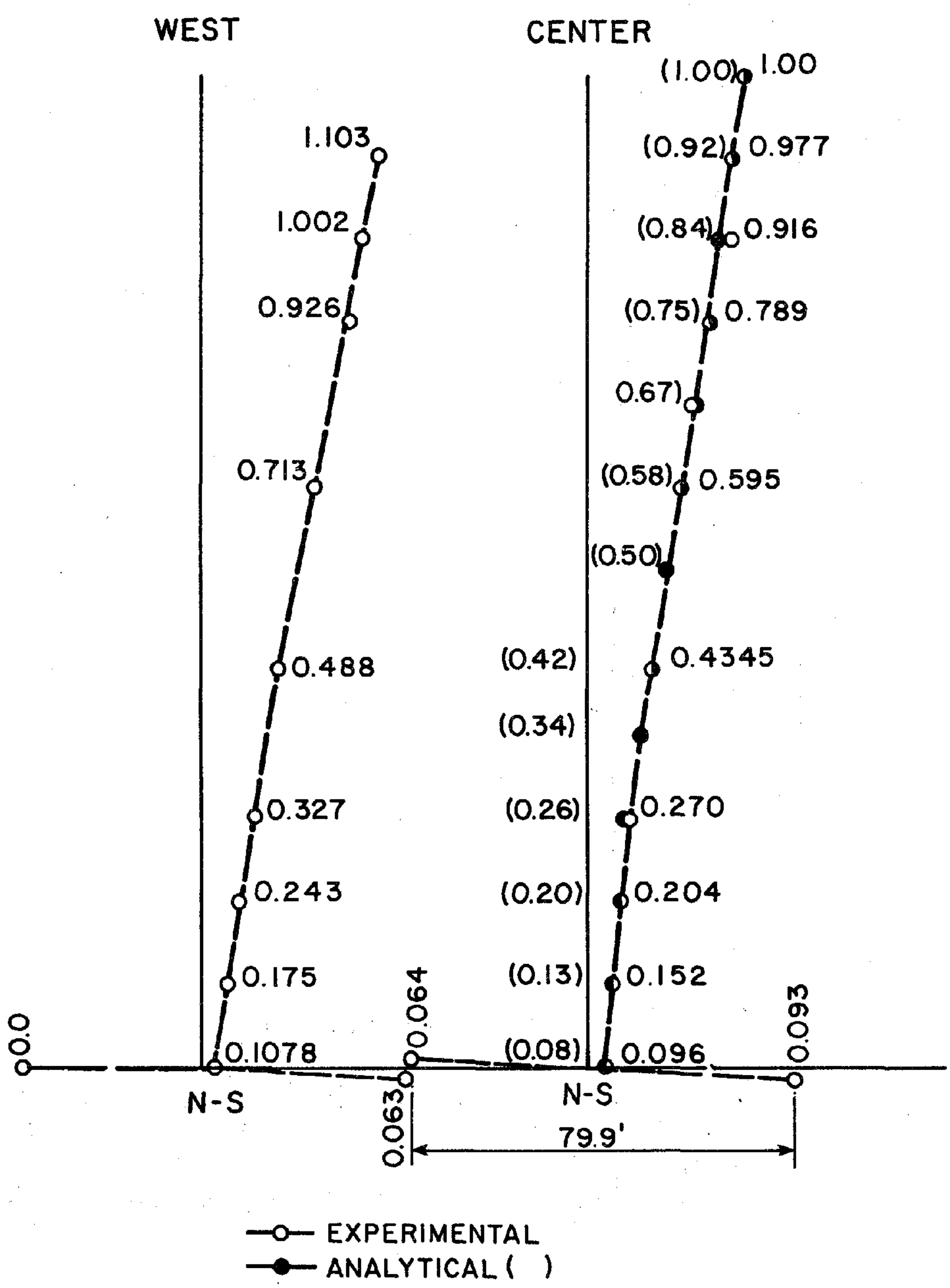


FIG. 6.12 NS VERTICAL MODE SHAPES (2.18 CPS)

