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EARTHQUAKE ENGINEERING RESEARCH CENTER

DYNAMIC PROPERTIES OF A THIRTY-STORY CONDOMINIUM TOWER BUILDING

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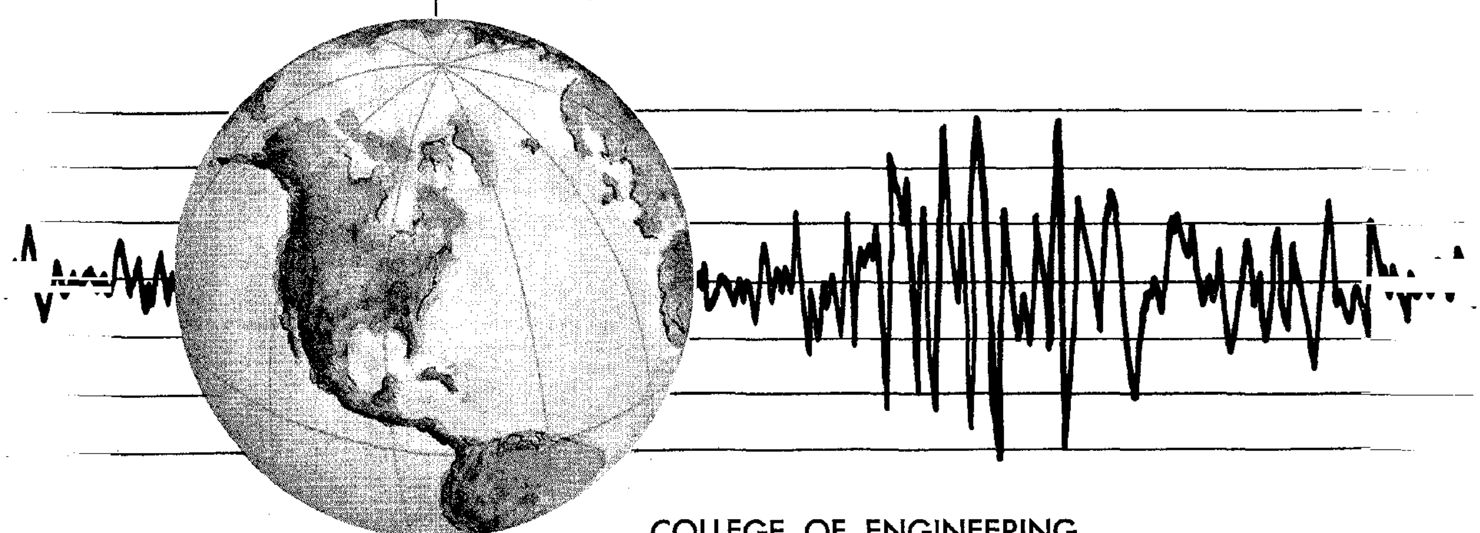
Report to the National Science Foundation



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Department of Civil Engineering
University of California
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1.a.

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ABSTRACT

The results of forced and ambient vibrations studies of a thirty story condominium building, constructed using a concrete ductile moment resistant space frame 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 SAP-80.

Rigid floor diaphragm action was noted at higher frequencies and very little structure foundation interaction was observed.

1. INTRODUCTION

1.1 General

The design of multistory structures subjected to dynamic forces resulting from foundation motions require 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 evaluate the dynamic characteristics of a multi-story reinforced concrete type structure, dynamic tests using both forced and ambient methods were performed on the Pacific Park Plaza Building in Emeryville, California. 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. 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 and the analytical dynamic results obtained, are described in Chapter 5. A general comparison and discussion of the experimental and analytical results is presented in Chapter 6.

1.2 Acknowledgement

The authors gratefully acknowledge the financial support provided by the National Science Foundation under Grant NSP CEE 81-100050-ZNF. They also wish to thank the owner, Tower House Associates of San Francisco; the structural engineers, T. Y. Lin, International; and the contractors, Lathrop/Kiewit of Emeryville, and especially Mr. Gary Kalian for his help and cooperation in coordinating and carrying out the test program.

2. DESCRIPTION OF BUILDING

2.1 General

The Pacific Park Plaza Building (Fig. 2.1) a thirty story reinforced concrete building, was tested in August 1983. This building system uses ductile moment-resistant concrete frames. The contractor was able to adhere to a rather rigid construction schedule, which required the 30-story superstructure to be completed in 13 months, by employing prefabricated reusable forms. In addition a large amount of the reinforcing steel was prefabricated and preassembled.

2.2 Architectural Layout

The overall project includes a five-level parking structure along with the thirty story tower housing 583 condominium units. The tower has three radial wings in plan, a form which maximizes the view of San Francisco bay. The central core area containes two elevator shafts and each wing contains a stairwell as shown in Fig. 2.2. As the building was to be used for condominiums, the architect could plan interior space so that partition walls were located at beam lines.

2.3 Structural Systems

The building is a 30 story reinforced concrete structure employing ductile moment-resistant concrete frames. All of the columns follow a continuous line from the foundation to the roof. In the west wing the space below the second floor is divided by a mezzanine floor. This mezzanine accommodates part of the parking structure that intersects the west wing up to the second floor level as indicated in the sections shown in Fig. 2.3. Reinforced concrete shear walls are located in each of the wings and the central core at the foundation to second floor levels only. All of the floors from the second upward are reinforced concrete frame systems with a story height of 9 ft. 6 in. The floor slabs are placed monolithically with the beams and are, in general, 5 inches thick. In all of the condominium units this gives a floor to ceiling height of 9 ft. 1 inch.

As all the frames were ductile moment-resistant, special consideration was required in order to handle the congestion of reinforcing steel in the beam-column connections. A typical beam-column floor framing plan is shown in Fig. 2.4.

The building was constructed adjacent to the San Francisco Bay and the site is underlain by a layer of soft silty clay known locally as Bay Mud. The building is therefore supported on piles driven in rows along both longitudinal and transverse column lines. A 5 ft. thick concrete mat was placed over these piles under the entire structure.

FIG. 2.1 PACIFIC PARK PLAZA BUILDING

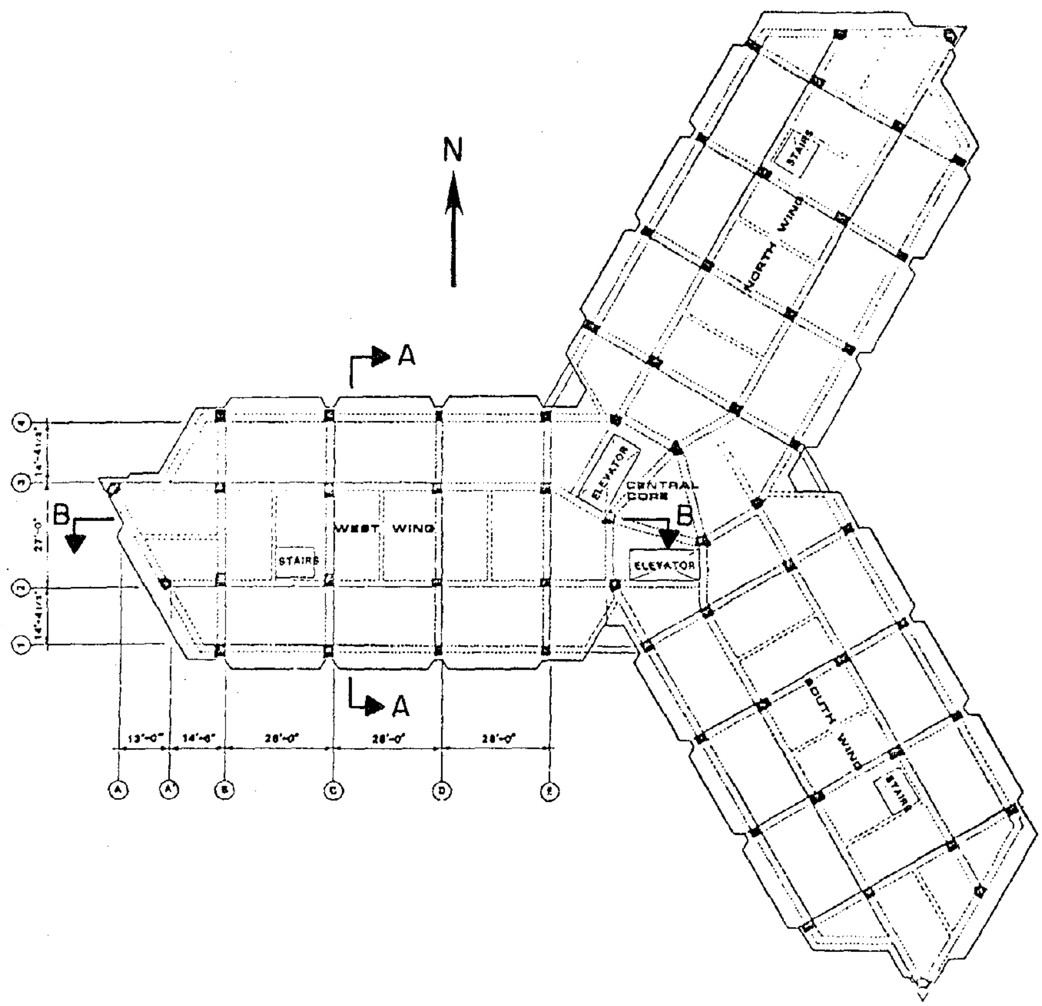


FIG. 2.2 TYPICAL FLOOR PLAN

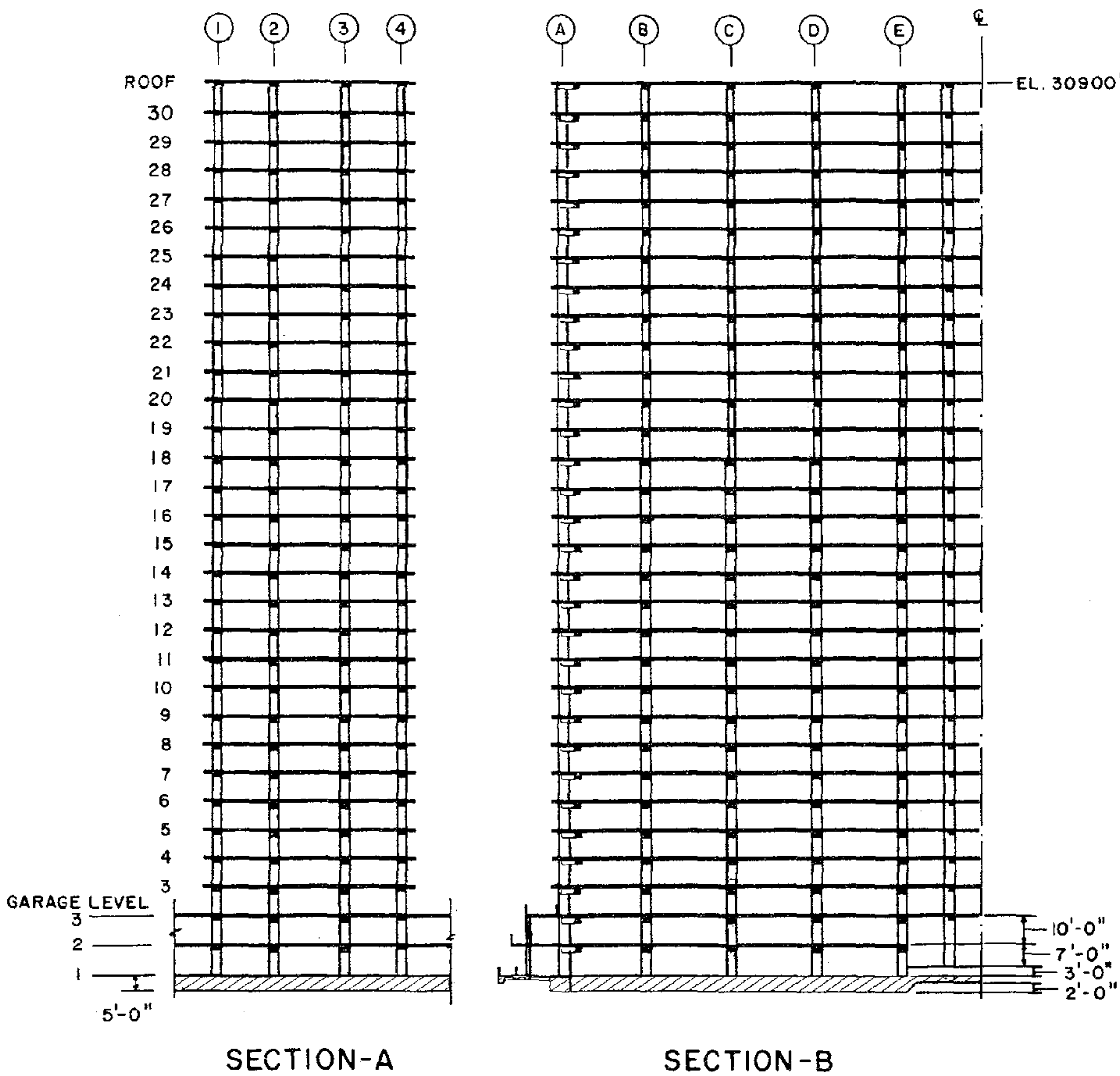


FIG. 2.3 TYPICAL SECTION OF BUILDING - WEST WING

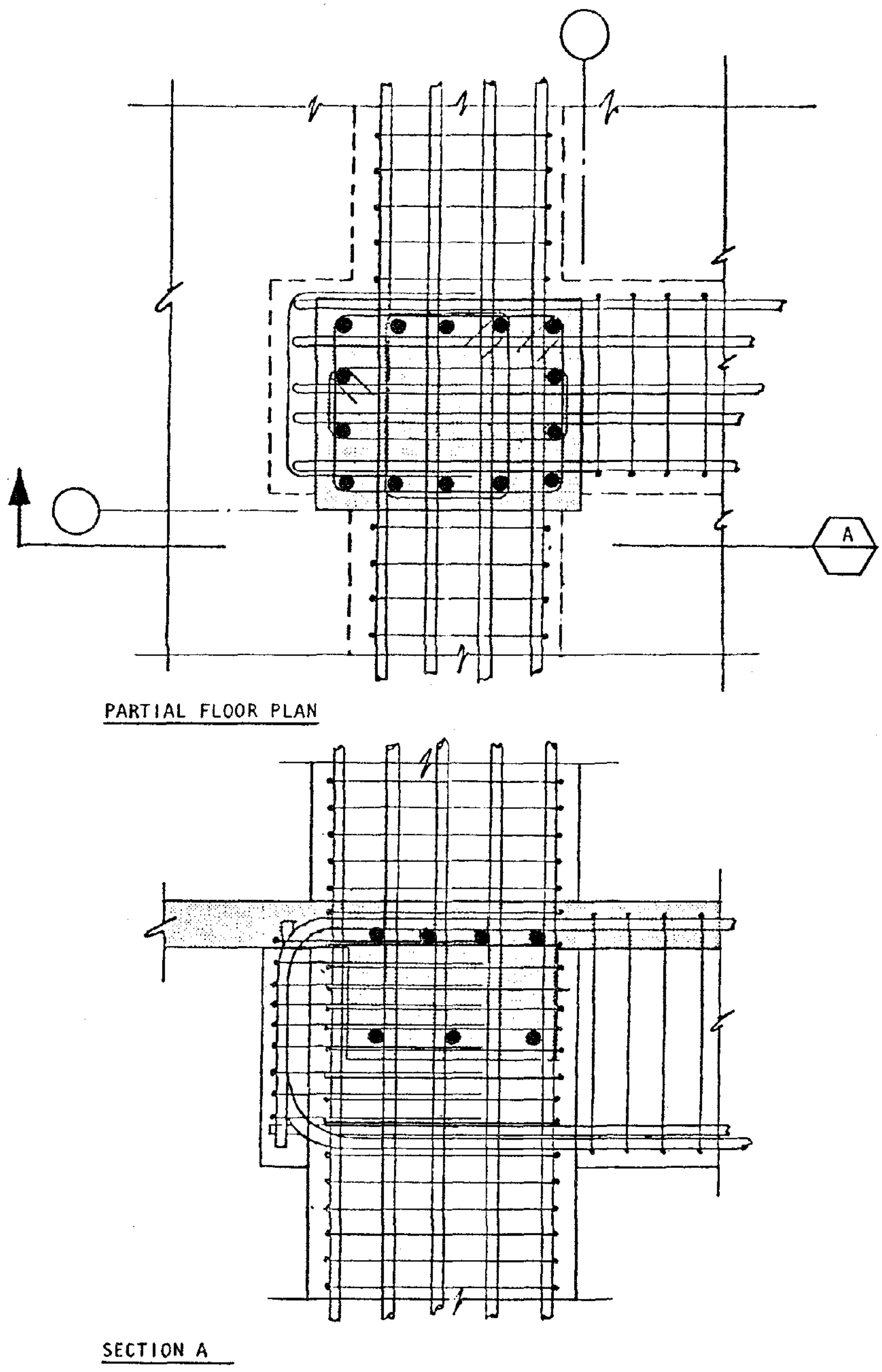


FIG. 2.4 TYPICAL FLOOR FRAMING PLAN

3. FORCED VIBRATION STUDY

3.1 General

The forced vibration study was carried out and completed during August and September 1983. The building was structurally completed prior to the experimental work. 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, ten 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 originally 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 hz, and the minimum practical speed is approximately 0.5 hz. At 0.5 hz 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 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 30th floor in the north and south wings of the building, as indicated in Fig. 3.3. They were located on a north-south line 40 feet on each side of the center of stiffness. The center-of-stiffness was located at the center of the central core. Associated vibration control and recording equipment was also placed on the 30th 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 $\pm$ 0.25 g.

3.2.3 Equipment for Measurement of Frequency

For the vibration generators, the vibration excitation frequencies were input through the terminal of the electronic control unit to an internal micro-processor which set and regulated the speed of the motor driving the baskets. An encoder attached to a rotating shaft driven by a transmission belt from the motor was used to monitor the vibration frequency of the generator.

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 input frequency was observed and checked with a digital counter 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 generally to one one-hundreth of a hertz. 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 is 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 ten accelerometers available, it was decided to develop the mode shapes by taking simultaneous measurements at ten floors, starting at the ground floor and going up in approximately three story increments.

In the operation of the vibration generators during the mode shape investigation, some difficulties were encountered with one of the control units. Therefore only one generator was used in carrying out this portion of the investigation. The resonant frequencies had been determined previously using the two generators and therefore even though it was somewhat inconvenient in using only one generator the data determined was adequate to define the mode shapes.

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 or end of each day.

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.

The horizontal mode shapes of the 30th floor for the N-S motion were investigated by locating the ten accelerometers in five pairs, two in the west wing, one pair each in the north and south wings and one pair in the central core.

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 is 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 extermely 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 30th floor throughout the test program as shown in Figure 3.3. Also shown is the center of stiffness (C.S.). The selection of the location of the two vibration generators 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°-out-of-phase excitation). In this case after reviewing the building floor plan for the 30th floor, it was decided to place shakers along the N-S line that passed through the center of stiffness of the building. The distance from the center of stiffness was taken as the maximum practical distance to the edge of the floor system in the south wing. 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).

3.4.2 Frequency Response Data

The frequency response curves for E-W, N-S and torsional forcing conditions are presented in Figures 3.4 through 3.19. The first six translational modes, respectively, in the E-W and the N-S direction were excited, as well as the first four torsional modes. The curves are plotted in the form of normalized displacement amplitude versus exciting frequency.

The ordinates were obtained by dividing the recorded acceleration amplitude by the square of the exciting frequency to obtain acceleration amplitudes for a constant equivalent force amplitude. The values thus obtained are divided by the square of the circular frequency (rad/sec) to obtain normalized displacement amplitudes. For convenience the actual exciting force (F_r) and displacement amplitude (u_r) for each of the excited resonancies are given in Figs. 3.4 through 3.19, as well as the calculated damping factors.

The resonant frequencies and damping factors evaluated from the response curves are summarized in Tables 3.1 and 3.2, respectively.

The generated exciting force by either one or both shaking machines and corresponding resonant displacement amplitude for each resonant frequency are given in Tables 3.3 and 3.4, respectively. The E-W and N-S displacement amplitudes are for the center of the building whereas the torsional displacements were measured near the end of the south wing.

TABLE 3.1 RESONANT FREQUENCIES (hz)

Excitation	Mode					
	1	2	3	4	5	6
E-W	0.595	1.675	3.12	4.39	6.19	7.45
N-S	0.590	1.66	3.09	4.36	6.24	7.45
Torsional	0.565	1.70	3.16	4.53	--	--

TABLE 3.2 DAMPING FACTORS (%) FROM RESONANCE CURVES

Excitation	Mode					
	1	2	3	4	5	6
E-W	1.8	1.9	3.2	2.6	1.8	1.3
N-S	1.7	1.3	2.9	2.6	1.8	2.1
Torsional	1.5	1.32	1.7	2.6	--	--

TABLE 3.3 RESONANT FORCE AMPLITUDES

Excitation	Mode					
	1	2	3	4	5	6
E-W (LB)	570	2406	8348	3723	7402	2944
N-S (LB)	283	2217	4094	3492	2065	2944
Torsional (LB FT)	20,547	183,835	342,552	158,576	--	--

TABLE 3.4 RESONANT DISPLACEMENT AMPLITUDES (x 10^{-2} in.) 30th FLOOR

Excitation	Mode					
	1	2	3	4	5	6
E-W	1.30	0.44	0.227	0.0422	0.0929	0.0312
N-S	1.18	0.496	0.132	0.0424	0.0648	0.0196
Torsional	0.120	0.60	0.176	0.0234	--	--

The vertical mode shapes for the translational and torsional modes that were excited are shown in Figs. 3.20 through 3.33. Particular attention has been given to observe in-plane deformations on the 30th floor for five of the N-S excited resonances. These in-plane floor vibrational modes are shown in Figs. 3.34 through 3.38.

3.5 Discussion of Experimental Results

The forced-vibration study of the building was conducive in obtaining accurate resonant frequencies for the first six translational modes of vibration in E-W and N-S direction, as well as the first four torsional modes of vibration. The resonant frequencies were well separated, and it was of interest to note the ratios of the observed higher mode frequencies with respect to the fundamental one. These ratios are given in Table 3.5 for all three directions of excited vibrations, and they indicate a type of over-all structural response.

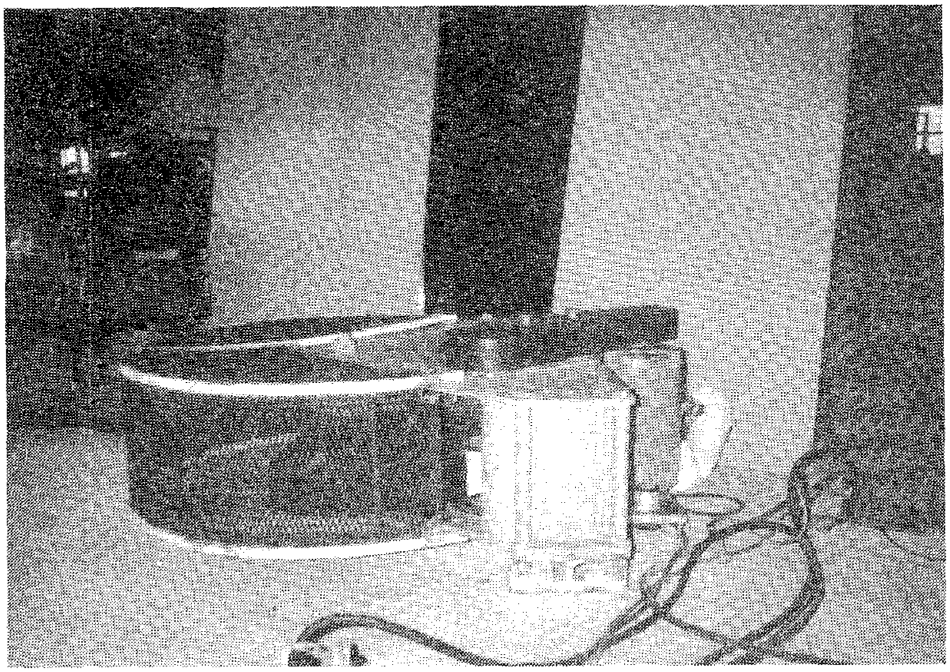
TABLE 3.5 RATIO OF RESONANT FREQUENCIES

Mode	Translational E-W		Translational N-S		Torsional	
	fi (cps)	fi/f_1	fi (cps)	fi/f_1	fi cps	fi/f_1
1	0.595	1.0	0.59	1.0	0.565	1.0
2	1.675	2.82	1.66	2.81	1.70	3.00
3	3.12	5.24	3.09	5.24	3.16	5.59
4	4.39	7.38	4.36	7.39	4.53	8.02
5	6.19	10.40	6.24	10.58		
6	7.45	12.52	7.45	12.63		

From these results it may be concluded that the building vibration in both translational directions as well as torsional vibration are predominantly of the shear type, because the determined frequency ratios follow closely the ratios 1, 3, 5, 7, 9, 11 . . ., which apply for the uniform shear beam.