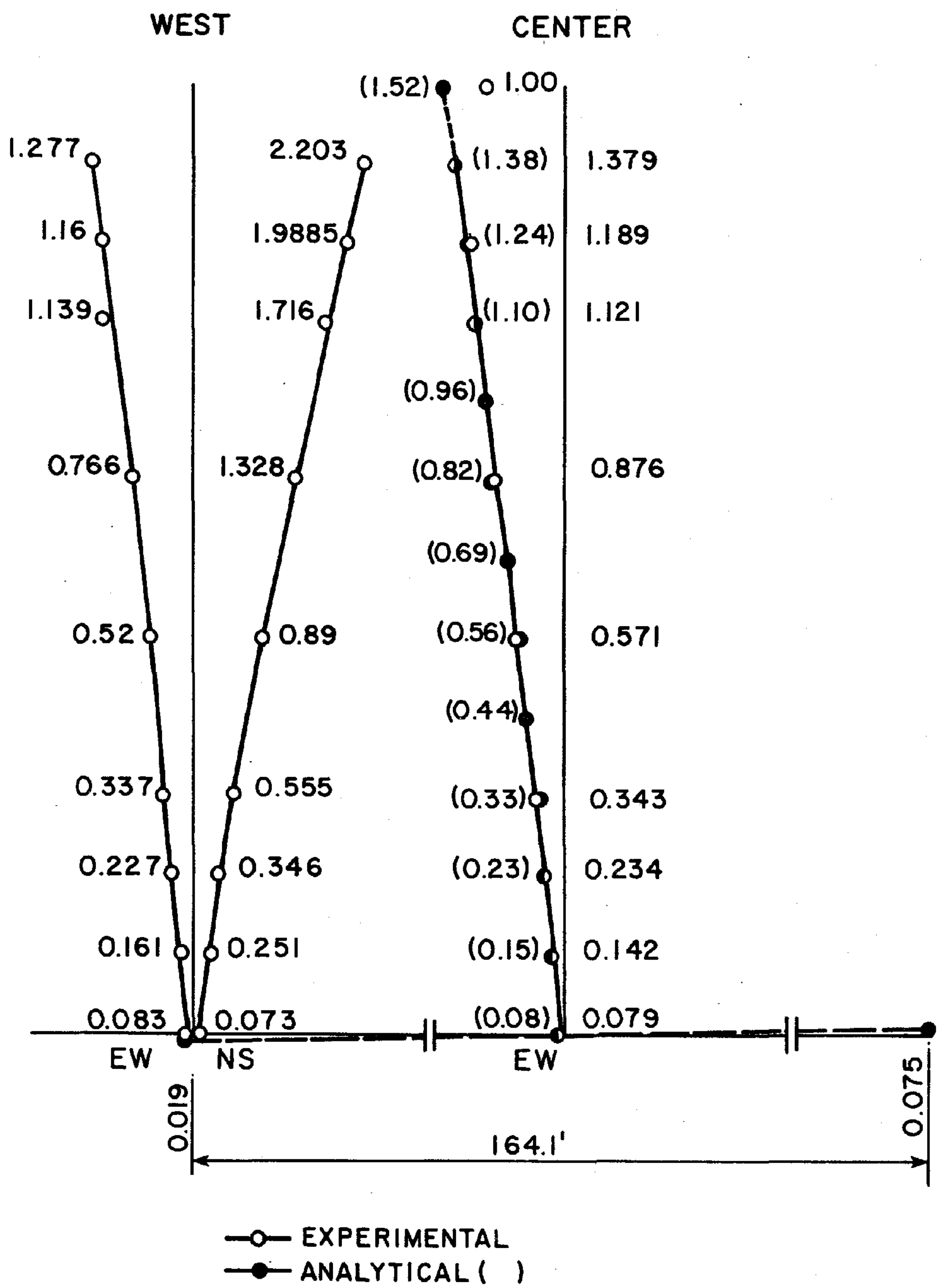


FIG. 6.13 EW VERTICAL MODE SHAPES (1.76 CPS)

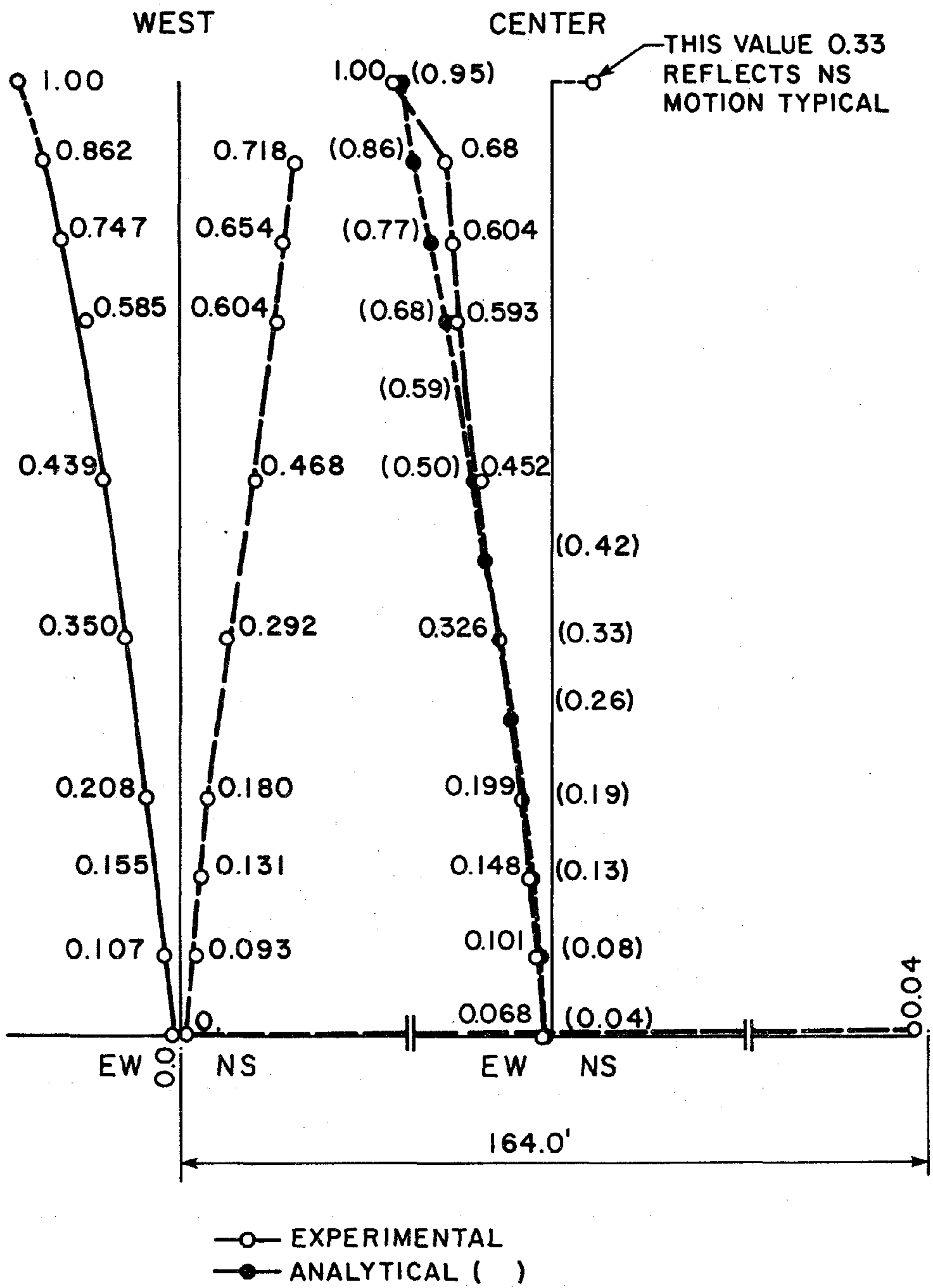


FIG. 6.14 EW VERTICAL MODE SHAPES (2.09 CPS)