For the radial wing floor plan of the building it should be expected that the translational modes will exist in the direction parallel to the axis of the building wings. One of the translational directions of vibration excited was selected along the E-W line of the west wing of the building. The second direction was orthogonal to this one in the N-S direction. The E-W and N-S translational frequencies and mode shapes are essentially identical. And, in addition, the torsional frequencies and modes shapes, at least for the first four that were identified, are practically the same as the translational values. From these results it could be concluded that the mass and stiffness center coincide with the geometric center. In the lower modes the floor slabs are rigid, however, as noted in Fig. 3.38 for the fifth mode N-S translation the floor slab exhibited some in-plane bending.

Damping coefficients varied from about 1 to 3% of critical damping in all modes. Similar damping values have been reported from other full-scale forced vibration studies on buildings, (5, 12, 13, 14).



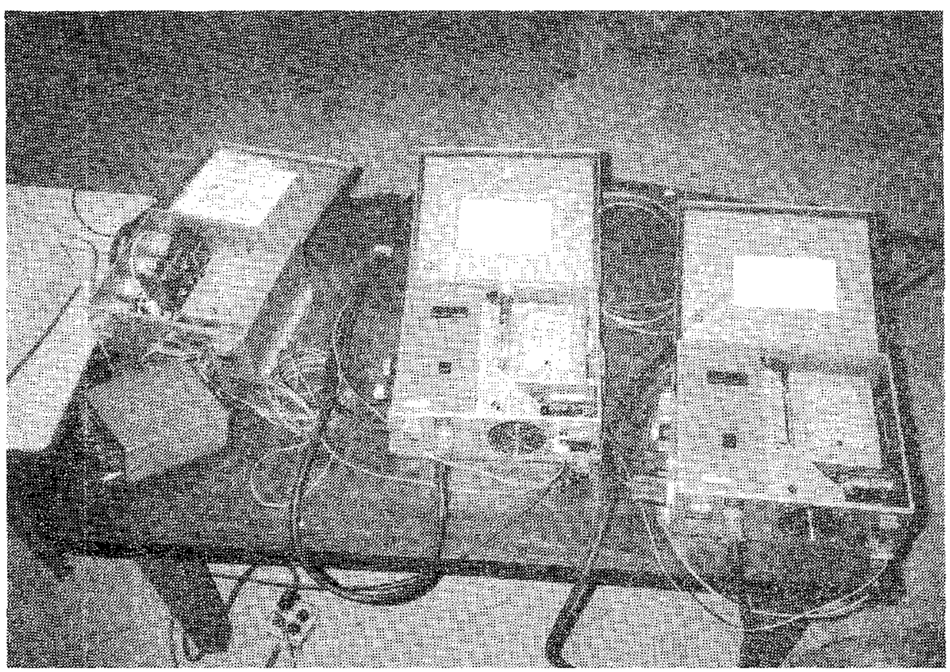


FIG. 3.1 FORCED VIBRATION GENERATOR AND CONTROL UNIT

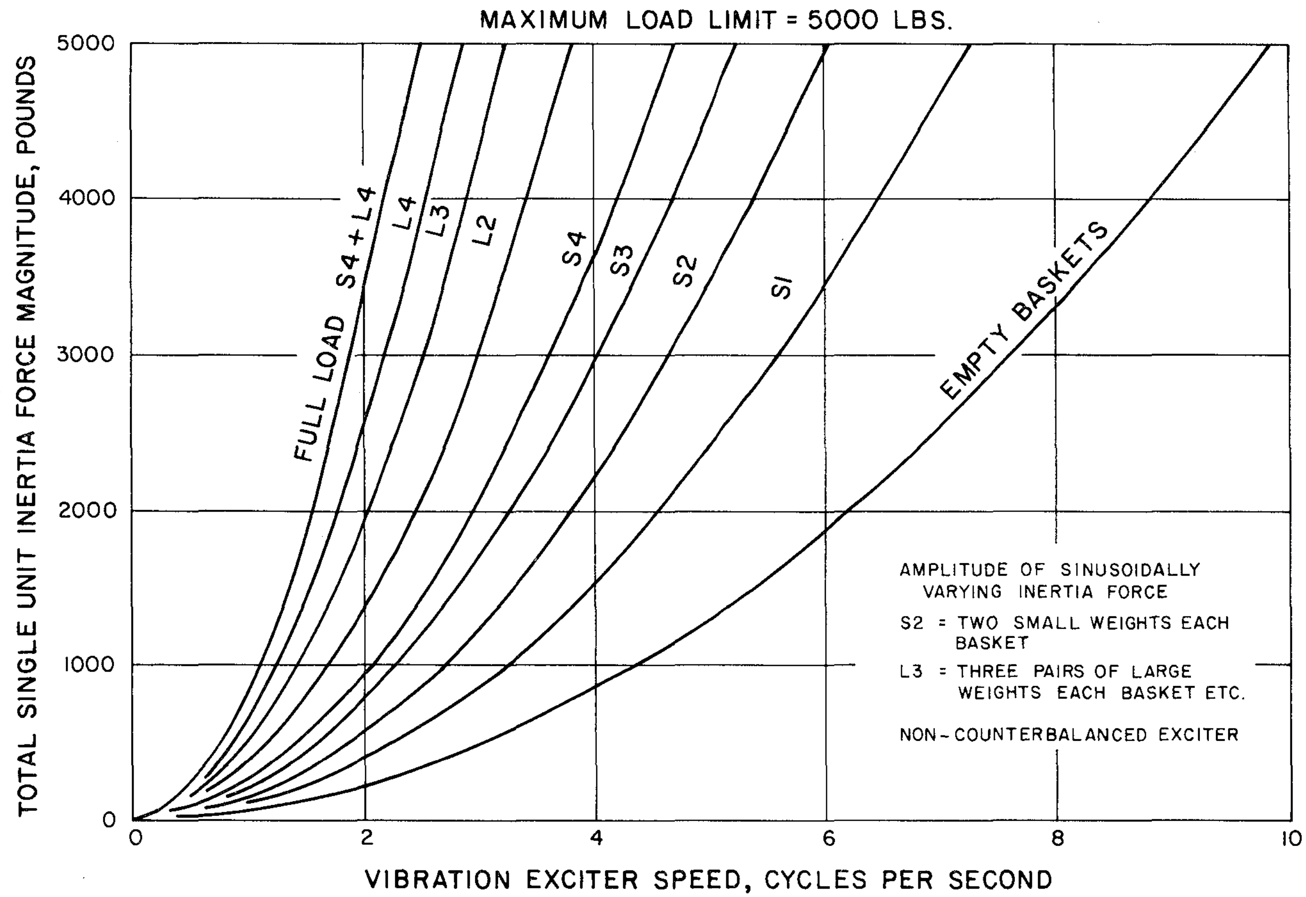


FIG. 3.2 VIBRATION FORCE VS. SPEED NON-COUNTER BALANCED

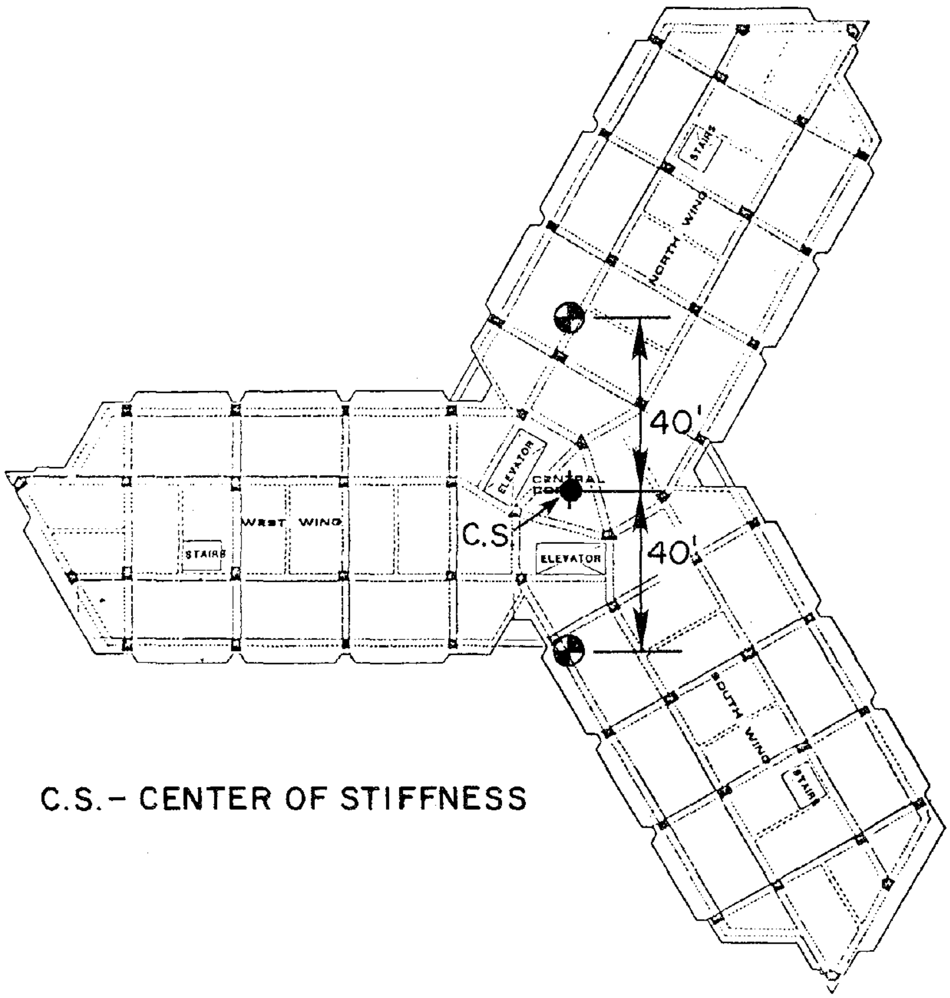


FIG. 3.3 LOCATION OF VIBRATION GENERATORS

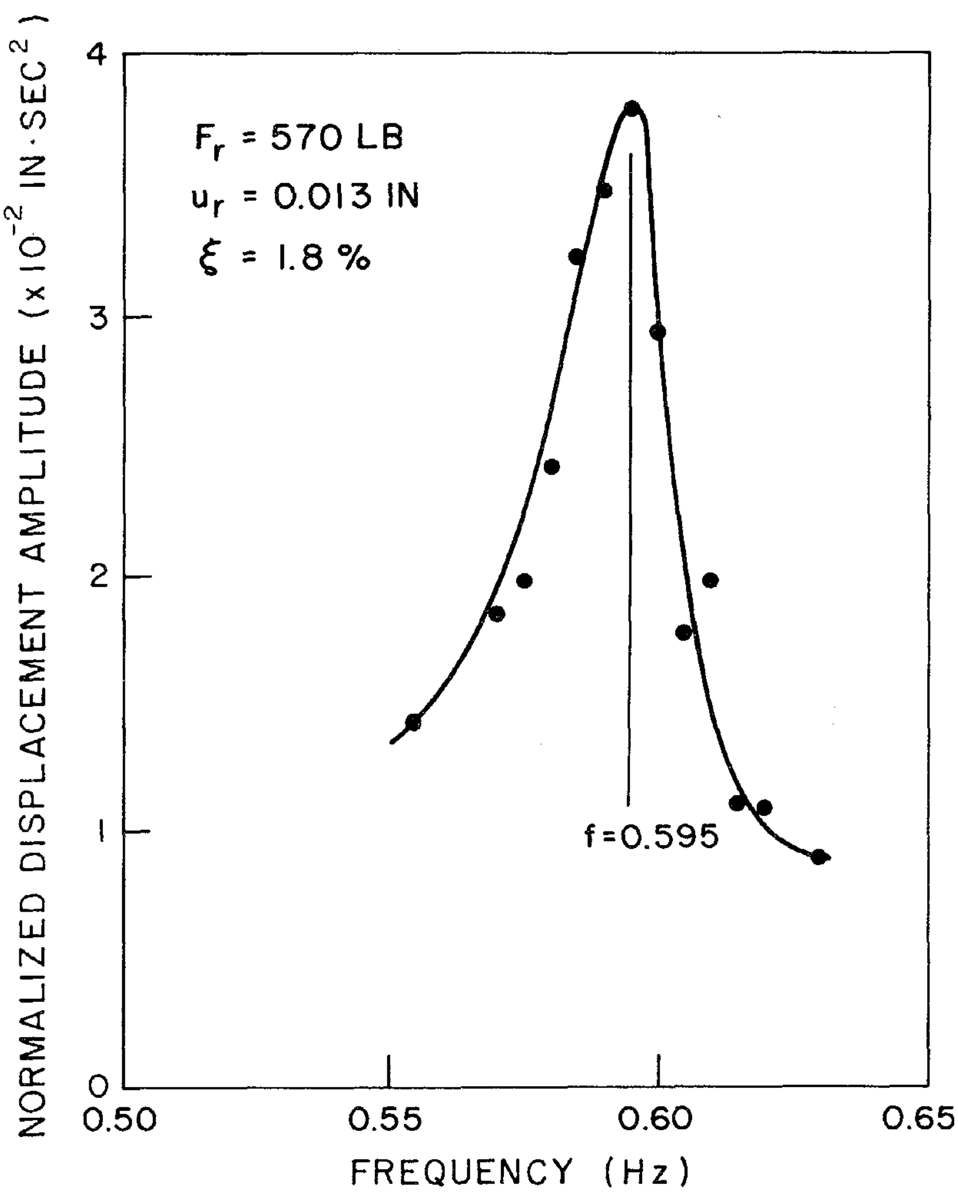


FIG. 3.4 FREQUENCY RESPONSE, FIRST MODE E-W