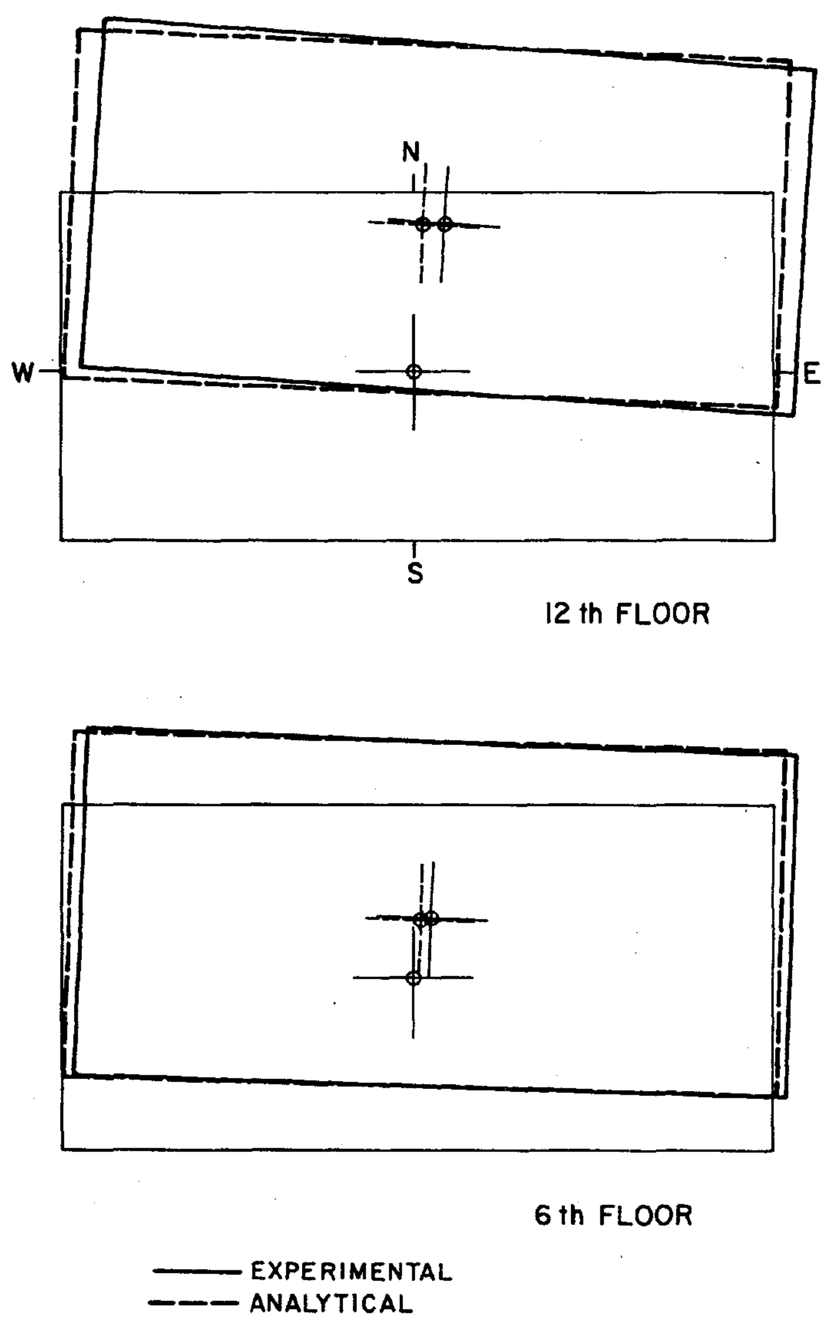


FIG. 6.15 NS FLOOR MODE SHAPES (2.18 CPS)