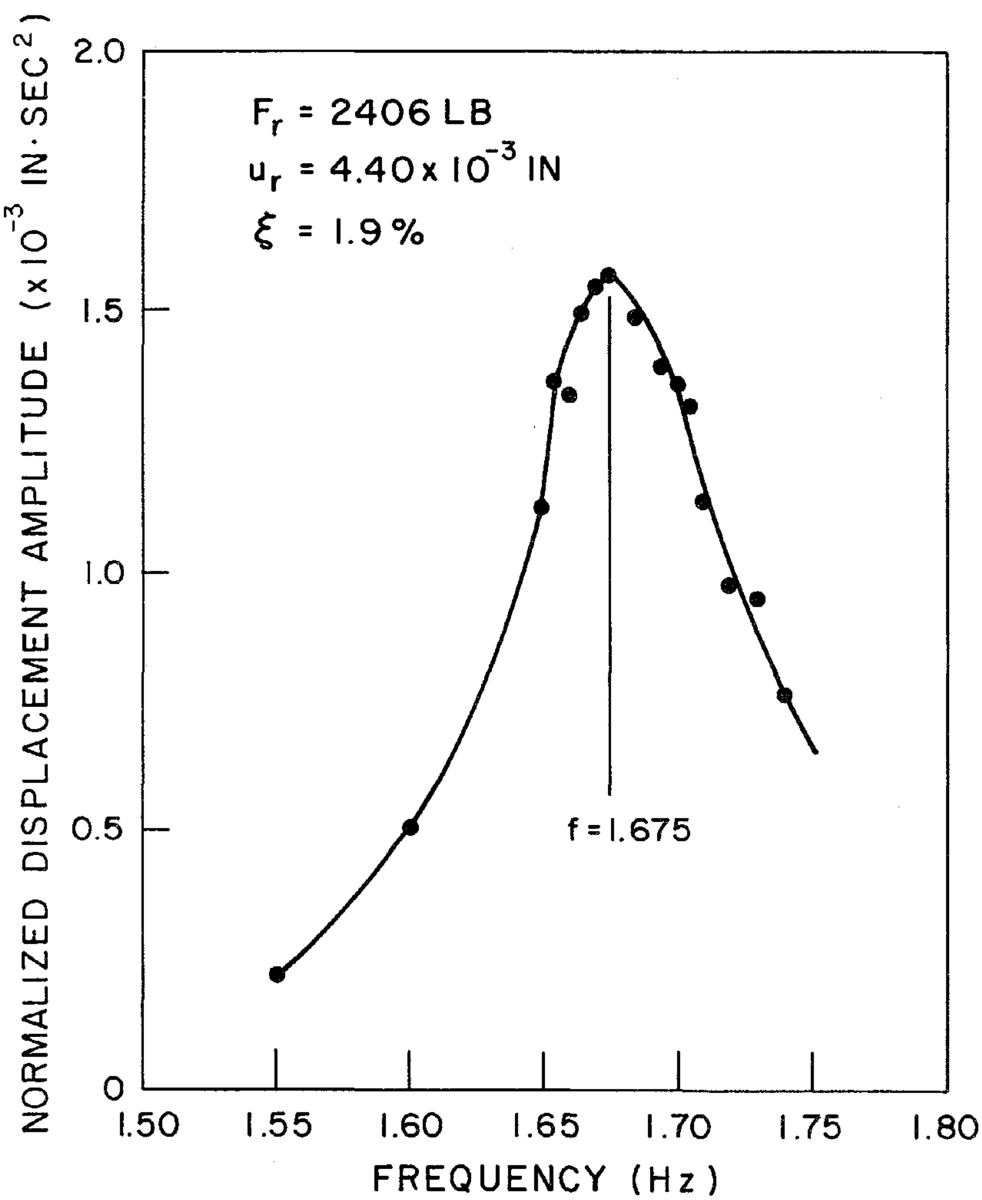


FIG. 3.5 FREQUENCY RESPONSE, SECOND MODE E-W

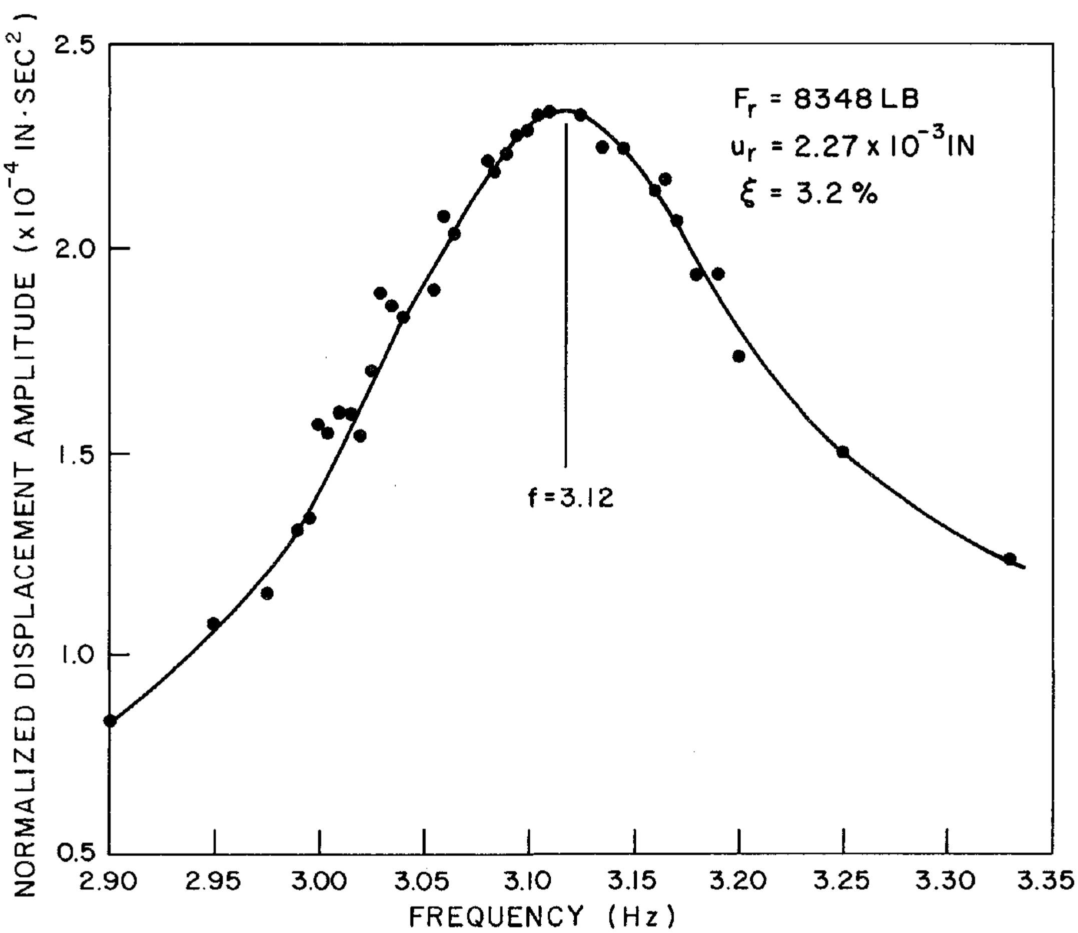


FIG. 3.6 FREQUENCY RESPONSE, THIRD MODE E-W

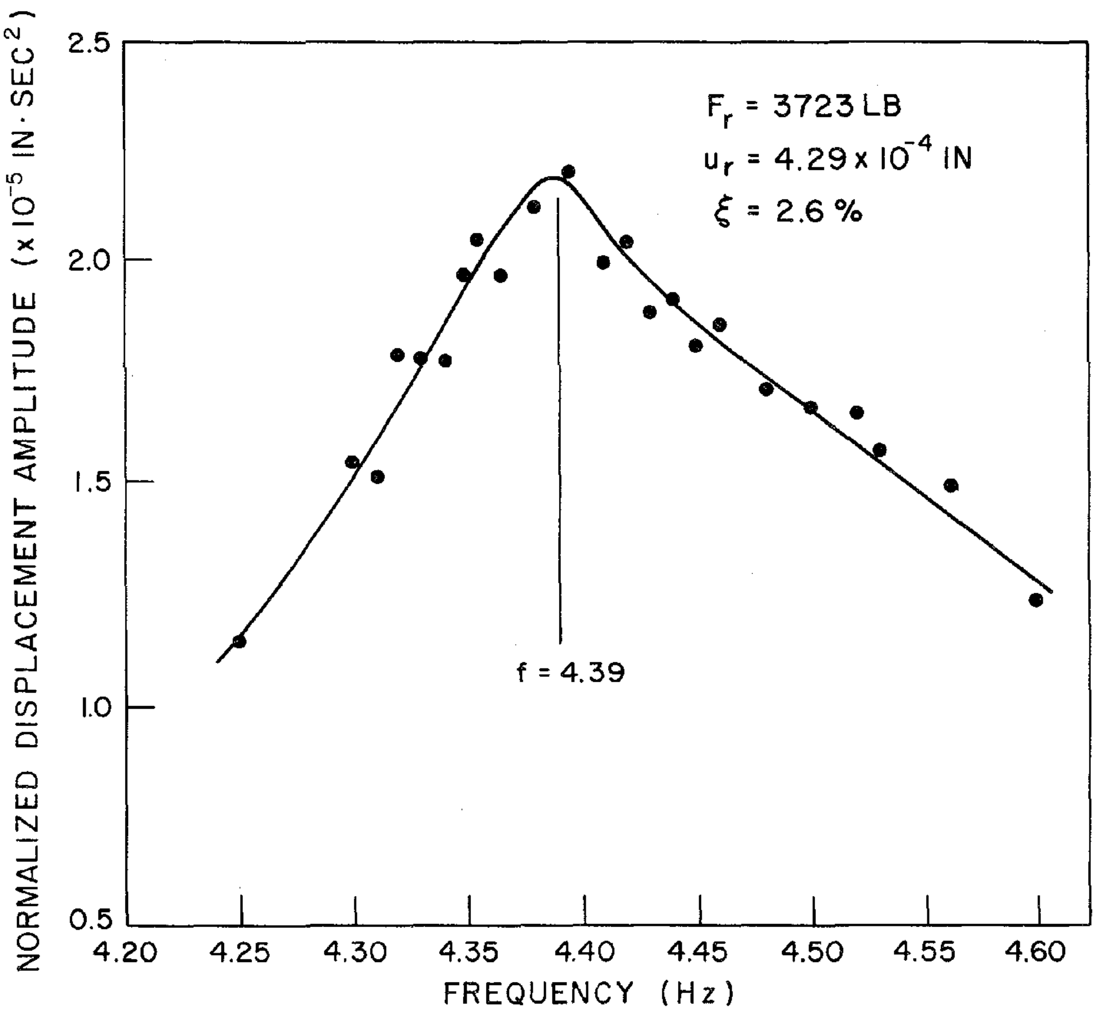


FIG. 3.7 FREQUENCY RESPONSE, FOURTH MODE E-W

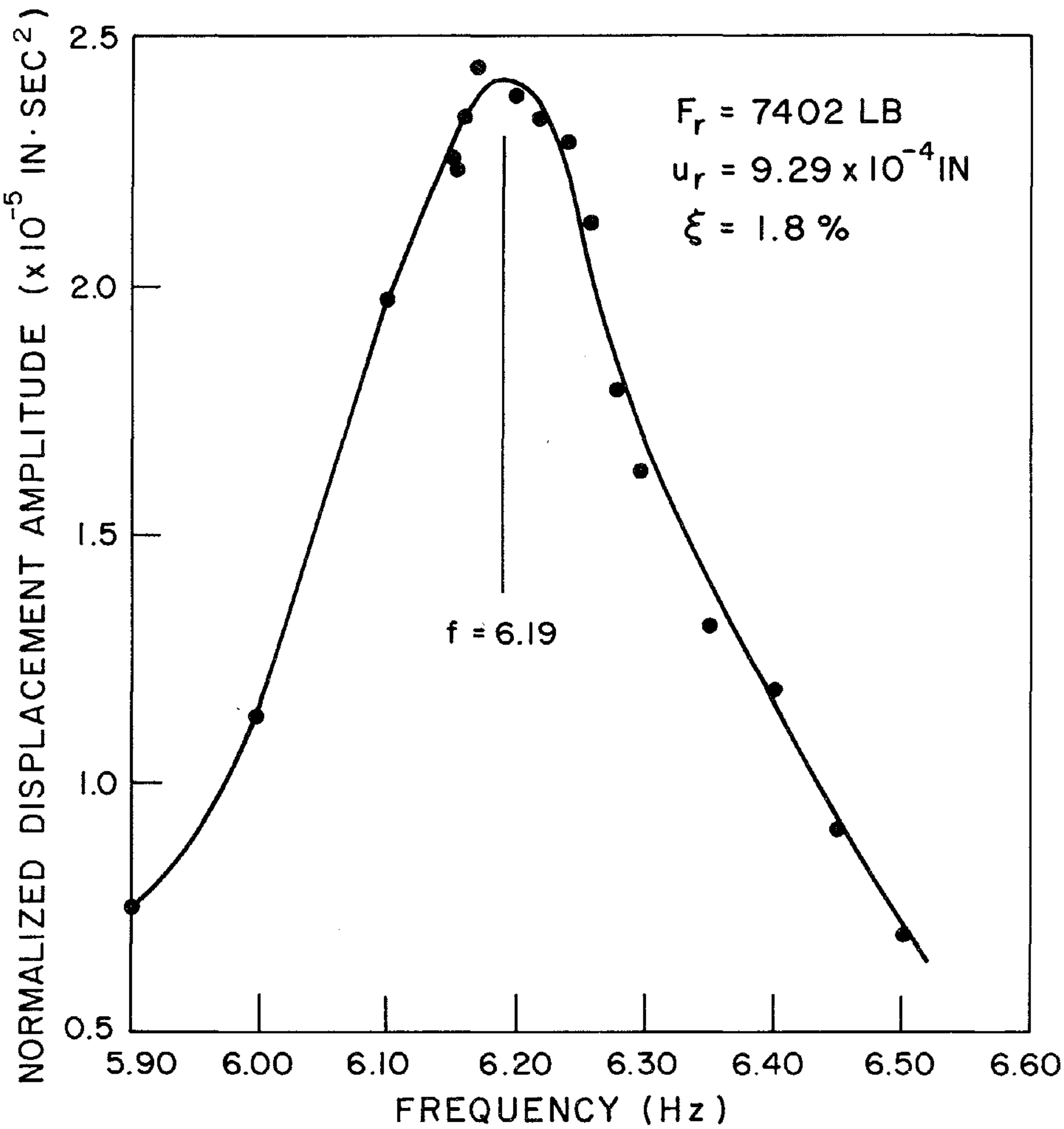


FIG. 3.8 FREQUENCY RESPONSE, FIFTH MODE E-W

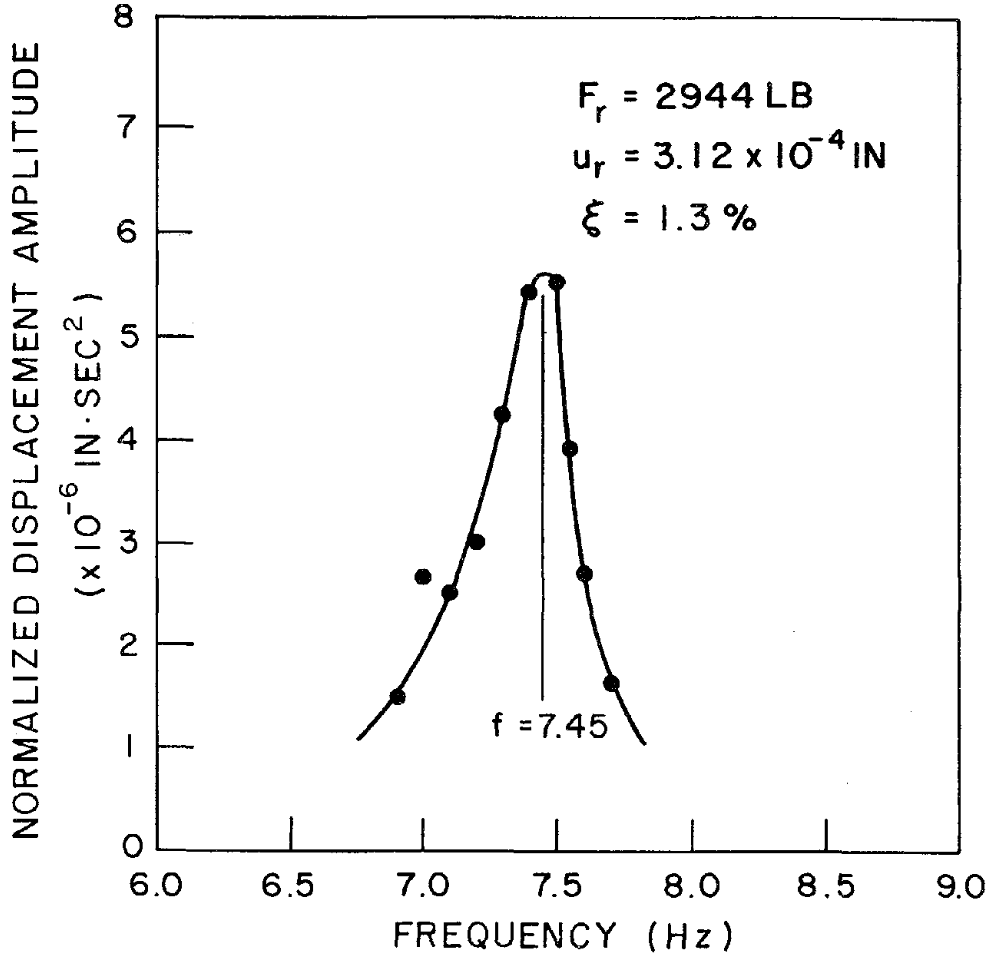


FIG. 3.9 FREQUENCY RESPONSE, SIXTH MODE E-W

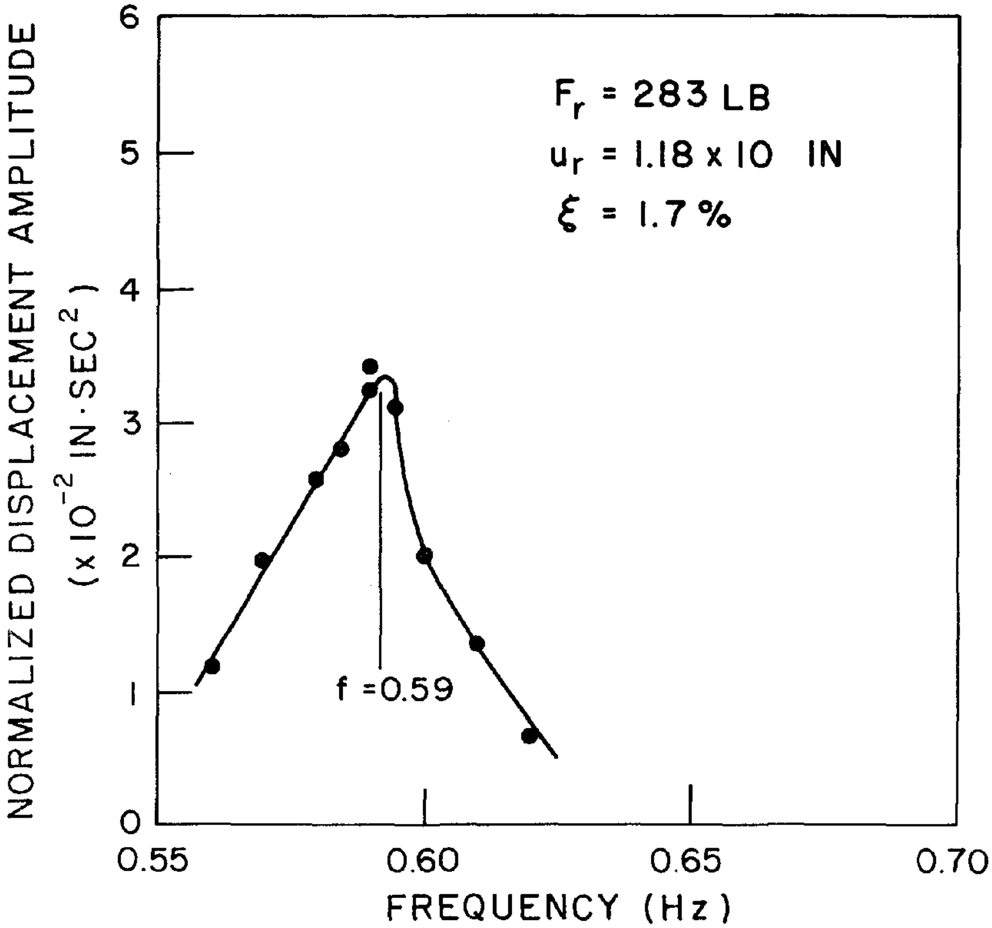


FIG. 3.10 FREQUENCY RESPONSE, FIRST MODE N-S

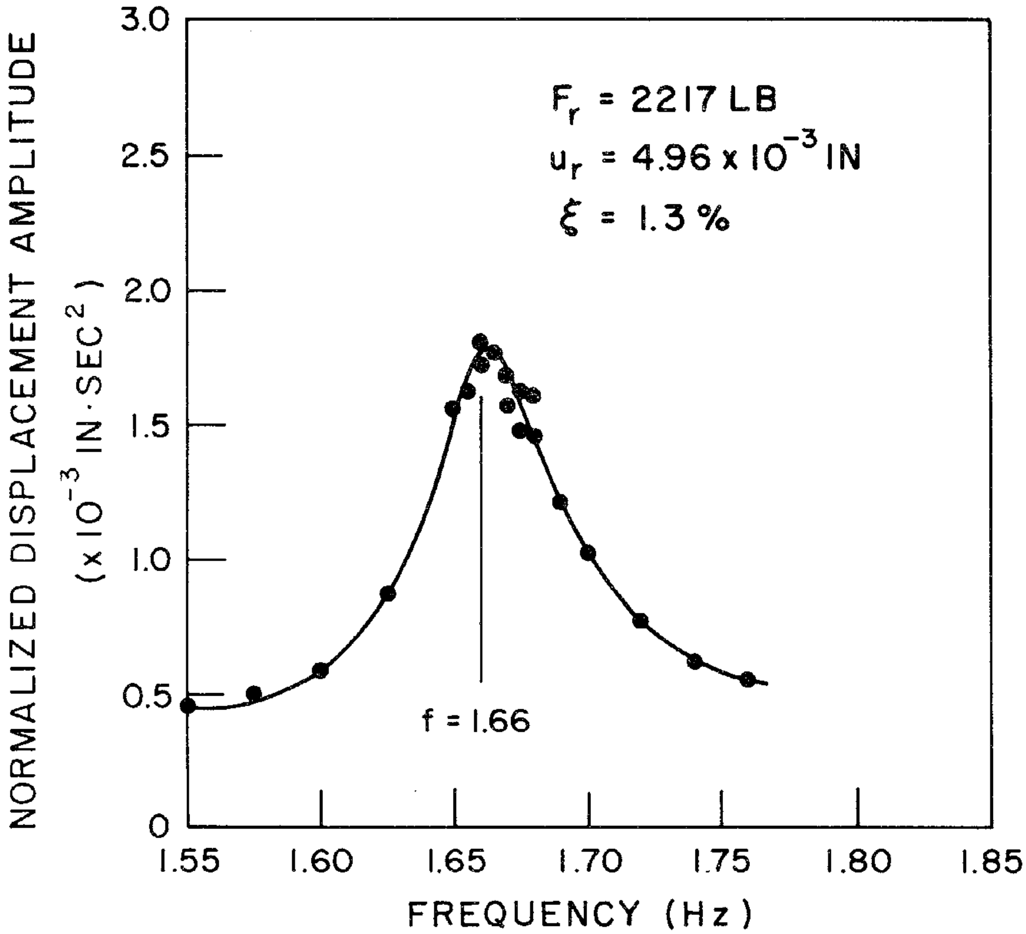


FIG. 3.11 FREQUENCY RESPONSE, SECOND MODE N-S

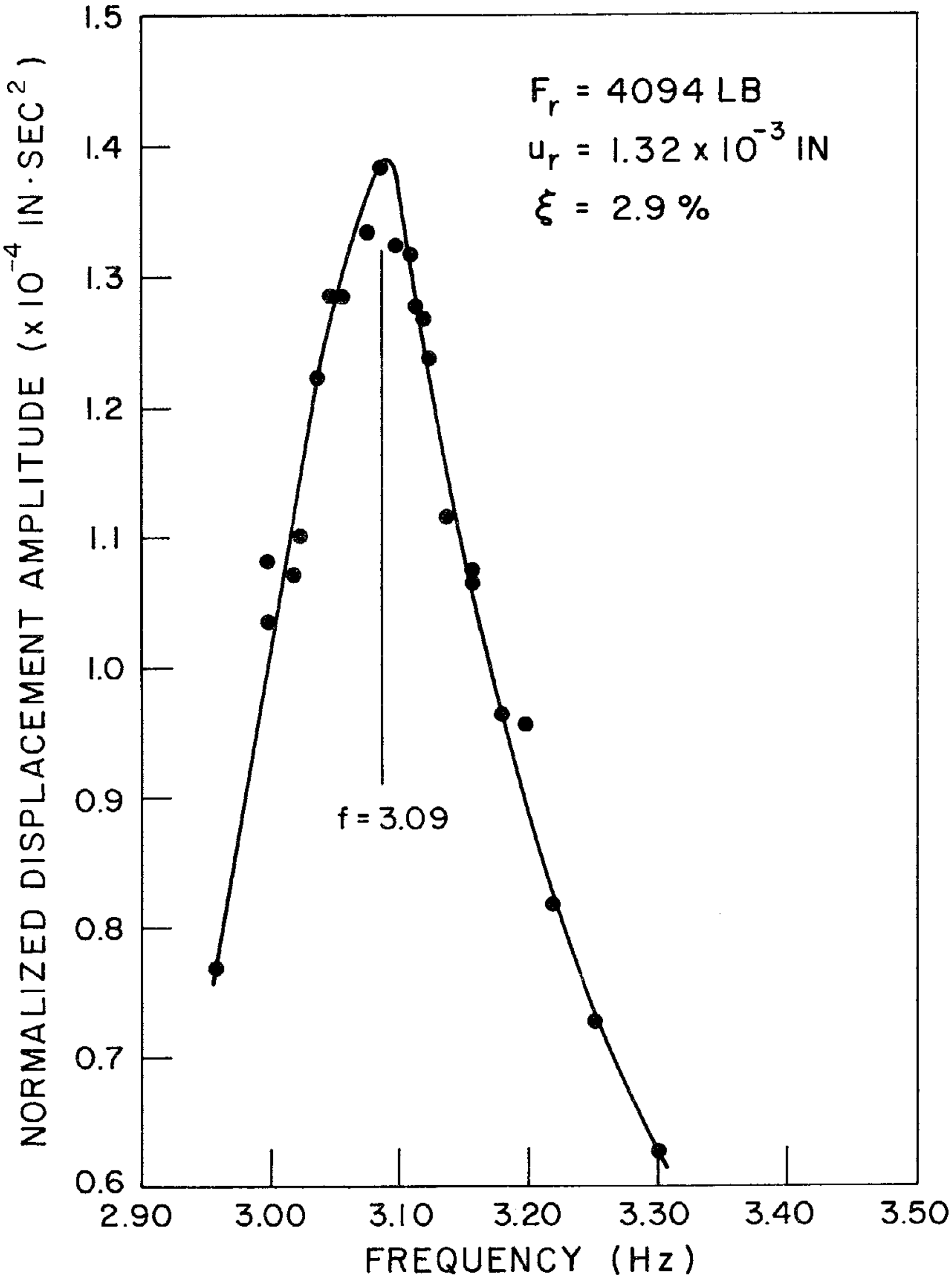


FIG. 3.12 FREQUENCY RESPONSE, THIRD MODE N-S

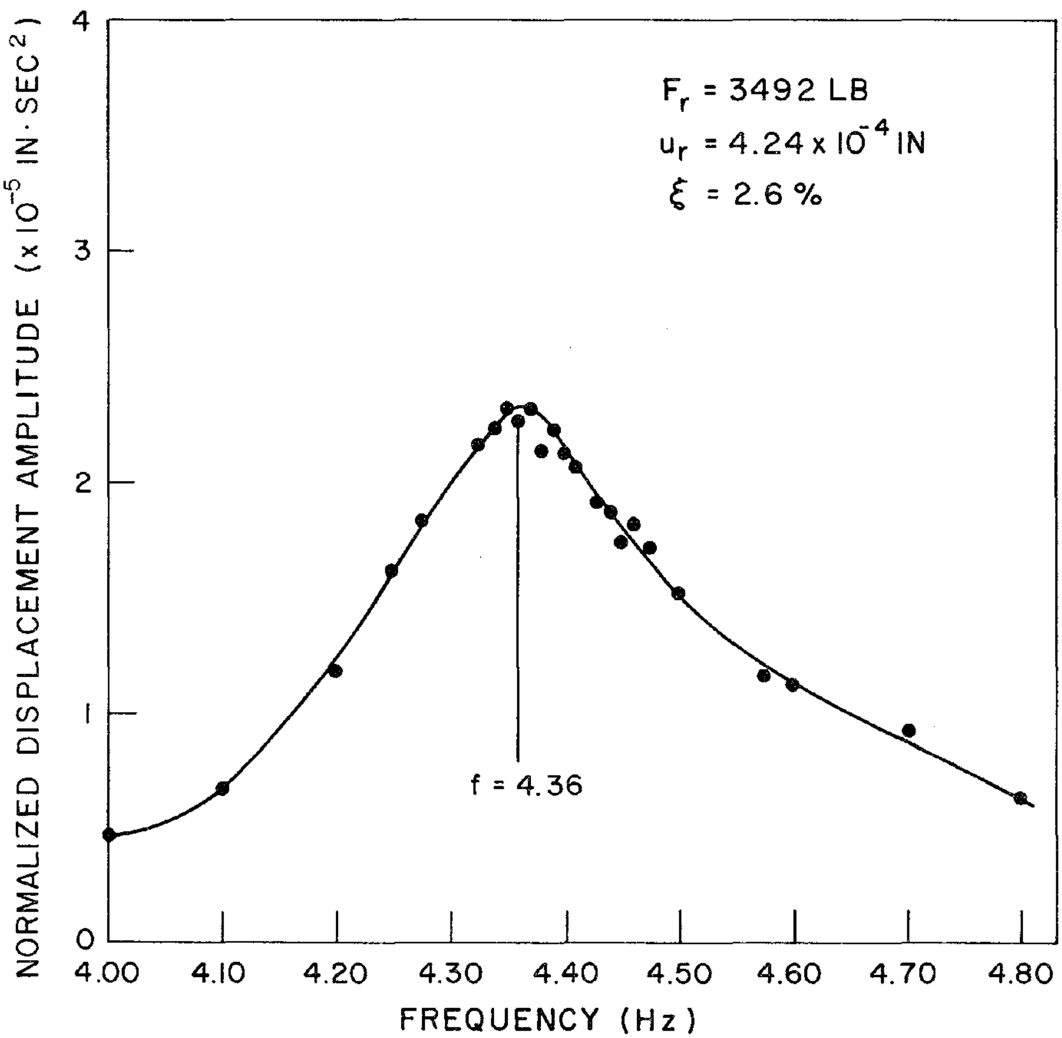


FIG. 3.13 FREQUENCY RESPONSE, FOURTH MODE N-S

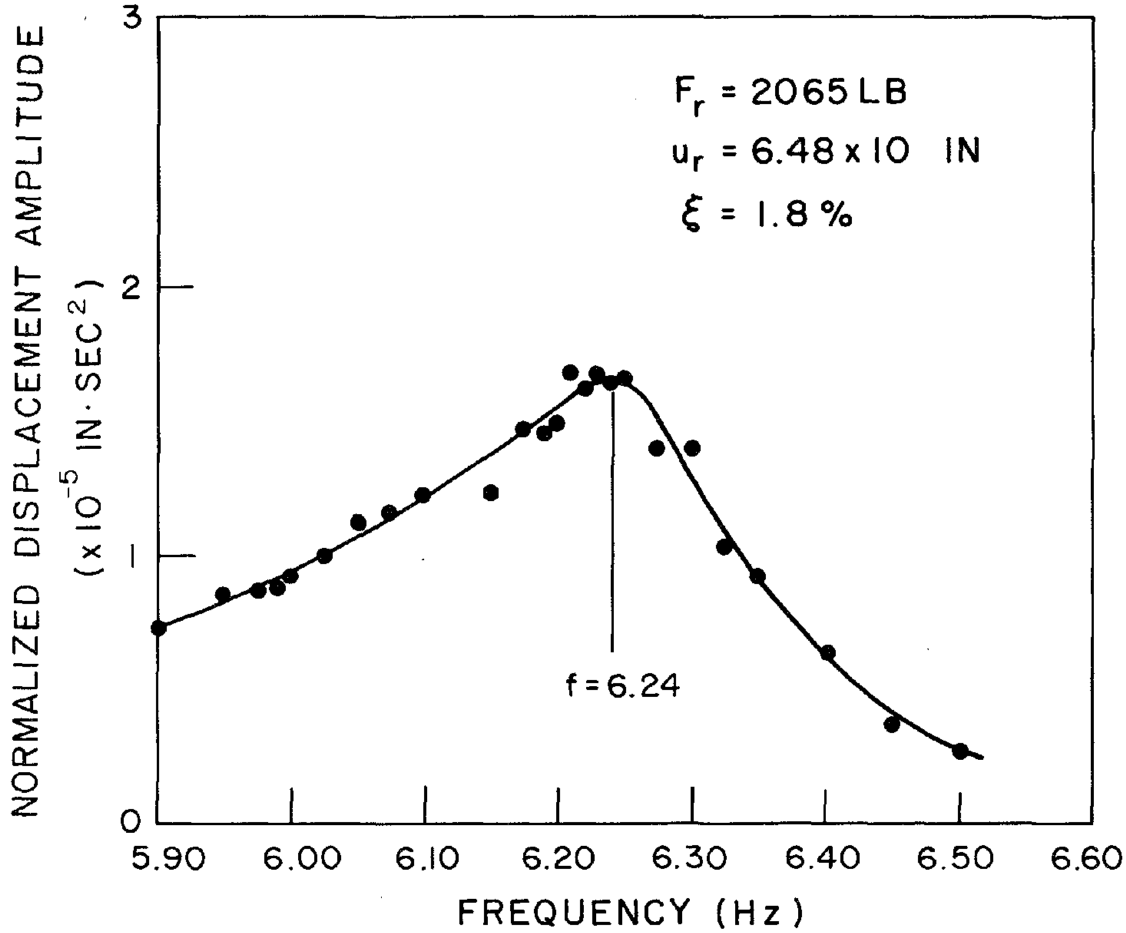


FIG. 3.14 FREQUENCY RESPONSE, FIFTH MODE N-S

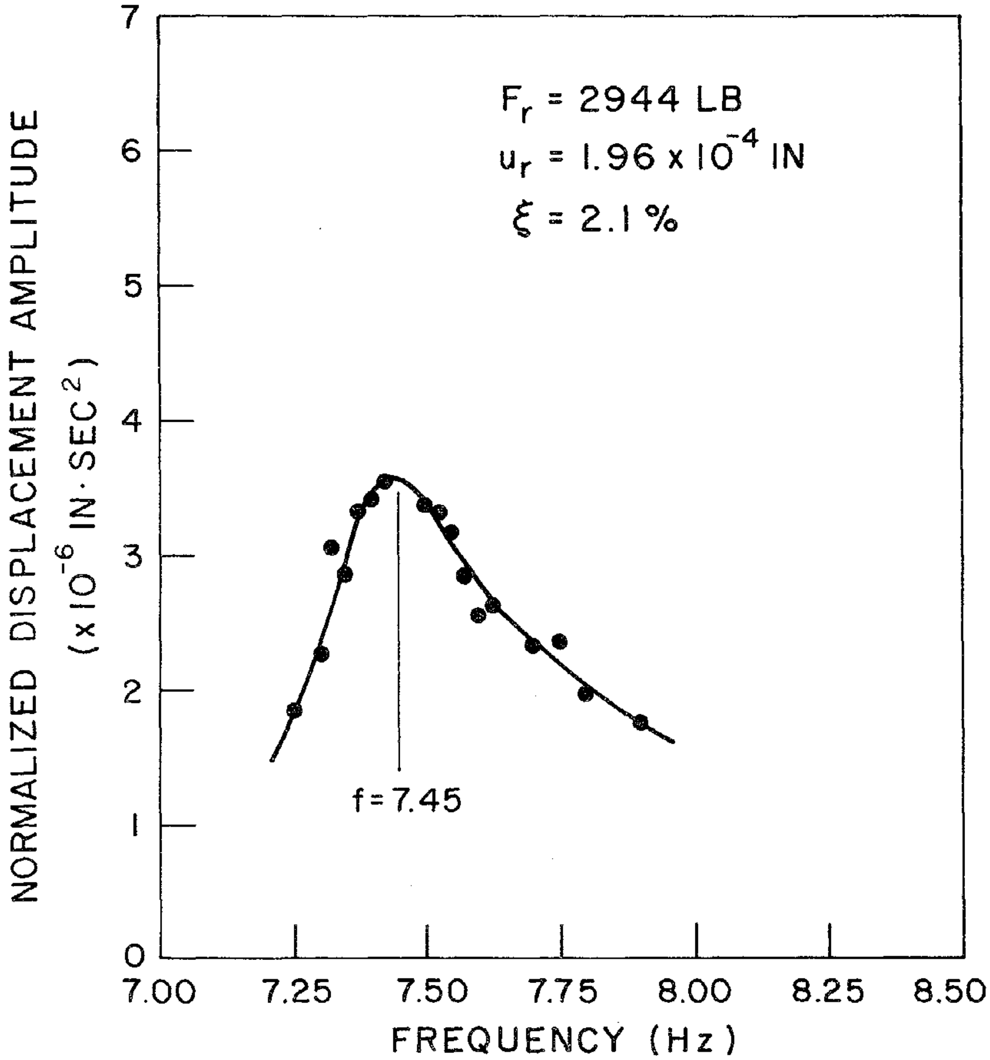


FIG. 3.15 FREQUENCY RESPONSE, SIXTH MODE N-S

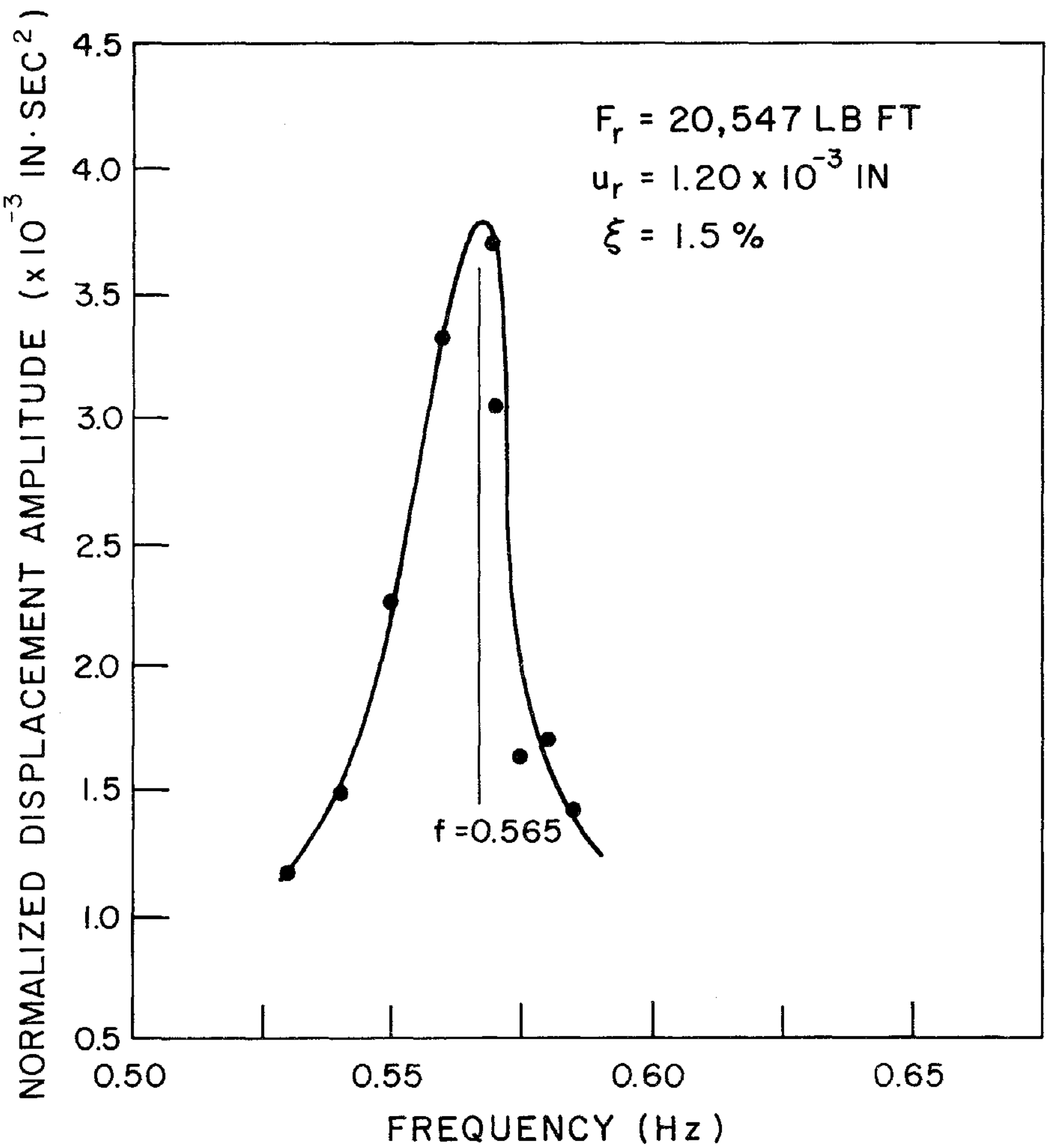


FIG. 3.16 FREQUENCY RESPONSE, FIRST TORSIONAL MODE

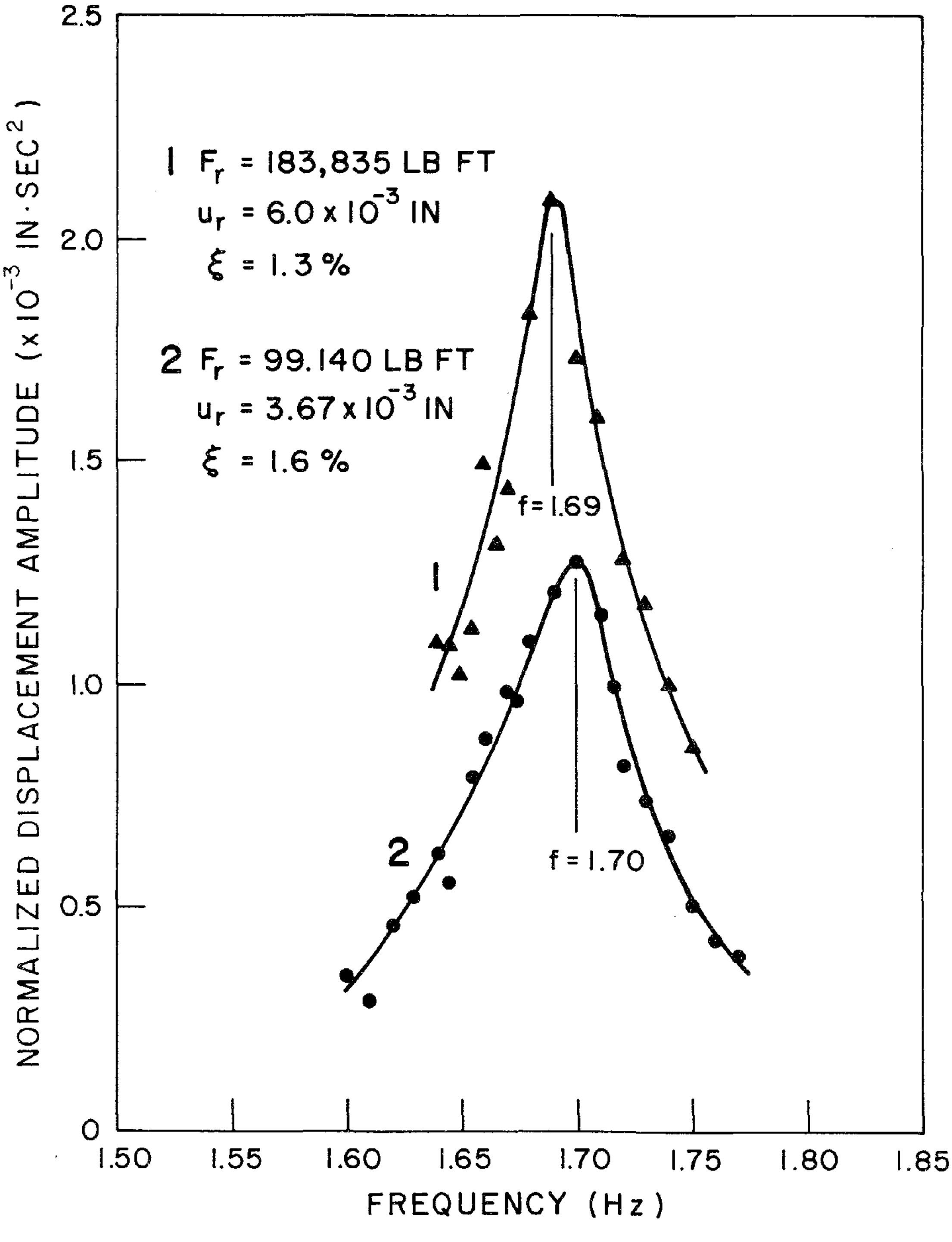


FIG. 3.17 FREQUENCY RESPONSE, SECOND TORSIONAL MODE

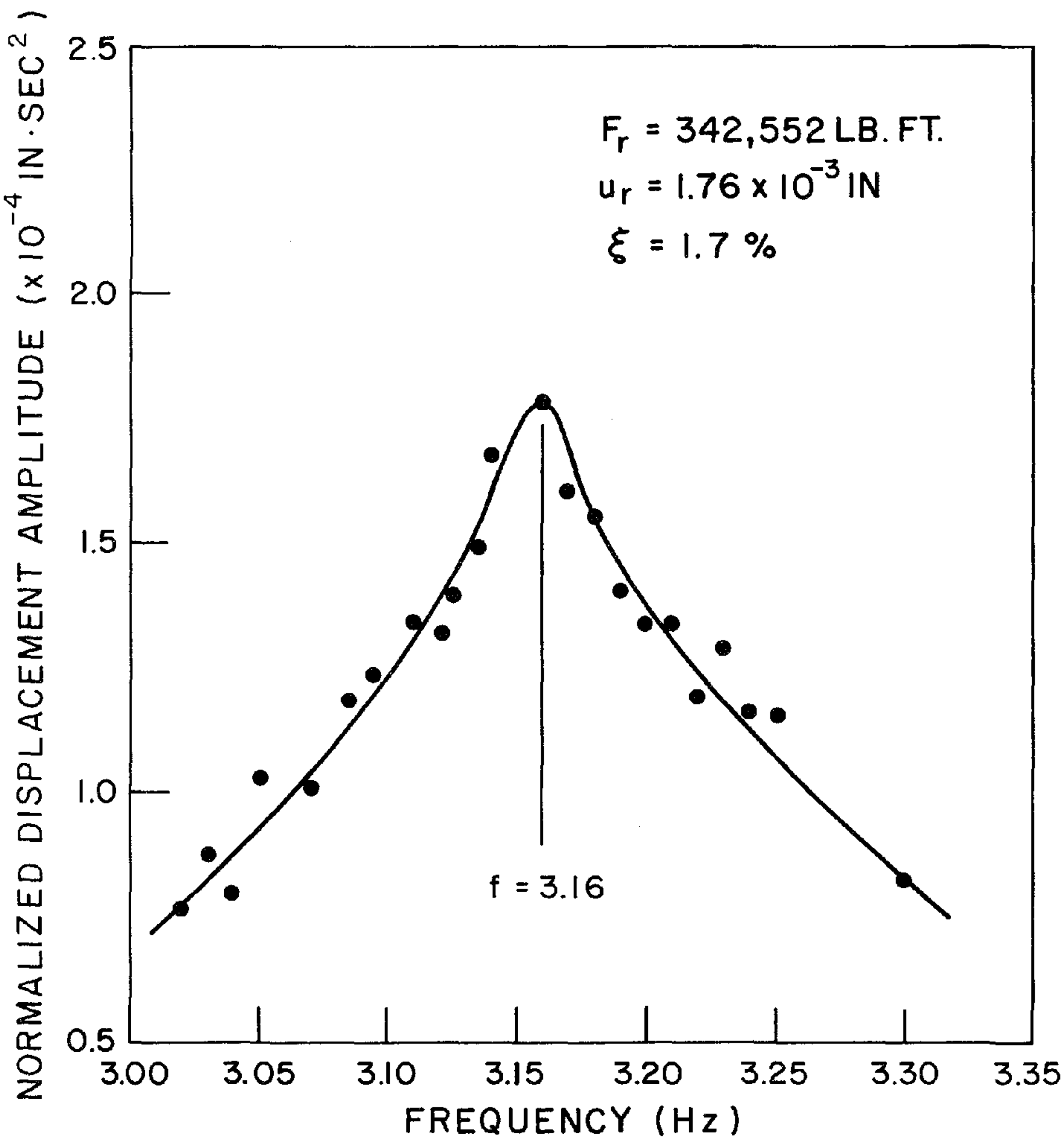


FIG. 3.18 FREQUENCY RESPONSE, THIRD TORSIONAL MODE

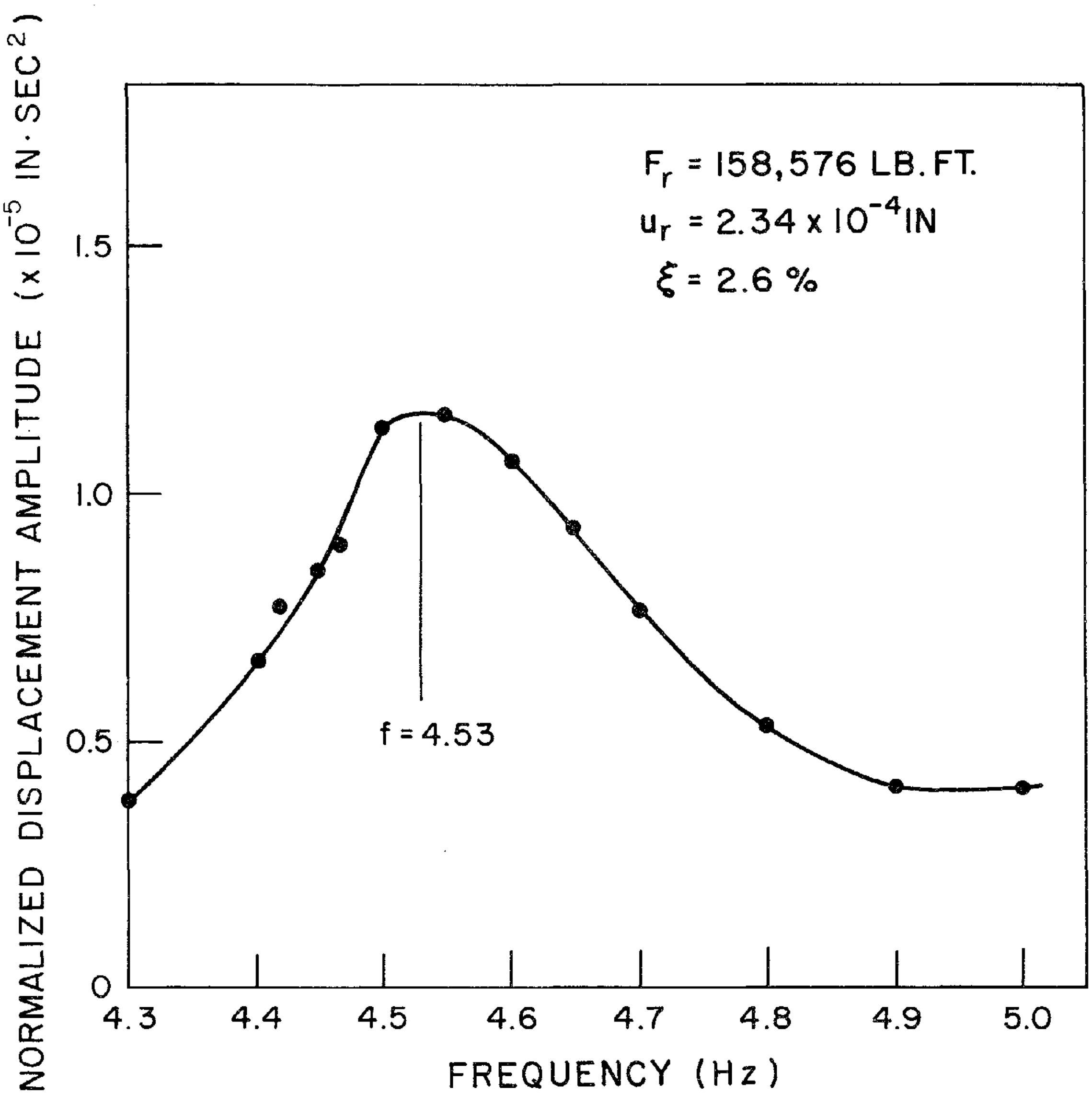


FIG. 3.19 FREQUENCY RESPONSE, FOURTH TORSIONAL MODE

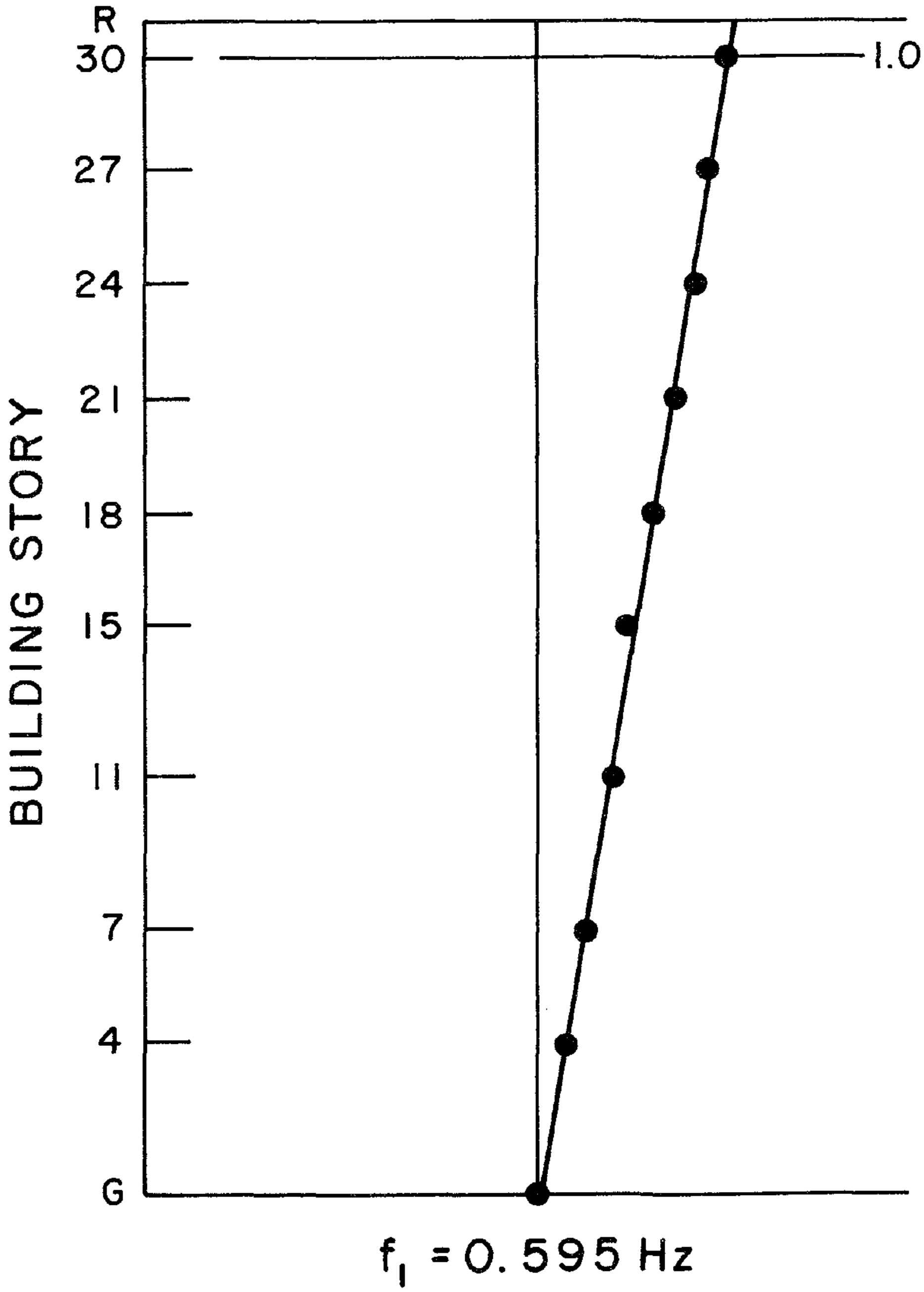


FIG. 3.20 MODE SHAPES, FIRST TRANSLATIONAL MODE E-W