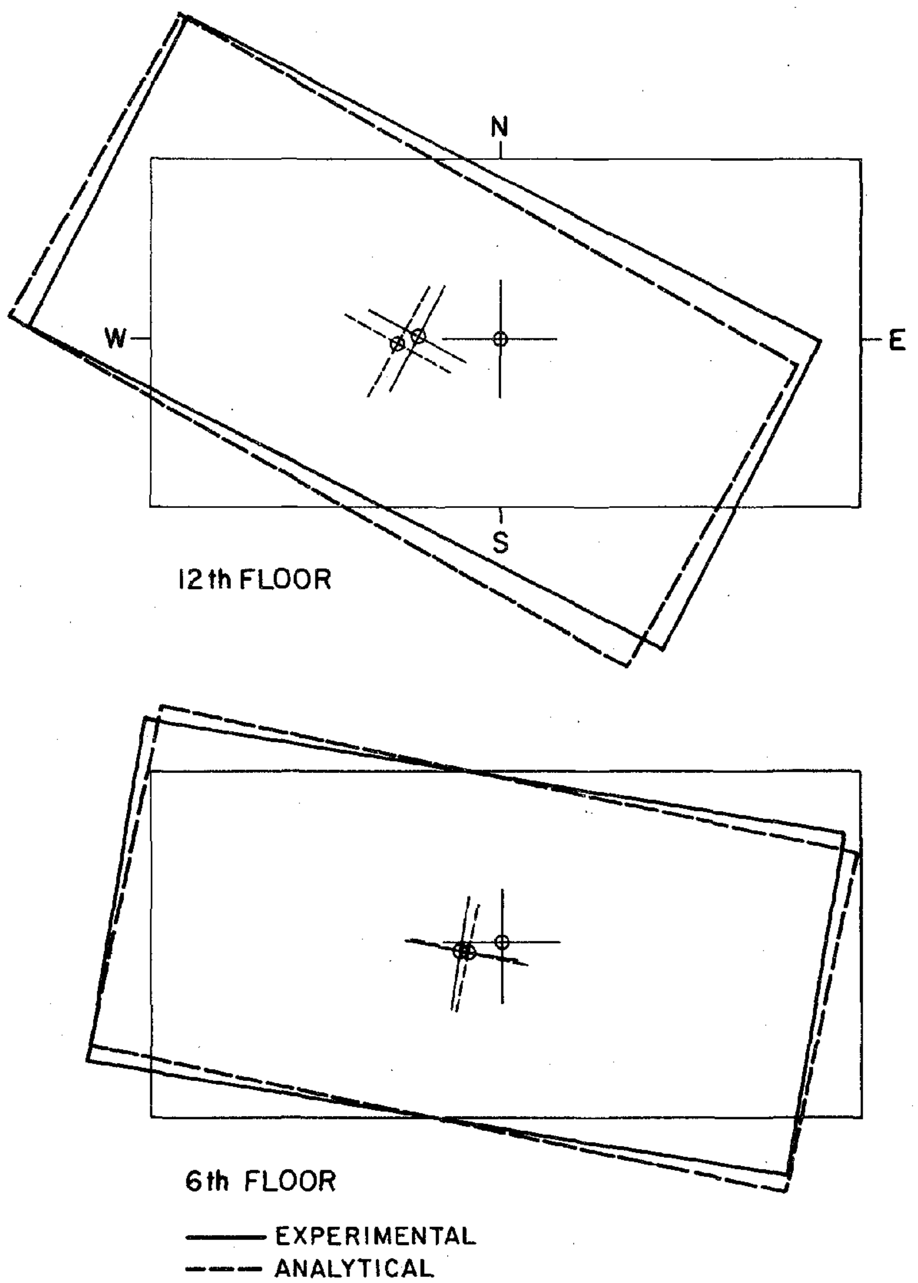


FIG. 6.16 EW FLOOR MODE SHAPES (1.76 CPS)