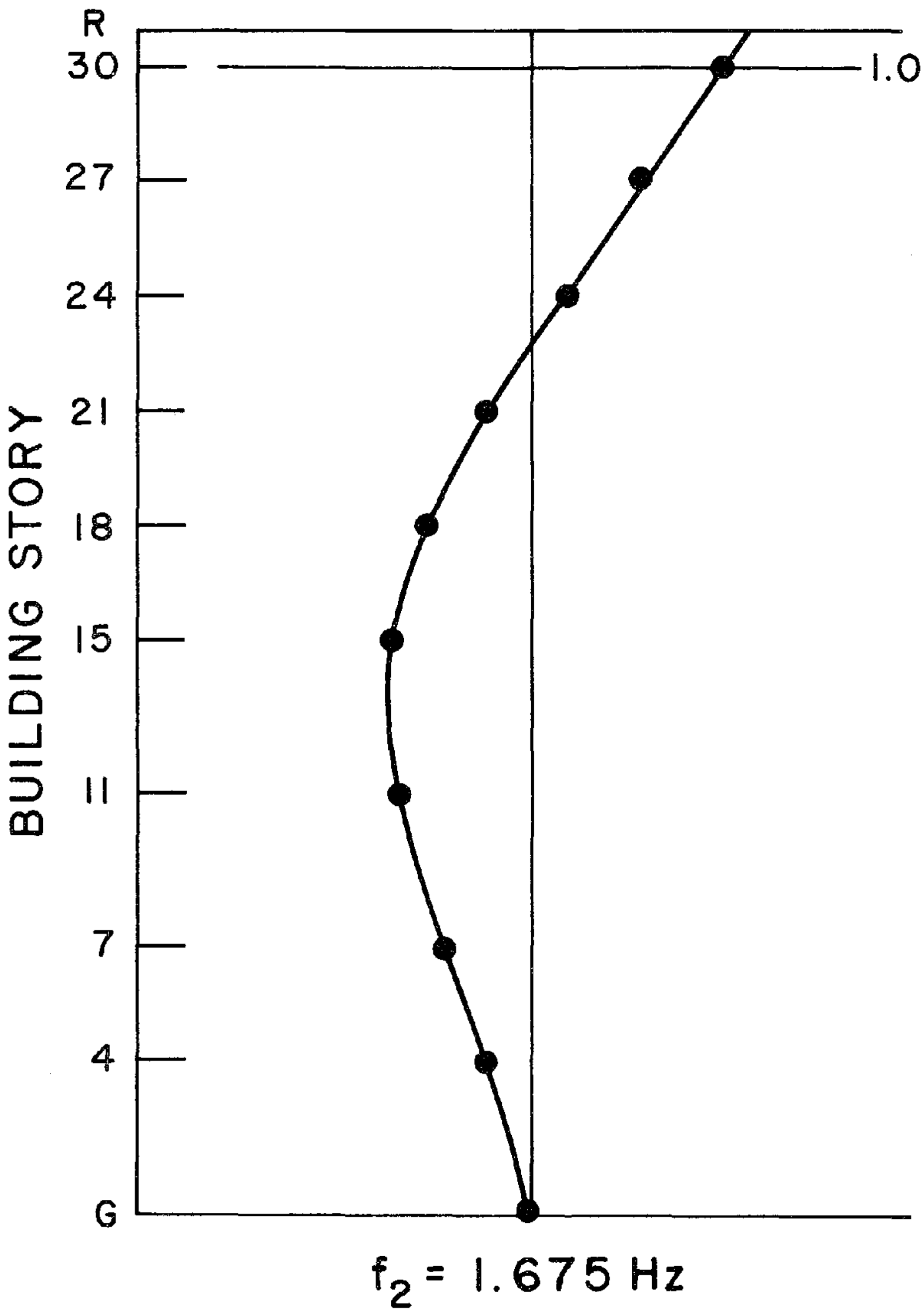


FIG. 3.21 MODE SHAPES, SECOND TRANSLATIONAL MODE E-W

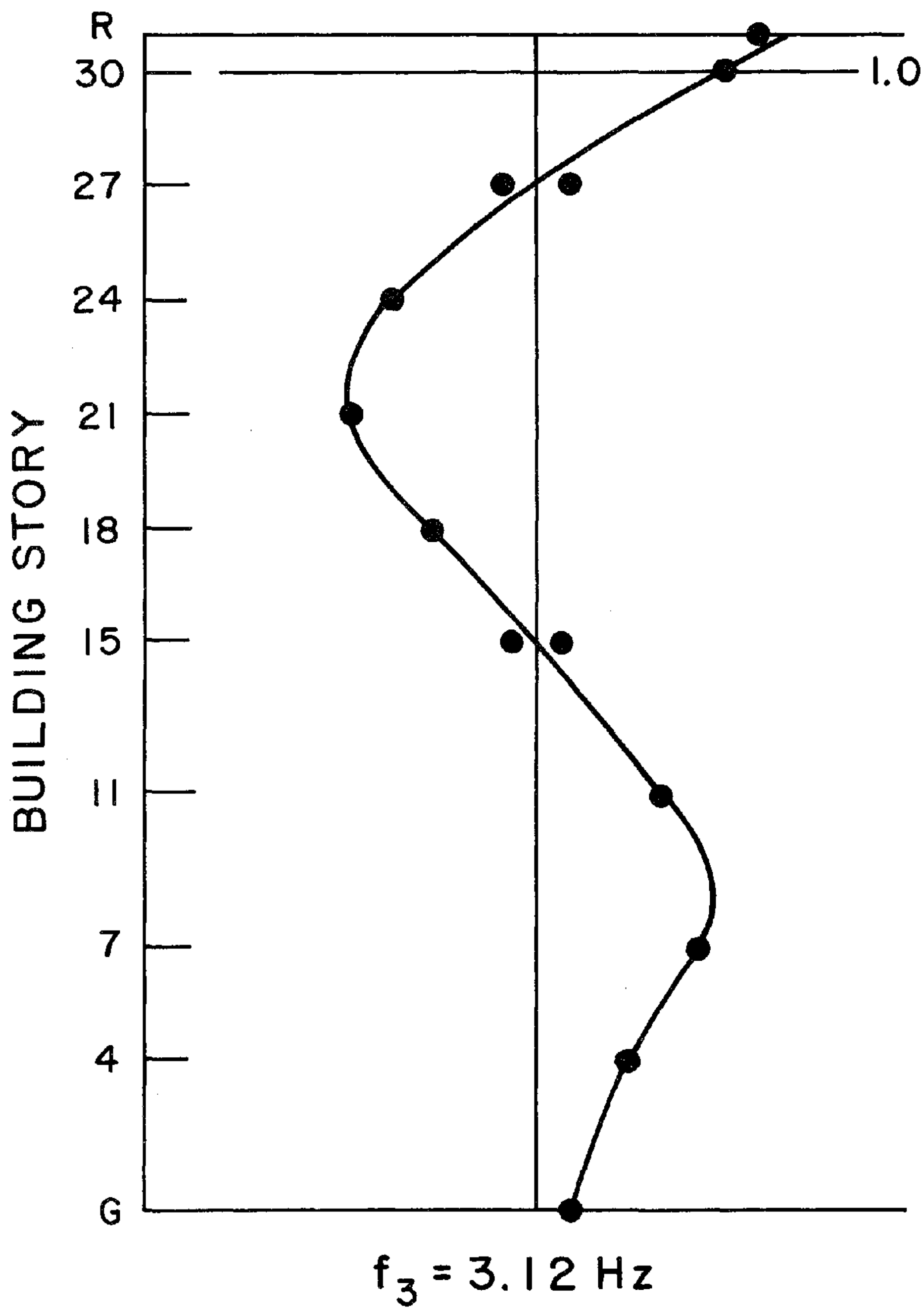


FIG. 3.22 MODE SHAPES, THIRD TRANSLATIONAL MODE E-W

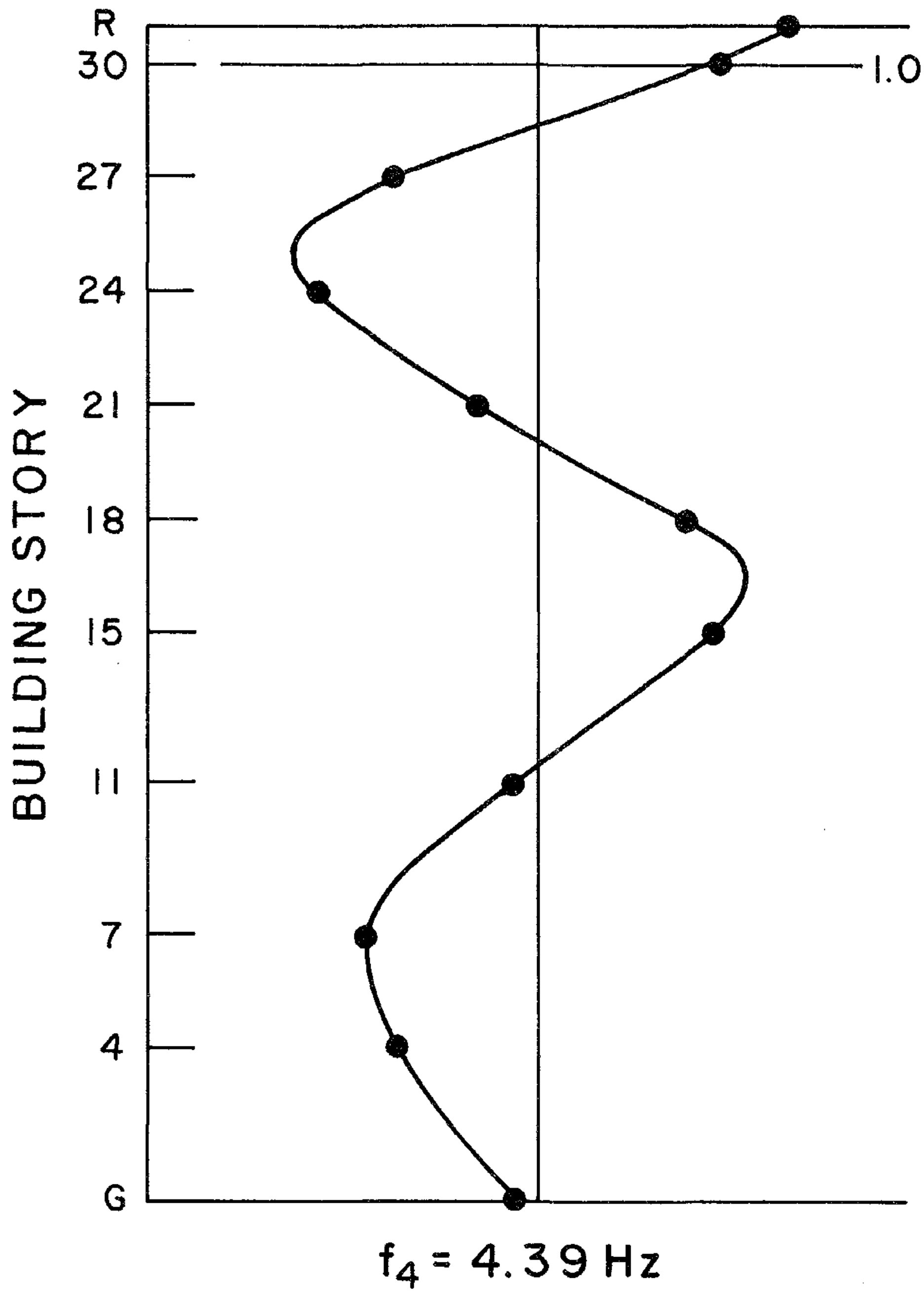


FIG. 3.23 MODE SHAPES, FOURTH TRANSLATIONAL MODE E-W

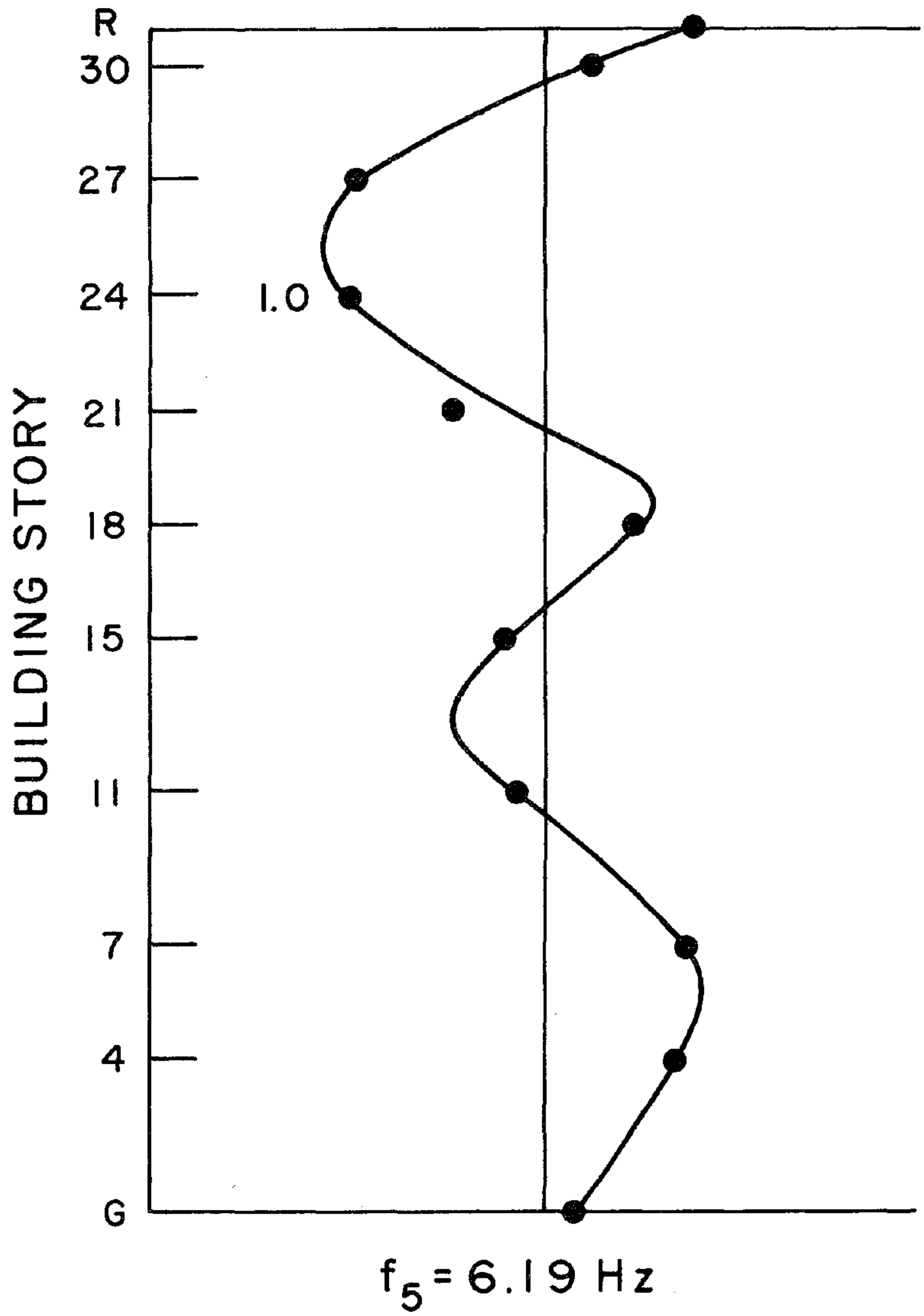


FIG. 3.24 MODE SHAPES, FIFTH TRANSLATIONAL MODE E-W

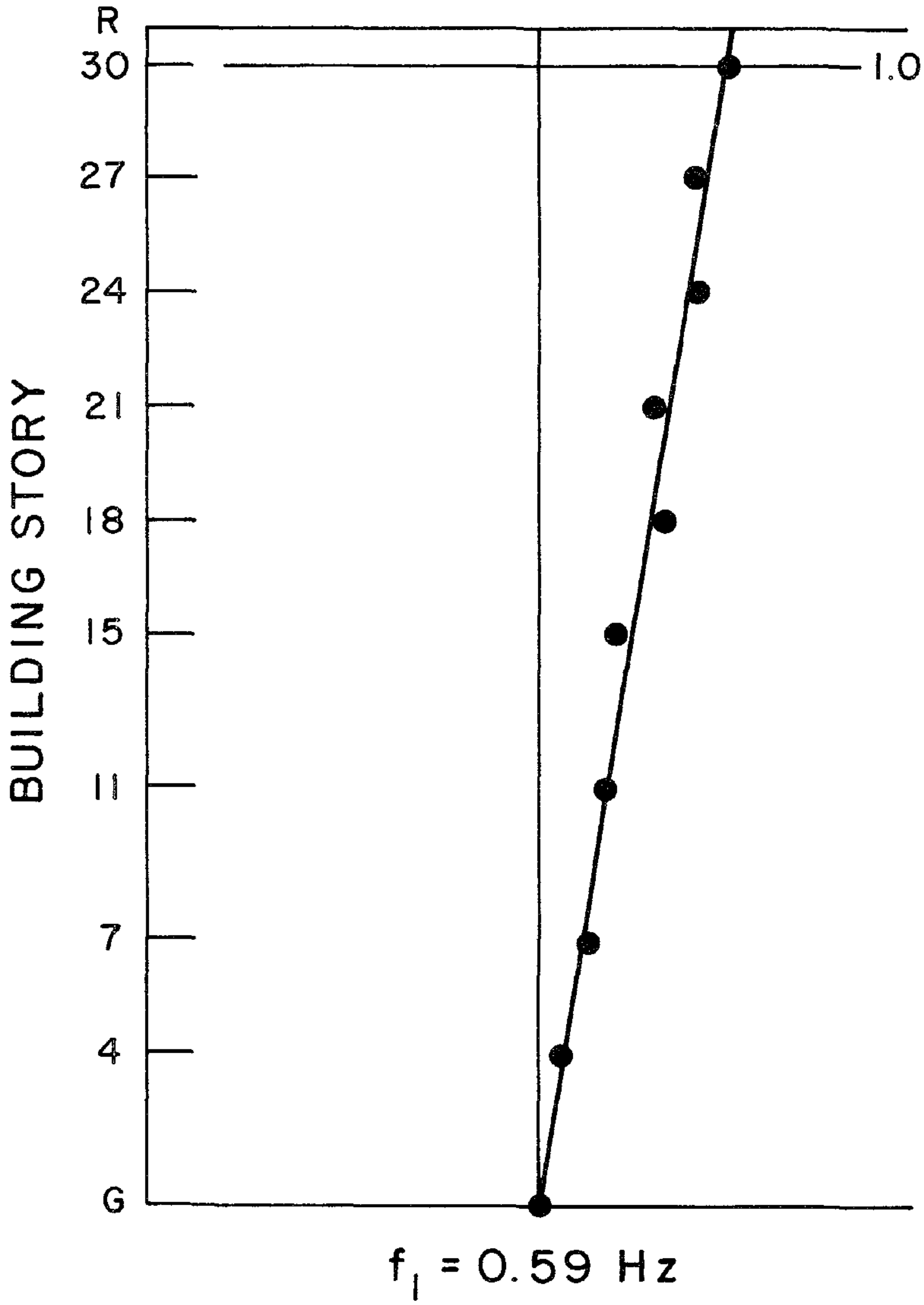


FIG. 3.25 MODE SHAPES, FIRST TRANSLATIONAL MODE N-S

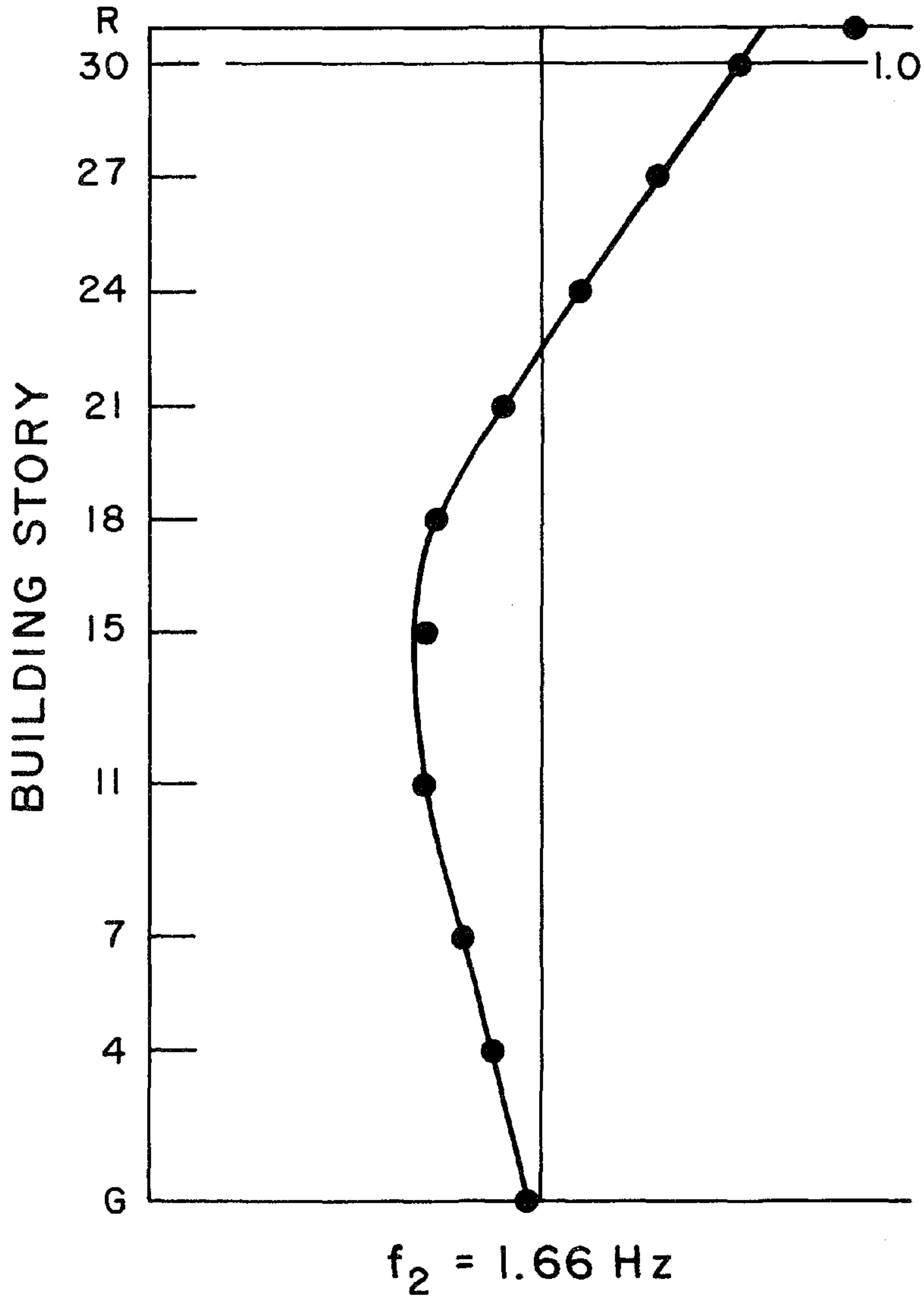


FIG. 3.26 MODE SHAPES, SECOND TRANSLATIONAL MODE N-S

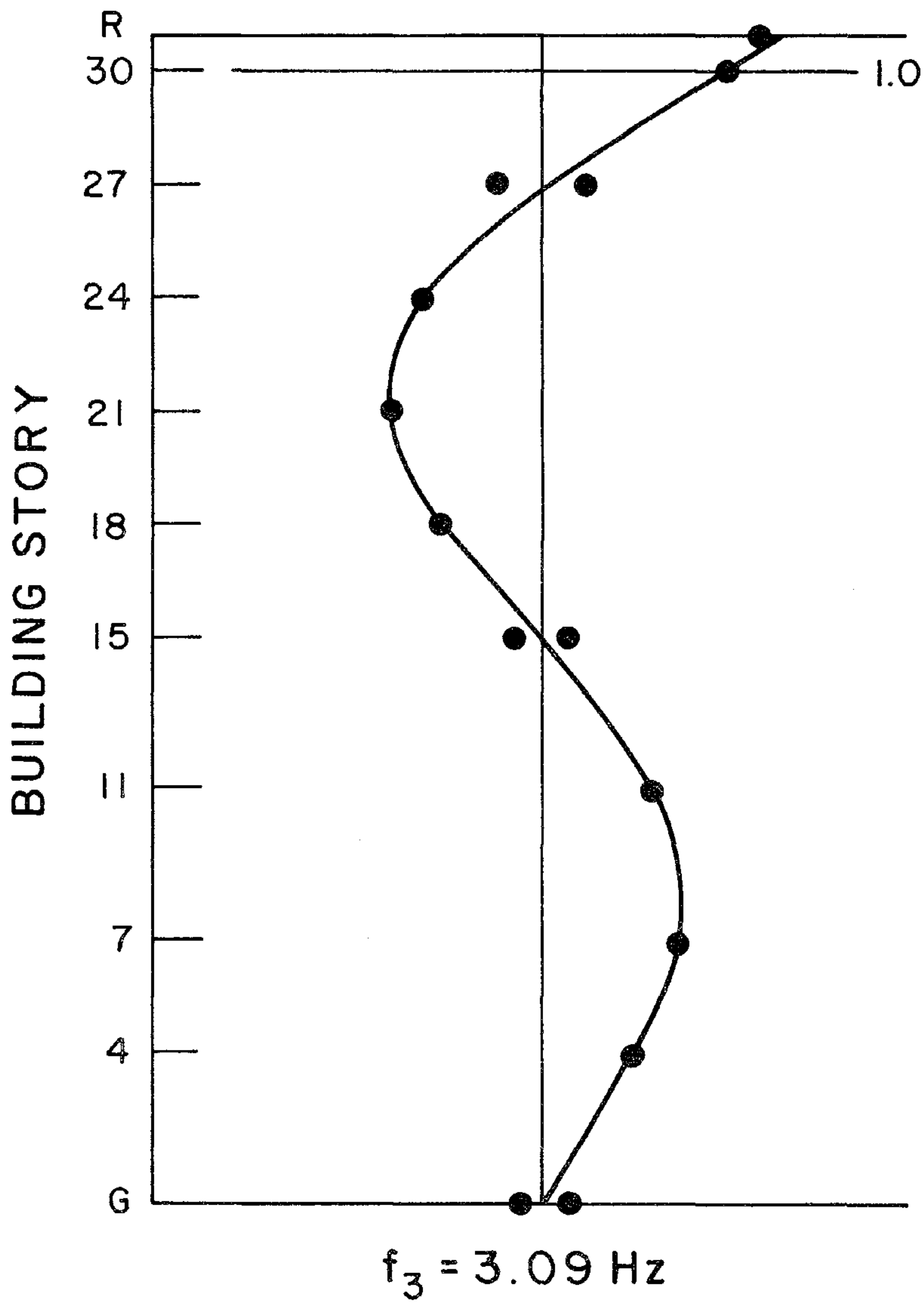


FIG. 3.27 MODE SHAPES, THIRD TRANSLATIONAL MODE N-S

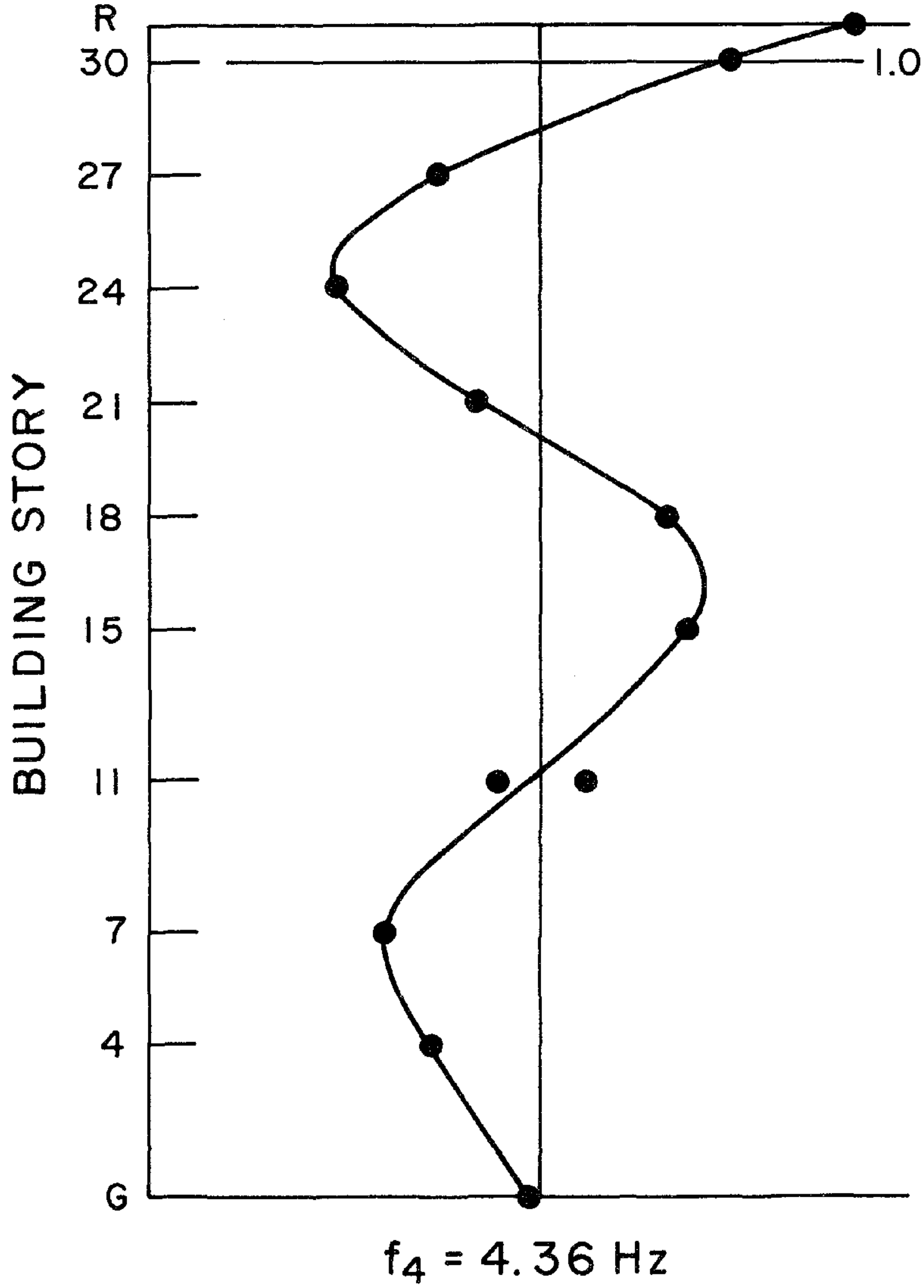


FIG. 3.28 MODE SHAPES, FOURTH TRANSLATIONAL MODE N-S

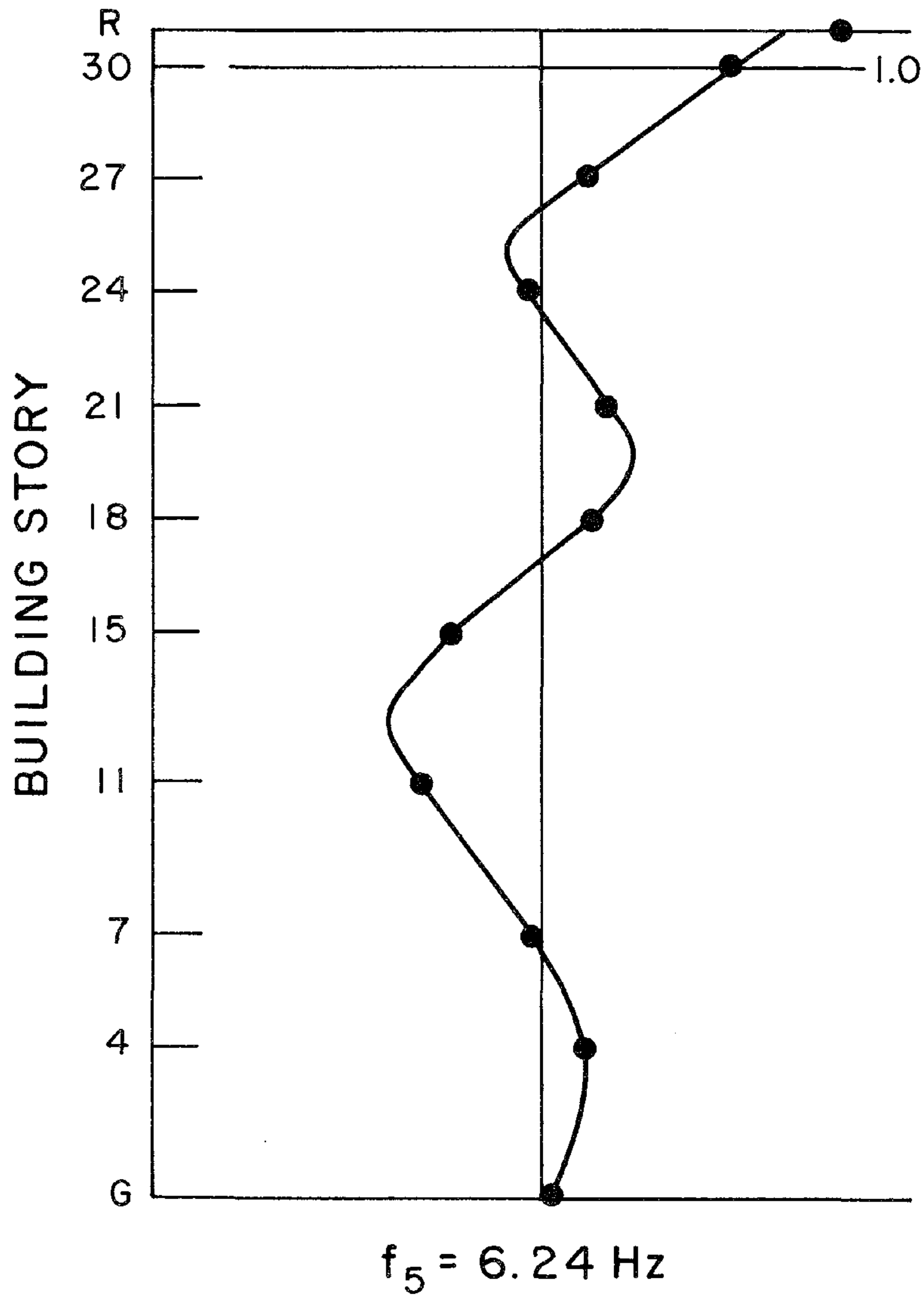


FIG. 3.29 MODE SHAPES, FIFTH TRANSLATIONAL MODE N-S

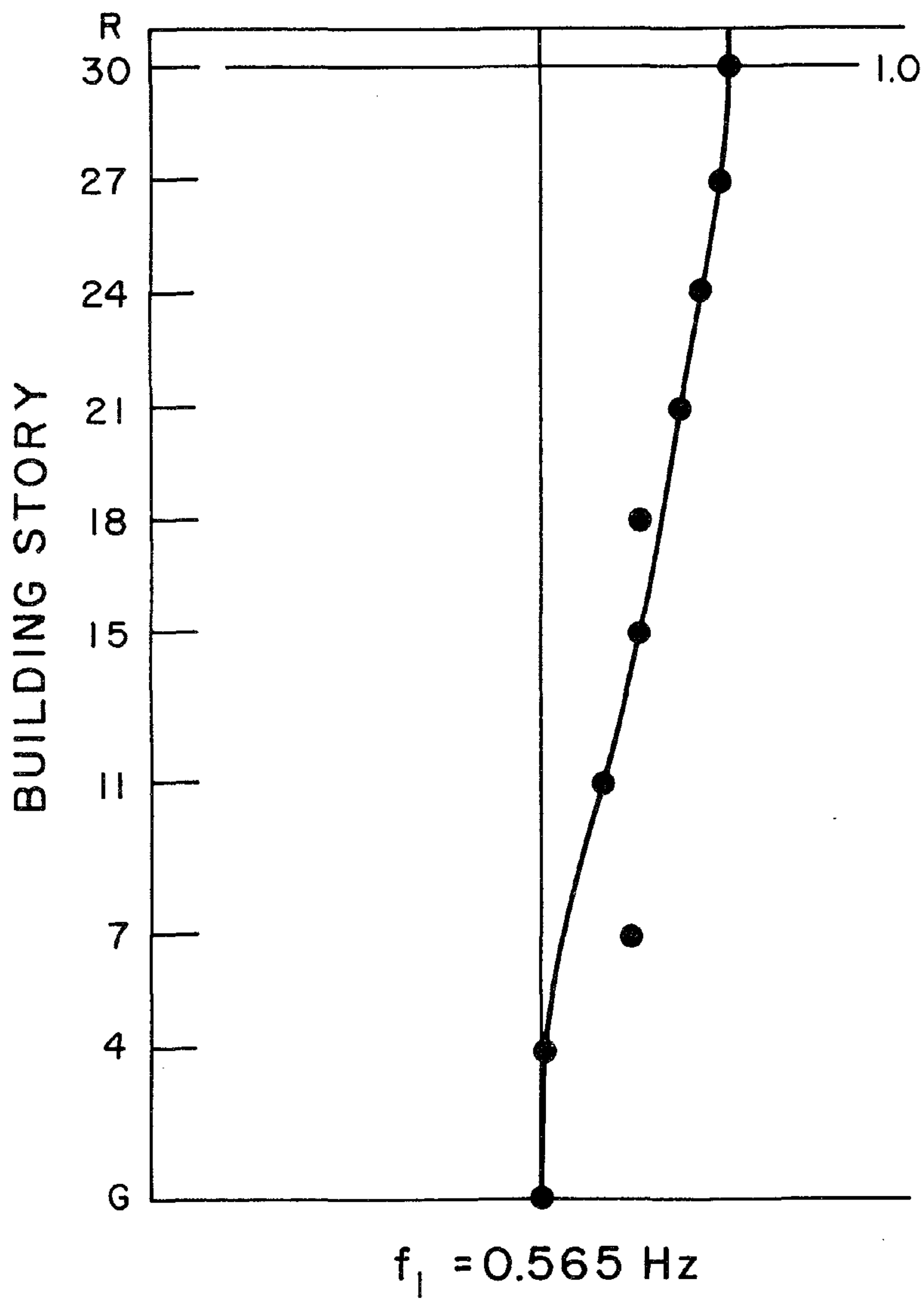


FIG. 3.30 MODE SHAPES, FIRST TORSIONAL MODE N-S

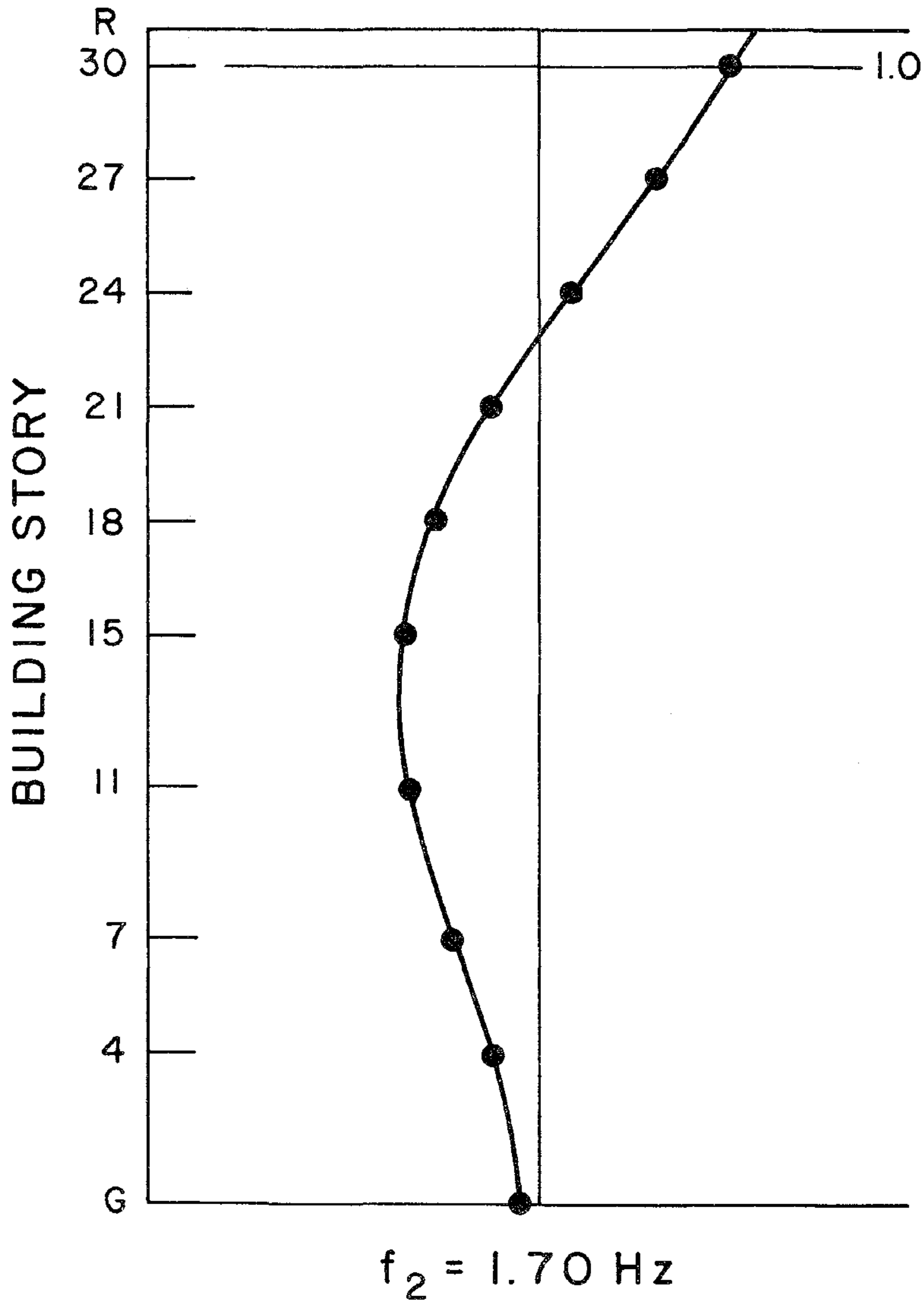


FIG. 3.31 MODE SHAPES, SECOND TORSIONAL MODE

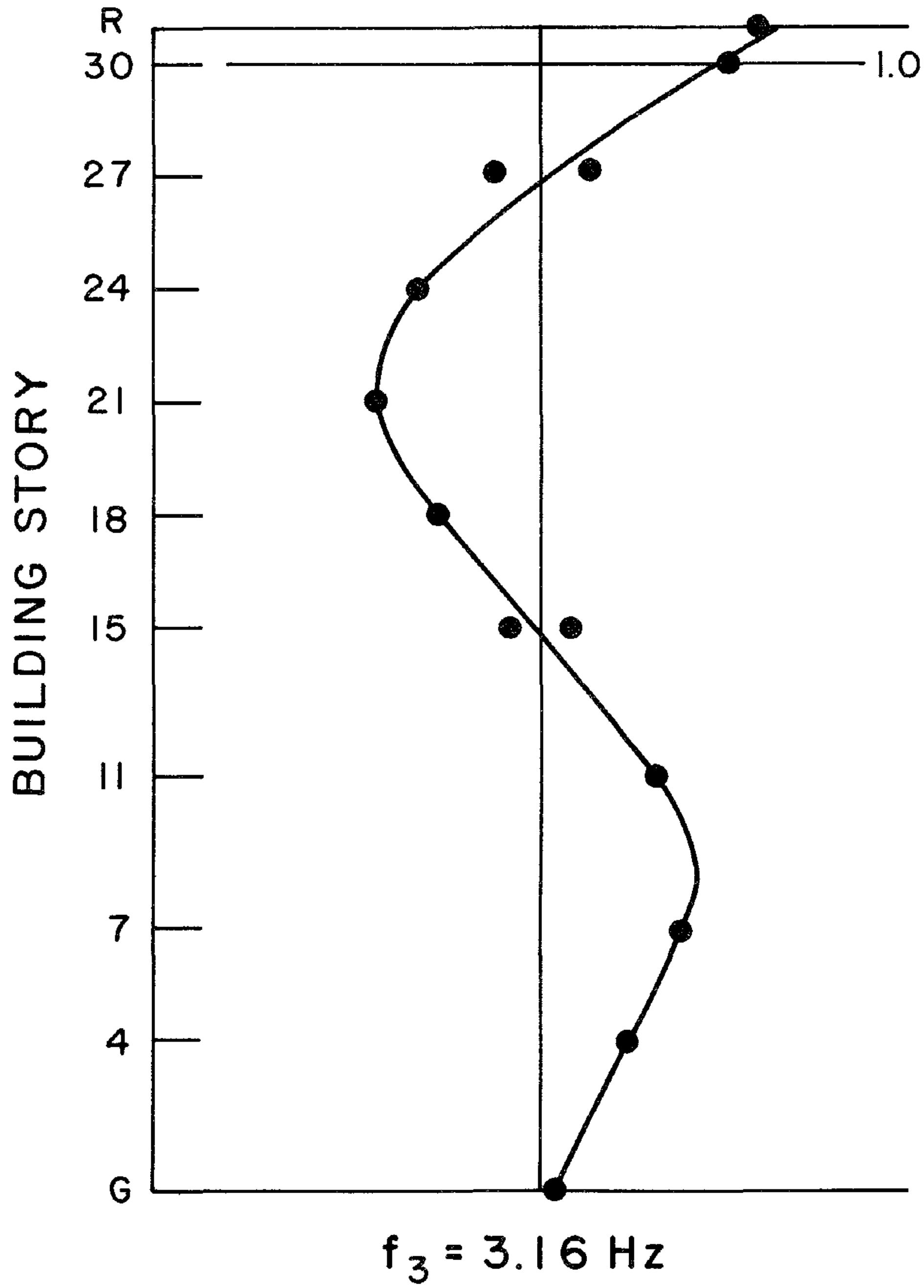


FIG. 3.32 MODE SHAPES, THIRD TORSIONAL MODE

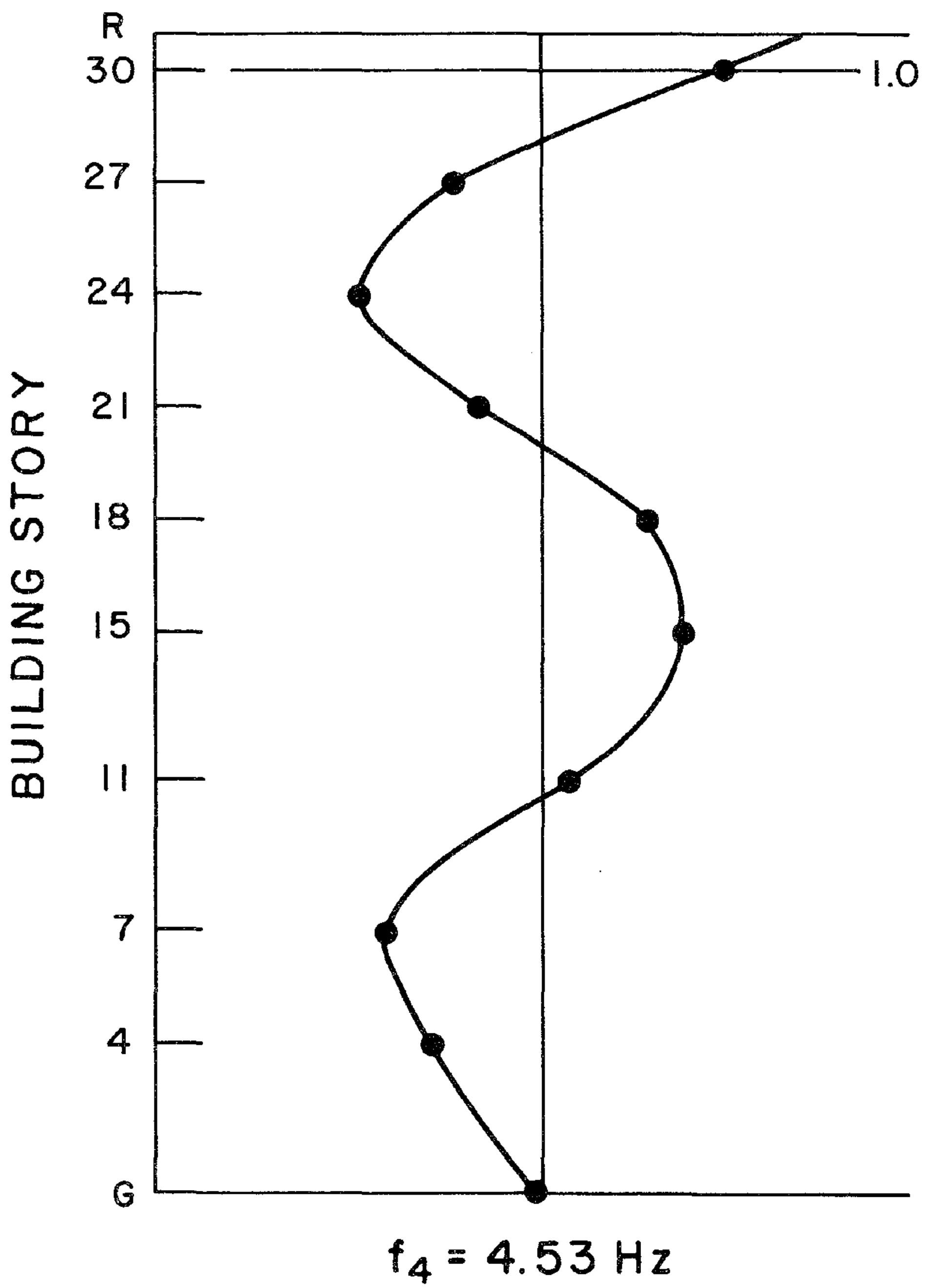


FIG. 3.33 MODE SHAPES, FOURTH TORSIONAL MODE

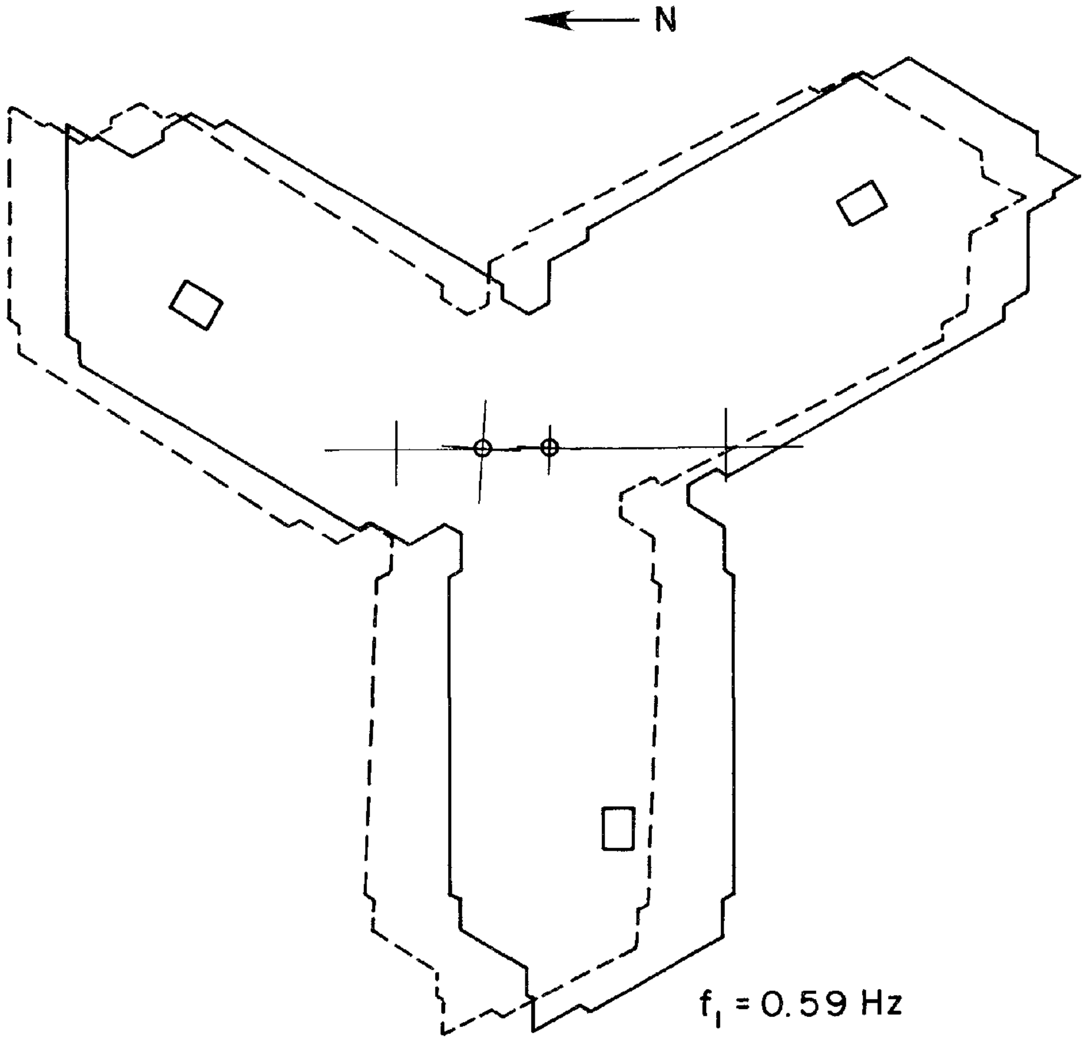


FIG. 3.34 30TH FLOOR MODE SHAPE N-S FORCING

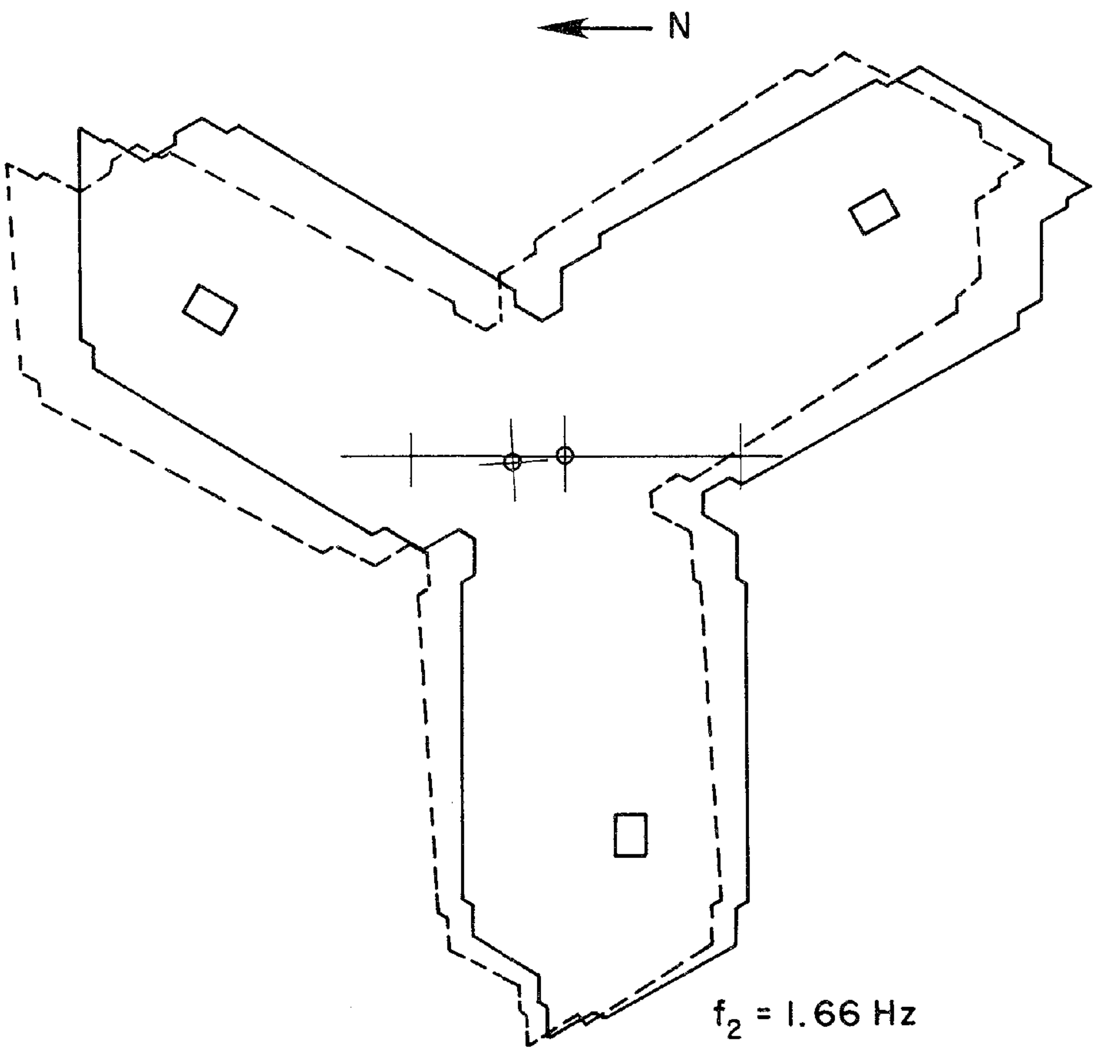


FIG. 3.35 30TH FLOOR MODE SHAPE N-S FORCING

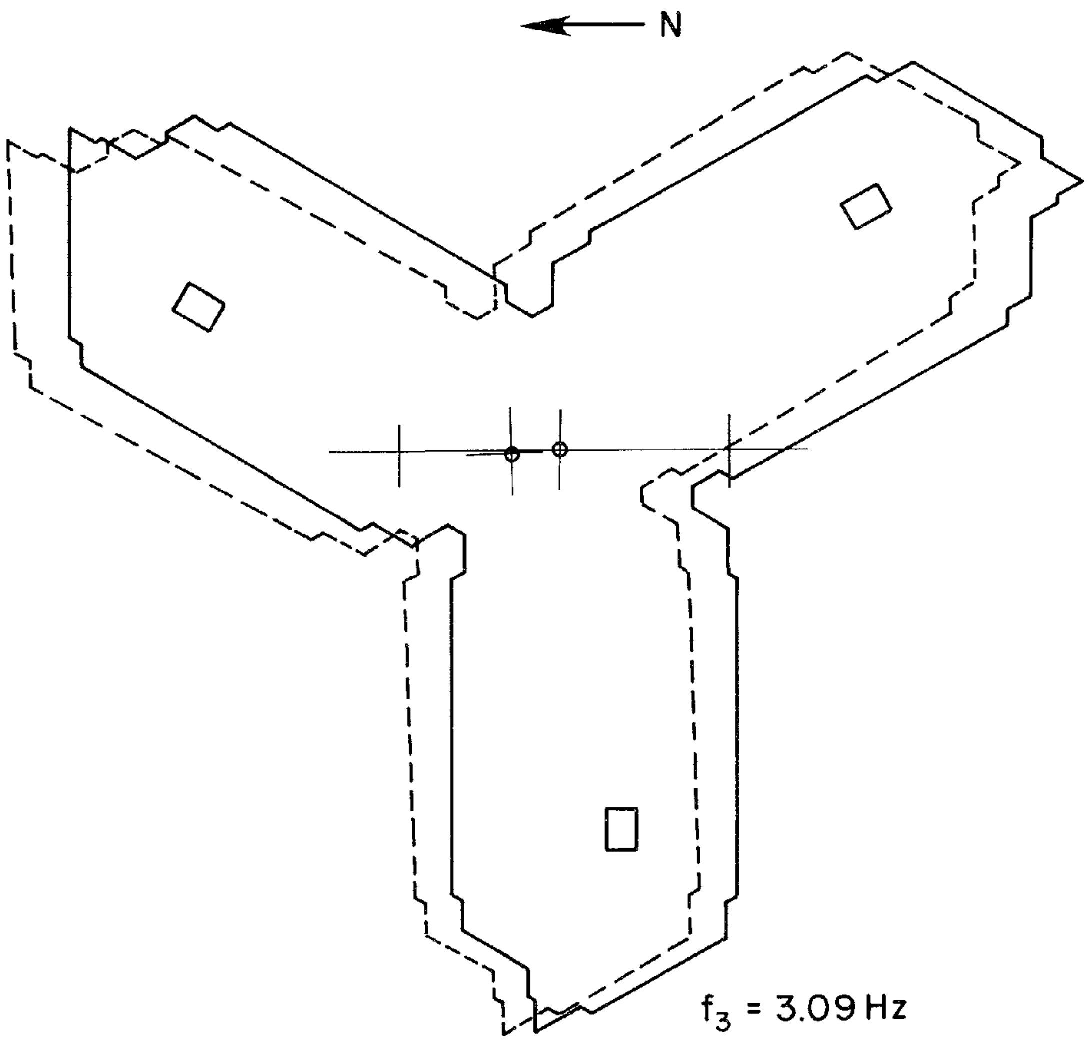


FIG. 3.36 30TH FLOOR MODE SHAPE N-S FORCING

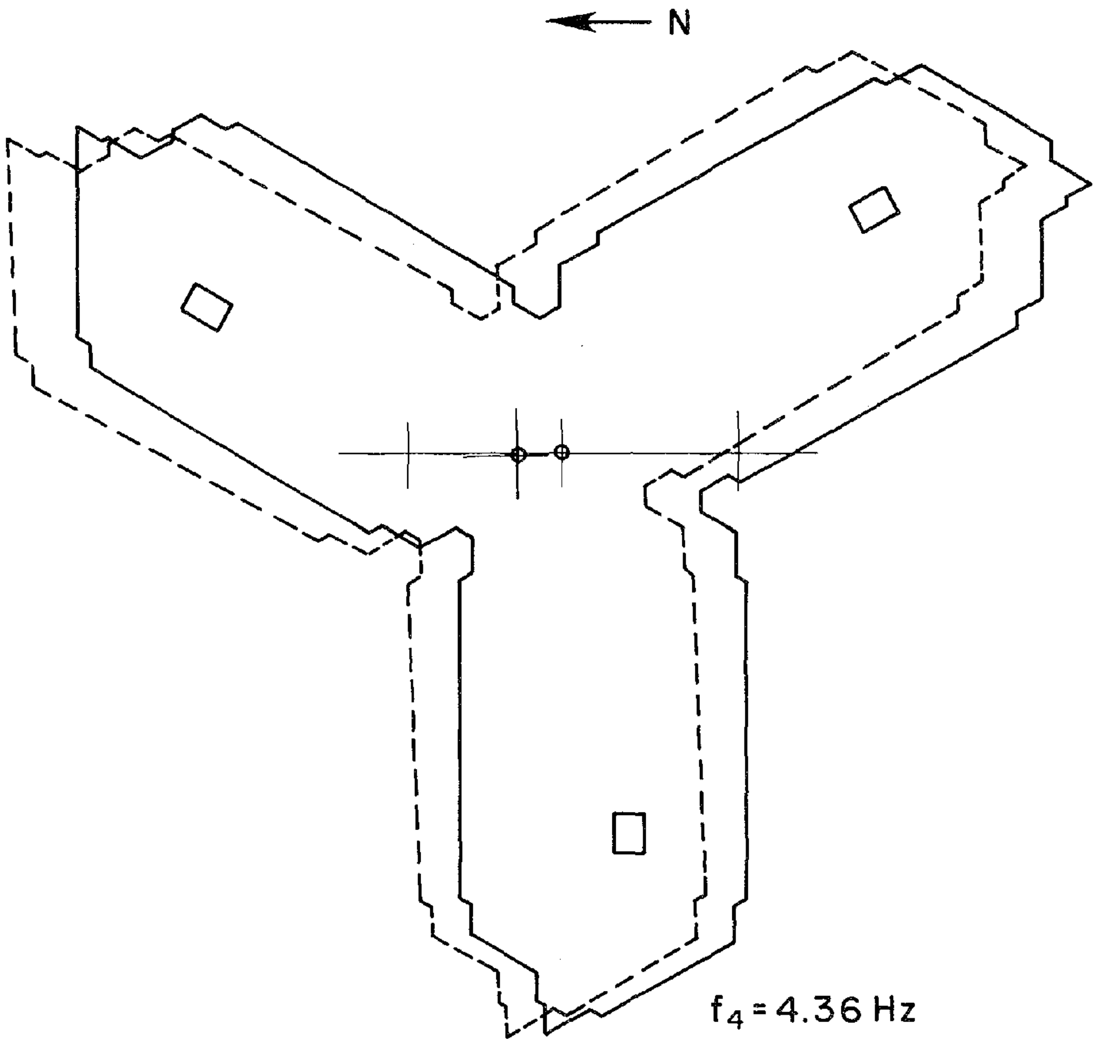


FIG. 3.37 30TH FLOOR MODE SHAPE N-S FORCING

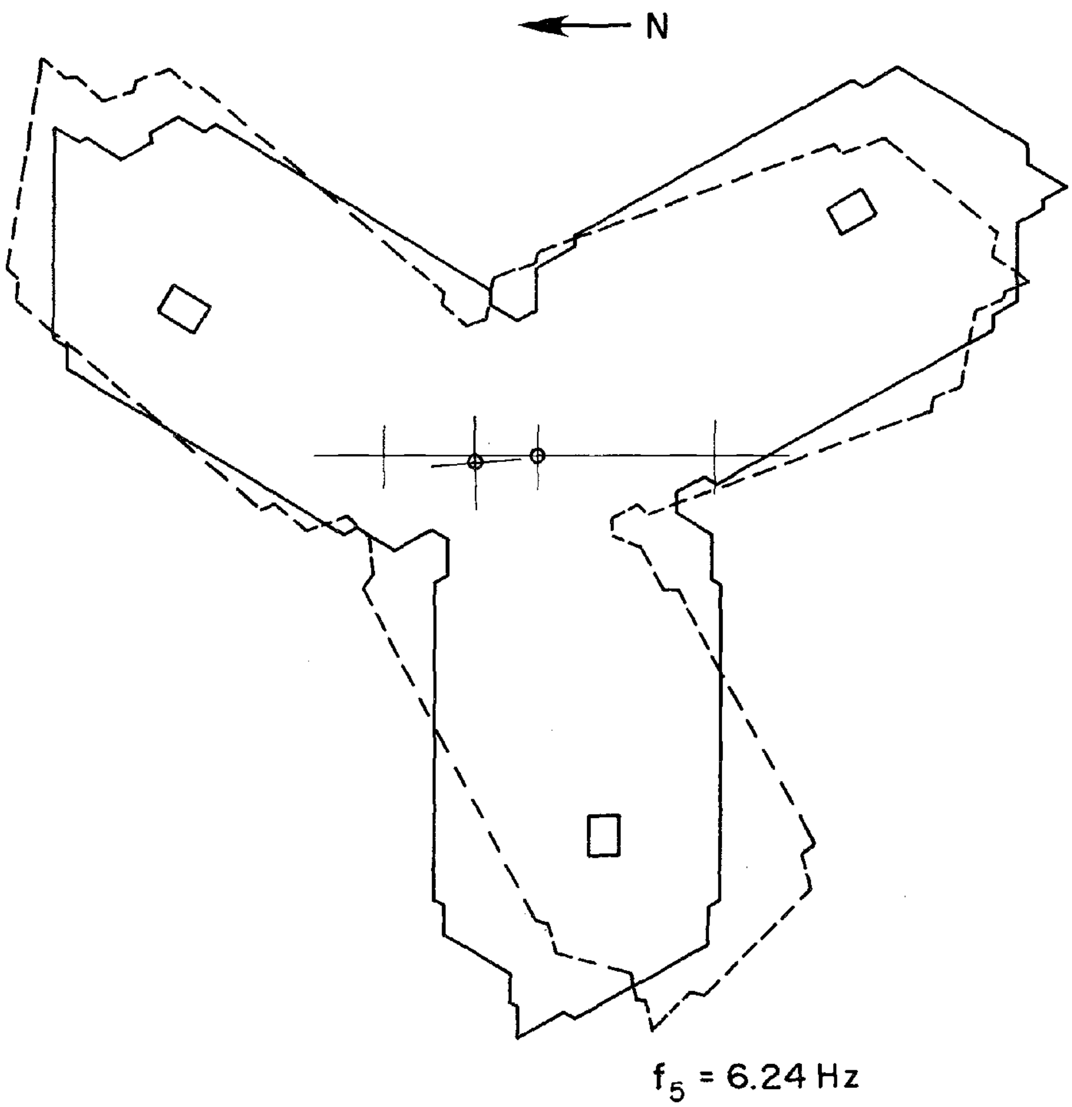


FIG. 3.38 30TH FLOOR MODE SHAPE N-S FORCING

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).

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 ambient vibration, experimental and analytical procedures were first suggested by Crawford and Ward (7, 11). 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.

Two Kinemetrics Signal Conditioners, Model SC-1 (Fig. 4.1) were used to amplify and control simultaneously eight seismometer signals. The four input channels on each signal conditioner 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 conditioners allows for outputting each channel separately or for taking the sum or difference on two channels and outputting 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. The data was digitalized at 100 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 analyzing errors. Twelve analysis ranges are provided from 0-2 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, 12), 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 (13, 14). 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 30th floor stairwells in the north and south wings to measure the E-W and torsional motion and for the N-S motion the seismometers were placed near the west stairwell in the west wing and near the center core on the east side of the building. 