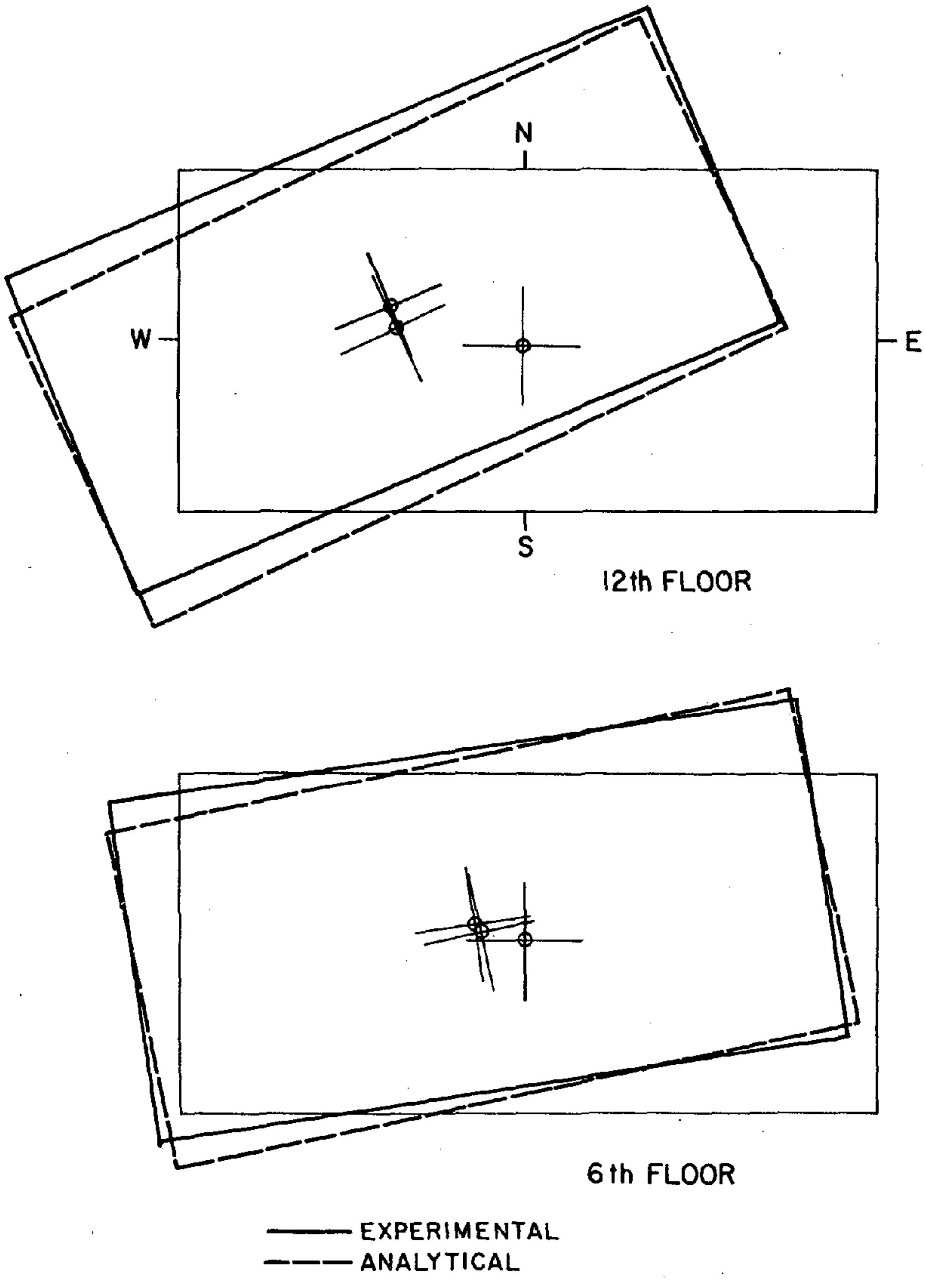


FIG. 6.17 EW FLOOR MODE SHAPES (2.09 CPS)

7. CONCLUSIONS

The results presented herewith clearly show that forced and ambient vibration studies can be carried out effectively and show very good agreement. Considering a frequency range of up to about 7 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 the translational EW and torsional modes. This highly coupled response could possibly be reduced by changing the floor plan layout.

Neglecting the foundation flexibility (rigid base model), shows an overestimation of the experimental frequencies by 30 to 50%. Thus, in the analysis of rigid structures on flexible foundations, the soil-structure interaction must be considered. In order to account for the flexibility of the foundation a dummy story was added to the analytical model in this case. Two approaches were used in determining stiffness values for the foundation, one using vibration test data and the other using available soil-pile data. In using the soil-pile data difficulty was encountered in evaluating the effective pile stiffness especially in assessing the dynamic behavior of the soil-structure system in the longitudinal direction of the building. With the final analytical model used, taking into account the flexible foundation through the addition of a dummy story, very good agreement was obtained with the experimental data.

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