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. 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 differences of the seismometers readings. Typically, the data was recorded for a total of 300 seconds.

For determining the translational and torsional modes, one pair of seismometers was used on the 30th floor as a reference placed near the stairwells and center core (see Fig. 4.2). The other three pair of seismometers were oriented in the same way and relocated successively at about 3 story increments to allow the evaluation of the model response over the height of the building. 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 various pairs of averaged readings provided a modal data point normalized to the 30th floor motion. Torsional modal information was obtained in a similar manner, except that the difference, rather than the sum of the seismometer signals at the floor level was used. On each channel the low pass filter was set at 20 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 volts/in/sec. Corresponding first mode acceleration and displacement were about $\pm 0.5 \times 10^{-5}$ g and $\pm 1.25 \times 10^{-4}$ inches, respectively.

4.3 Data Analysis

4.3.1 Fourier Analysis

It is convenient to use Fourier transforms to analyze low level structural vibrations (15) 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

Eight simultaneous outputs were recorded on magnetic tape during each run. All runs were digitized at a sample rate of 100 discrete points per second. Because of the high frequency filtering present in the field instrumentation, no significant frequencies above 20 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 2048 points were calculated and averaged.

For each mode shape run, 1024 data points were selected and a total of 5 transforms were taken. The Fourier amplitude spectrum was an average of the 5 transforms computed.

The spectral estimates were smoothed by 1/4, 1/2, 1/4 weights. The 2048 spectral estimates are uniformly distributed and with 100 sample/sec. gives a frequency resolution of 100/2048, or about 0.0488 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 conditions as well as under torsion. The results, together with modal data from forced vibrations are presented in Figs. 4.3 through 4.14. The 30th floor mode of vibration under ambient conditions was also determined and is shown in Figs. 4.15 through 4.18.

TABLE 4.1 RESONANT FREQUENCIES (Hz)

Excitation	Mode			
	1	2	3	4
E-W	0.586	1.685	3.125	4.443
N-S	0.586	1.685	3.149	4.517
Torsional	0.586	1.709	3.125	4.443

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 (11). Using the bandwidth method provided the dampings ratios as presented in Table 4.2.

TABLE 4.2 DAMPING RATIOS

Excitation	Mode			
	1	2	3	4
E-W	2.6	1.2	0.4	0.6
N-S	2.6	1.8	0.8	0.7
Torsional	3.8	1.4	1.0	0.4

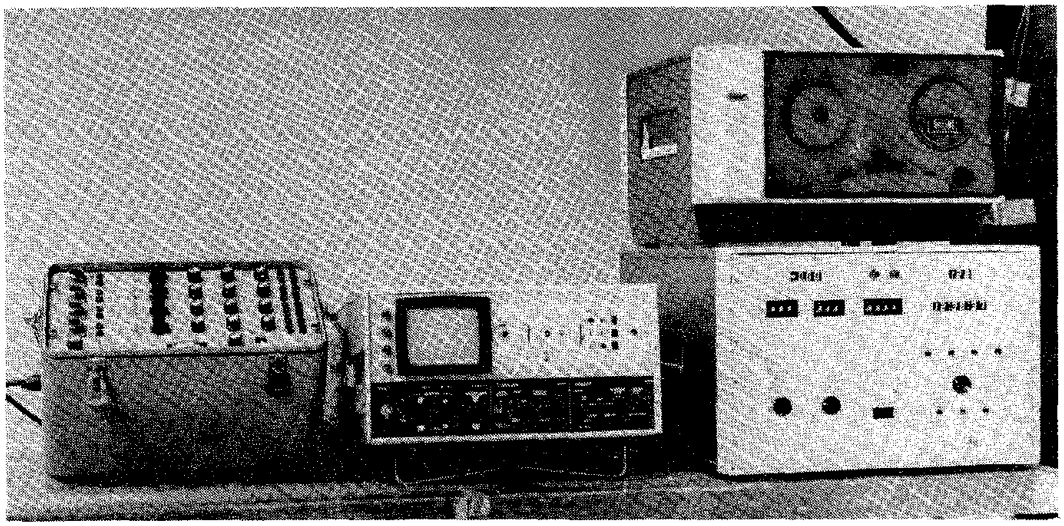


FIG. 4.1 AMBIENT VIBRATION EQUIPMENT

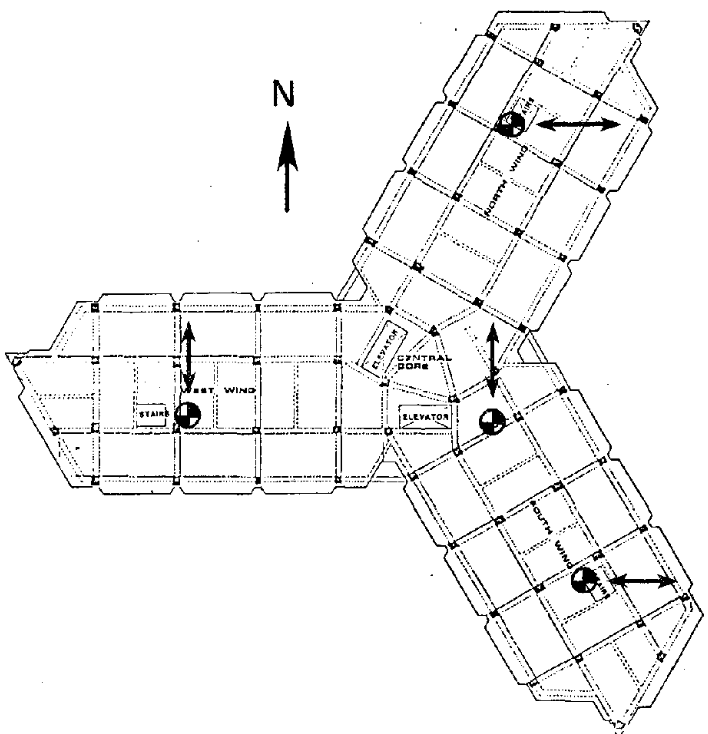


FIG. 4.2 LOCATION OF RANGER SEISMOMETERS ON THE 30TH FLOOR FOR RESONANT FREQUENCY RESPONSE

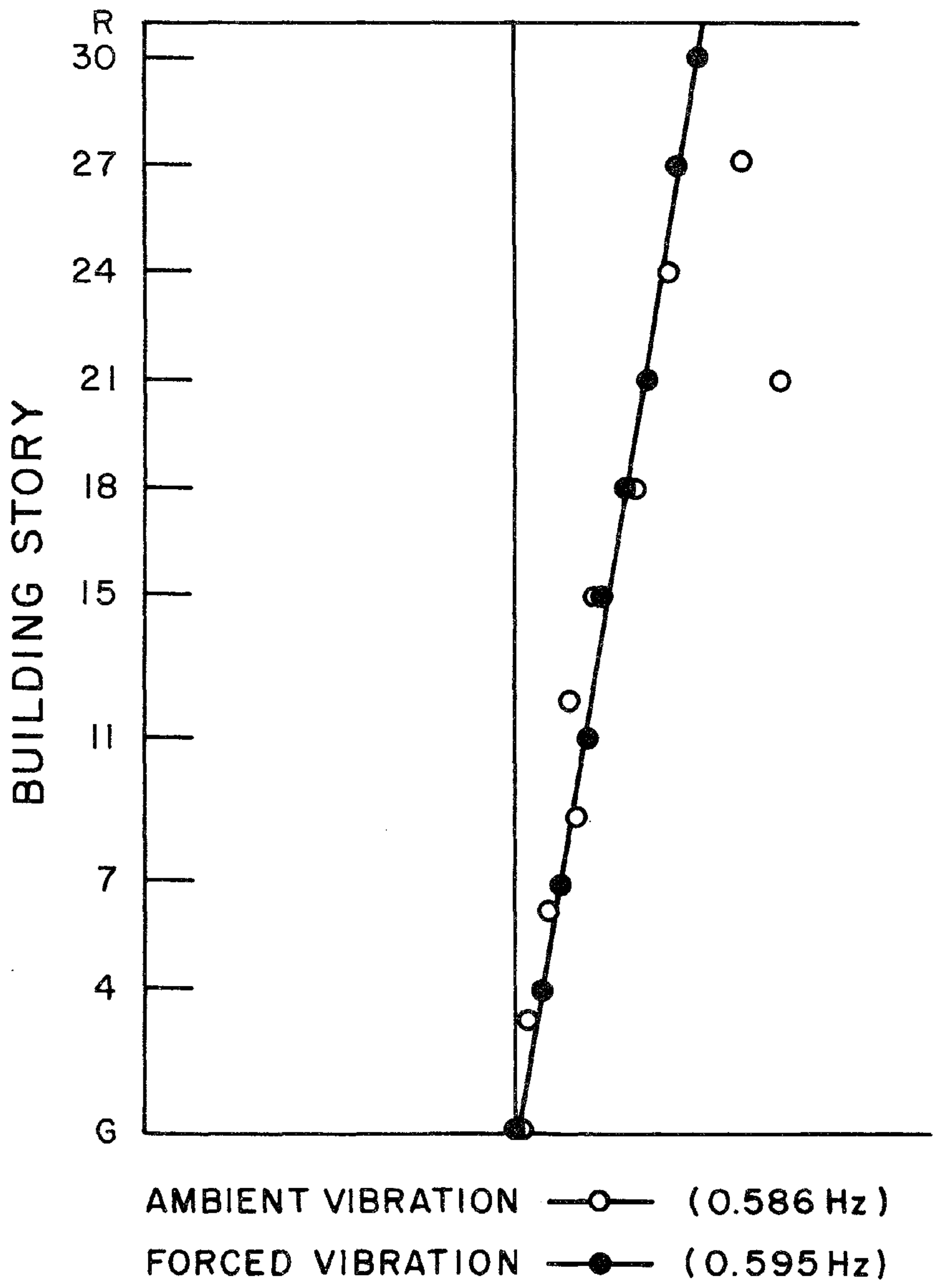


FIG. 4.3 E-W VERTICAL MODE SHAPE

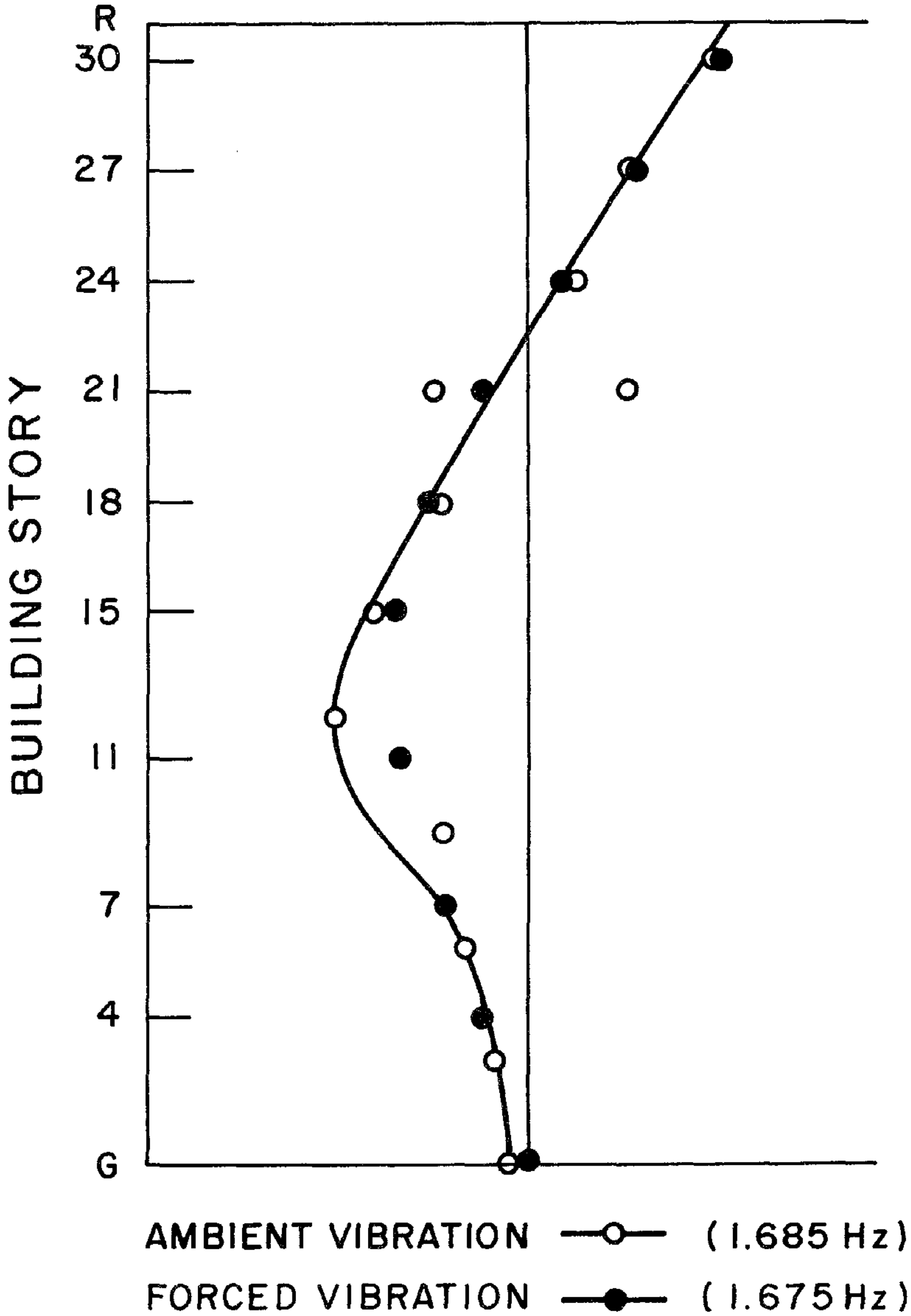


FIG. 4.4 E-W VERTICAL MODE SHAPE

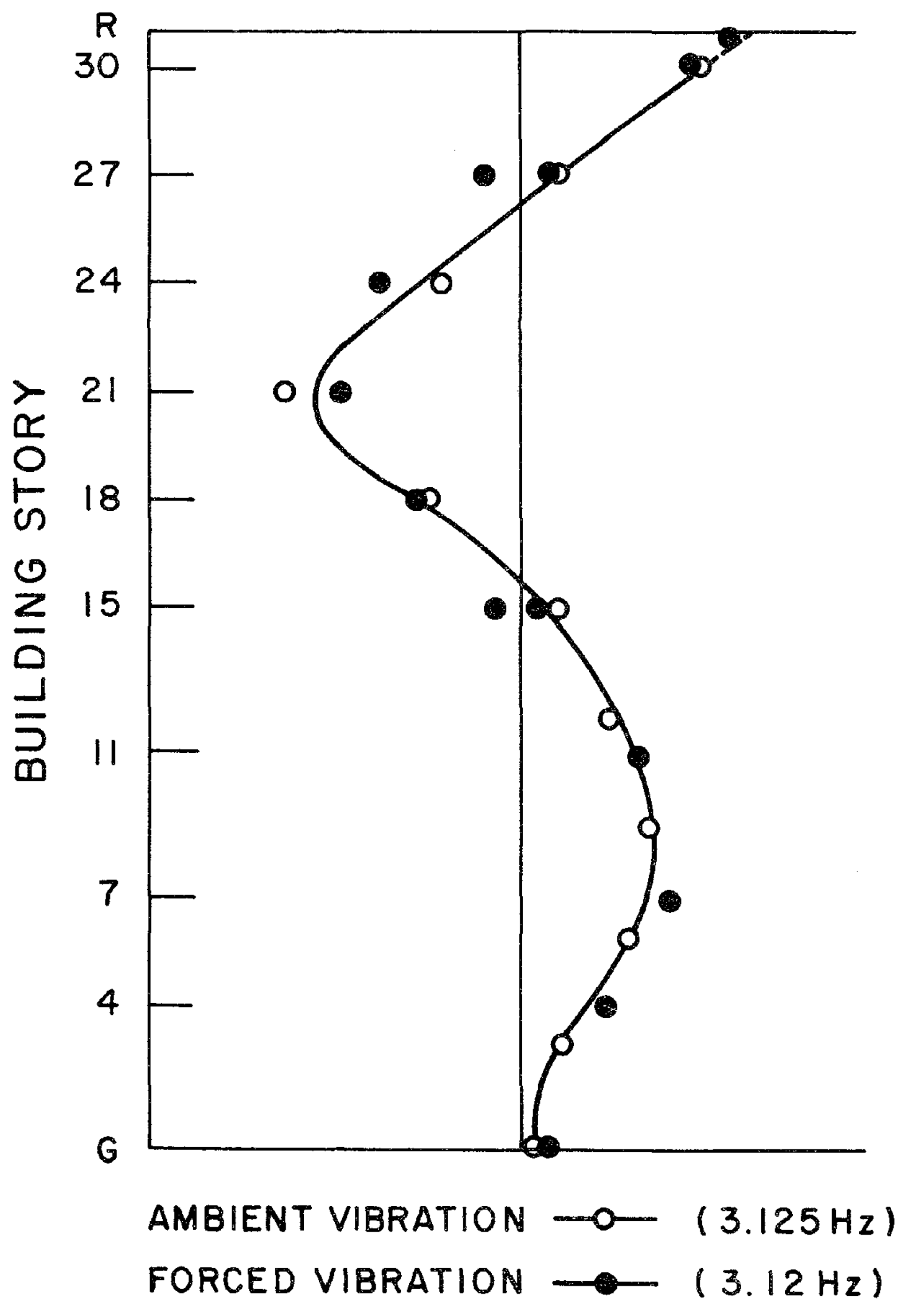


FIG. 4.5 E-W VERTICAL MODE SHAPE

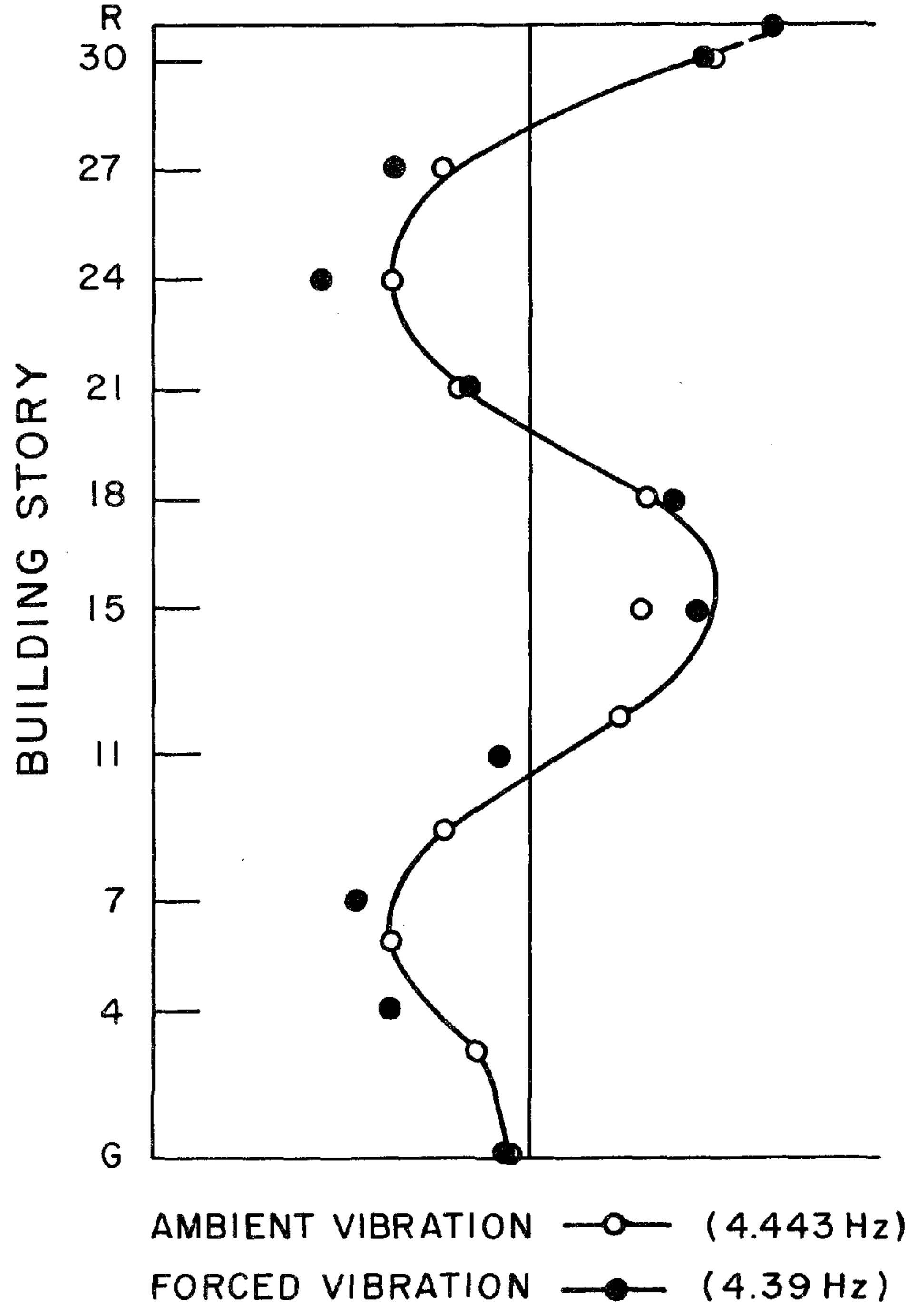


FIG. 4.6 E-W VERTICAL MODE SHAPE

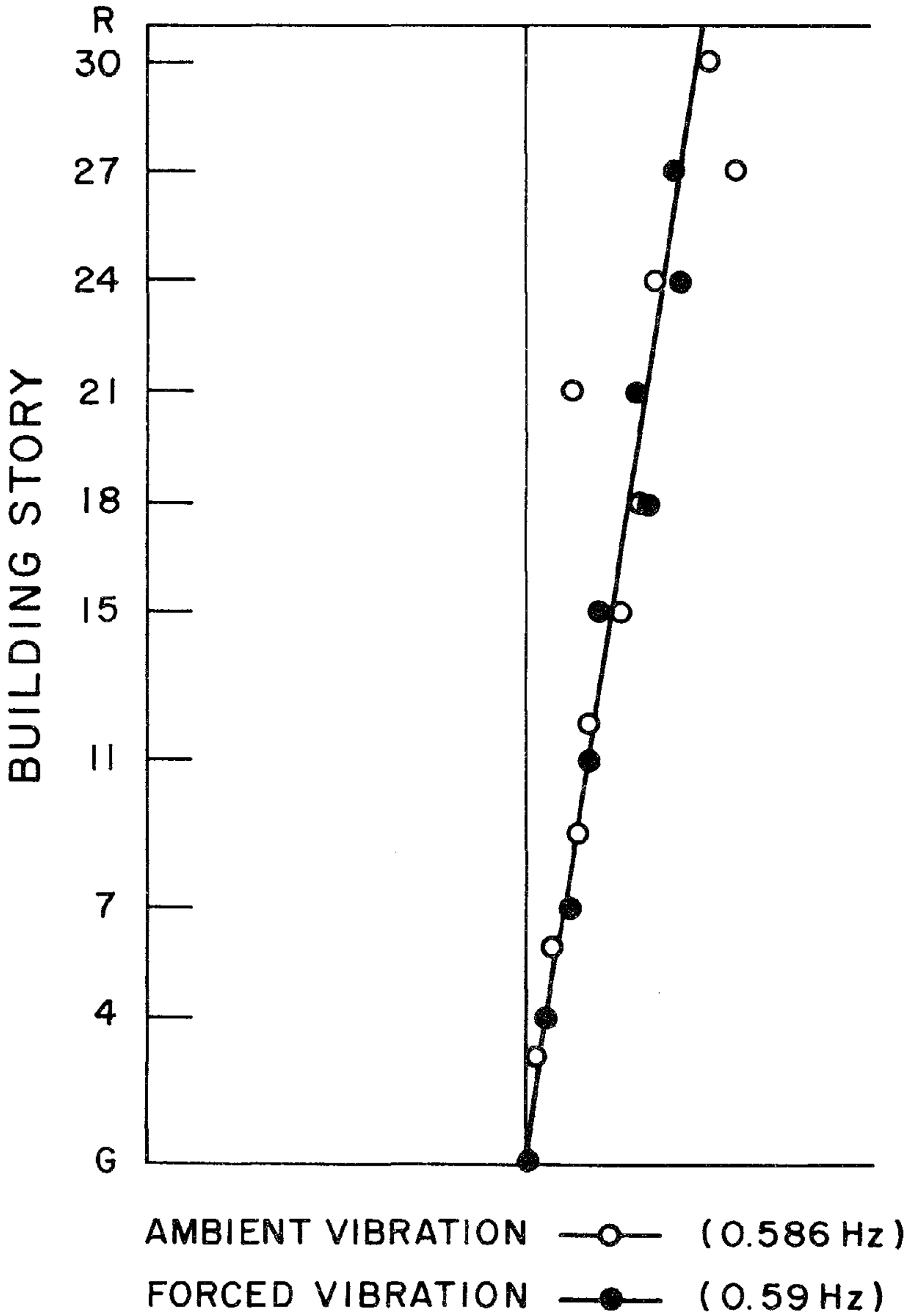


FIG. 4.7 N-S VERTICAL MODE SHAPE

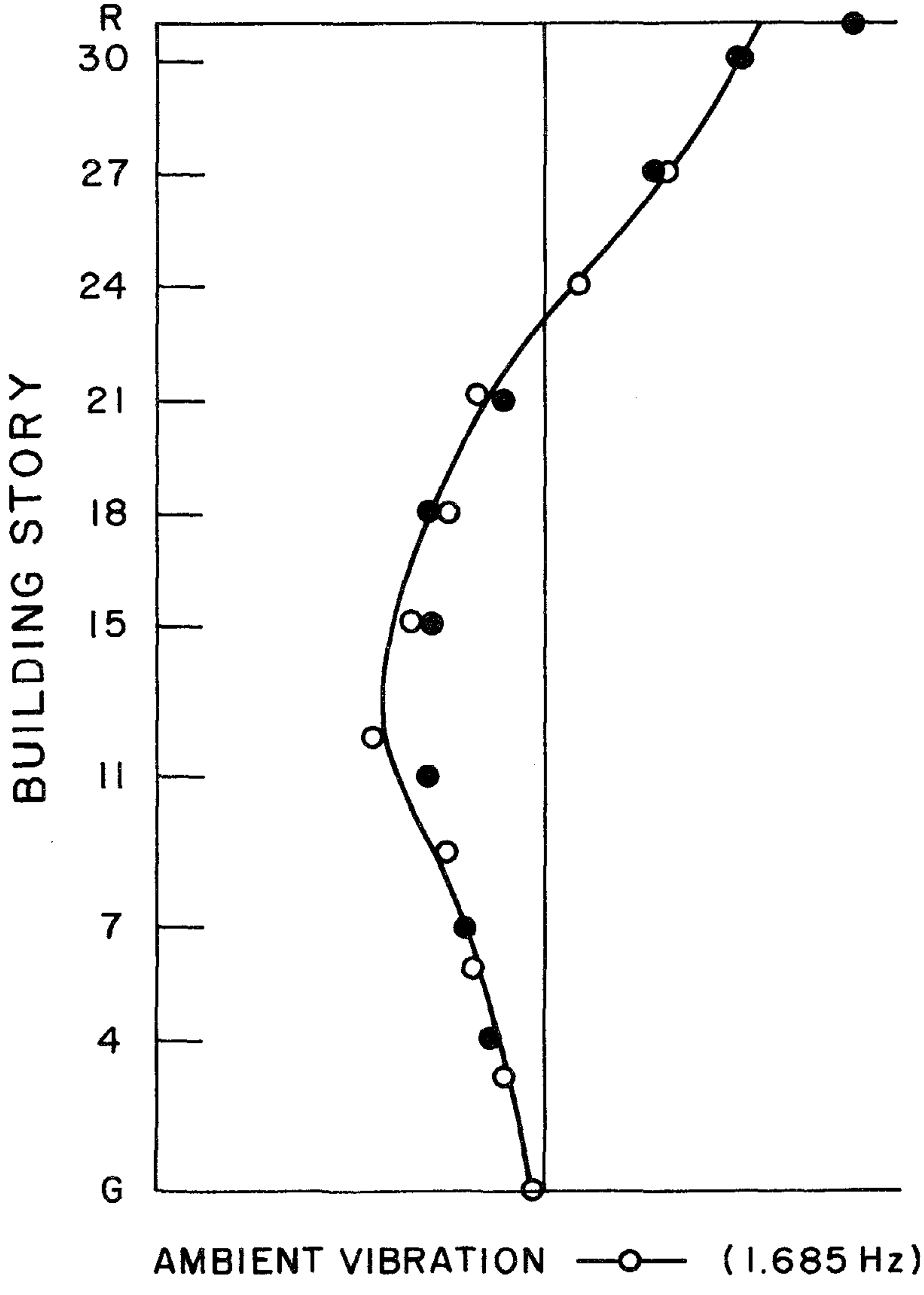


FIG. 4.8 N-S VERTICAL MODE SHAPE

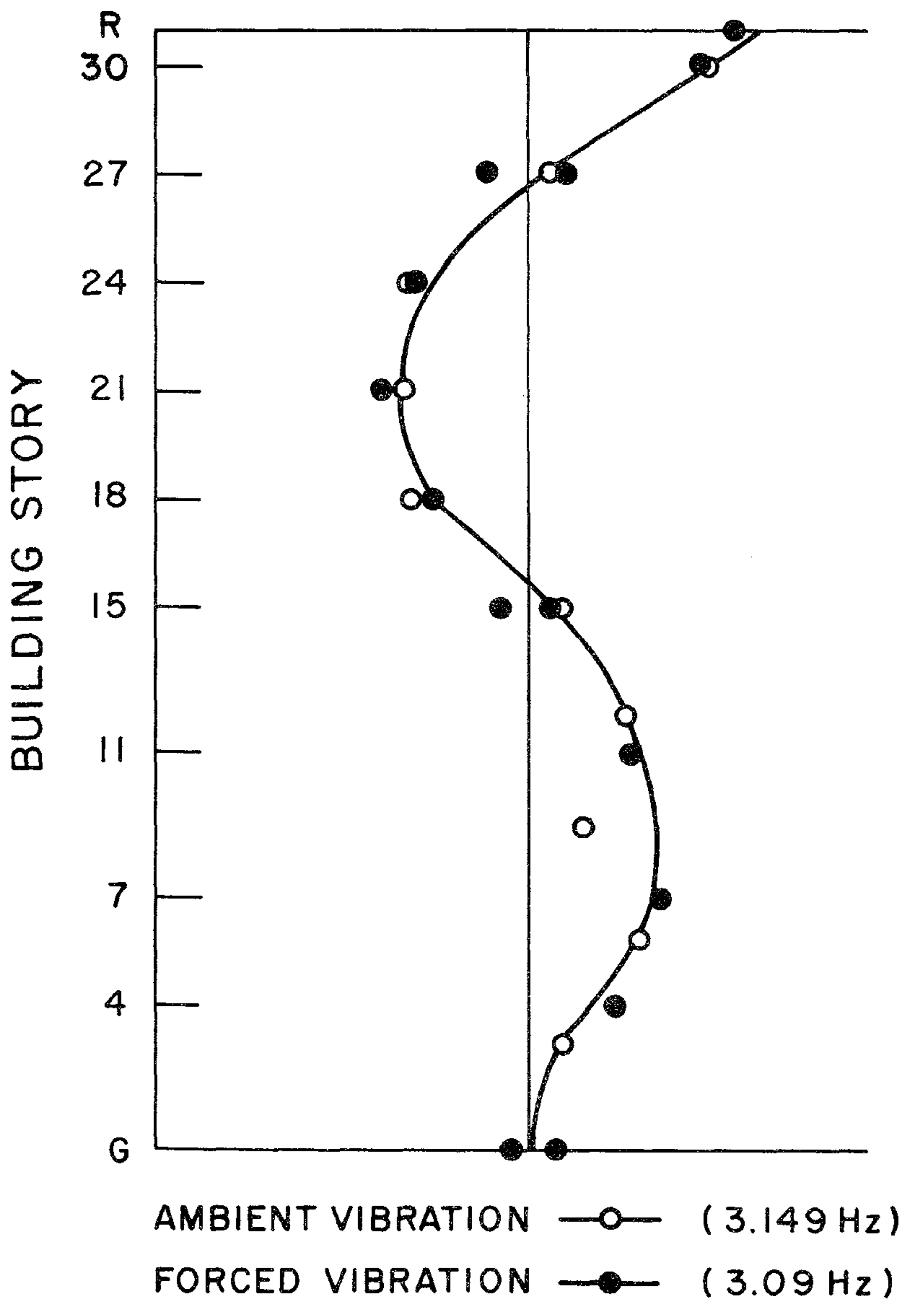


FIG. 4.9 N-S VERTICAL MODE SHAPE

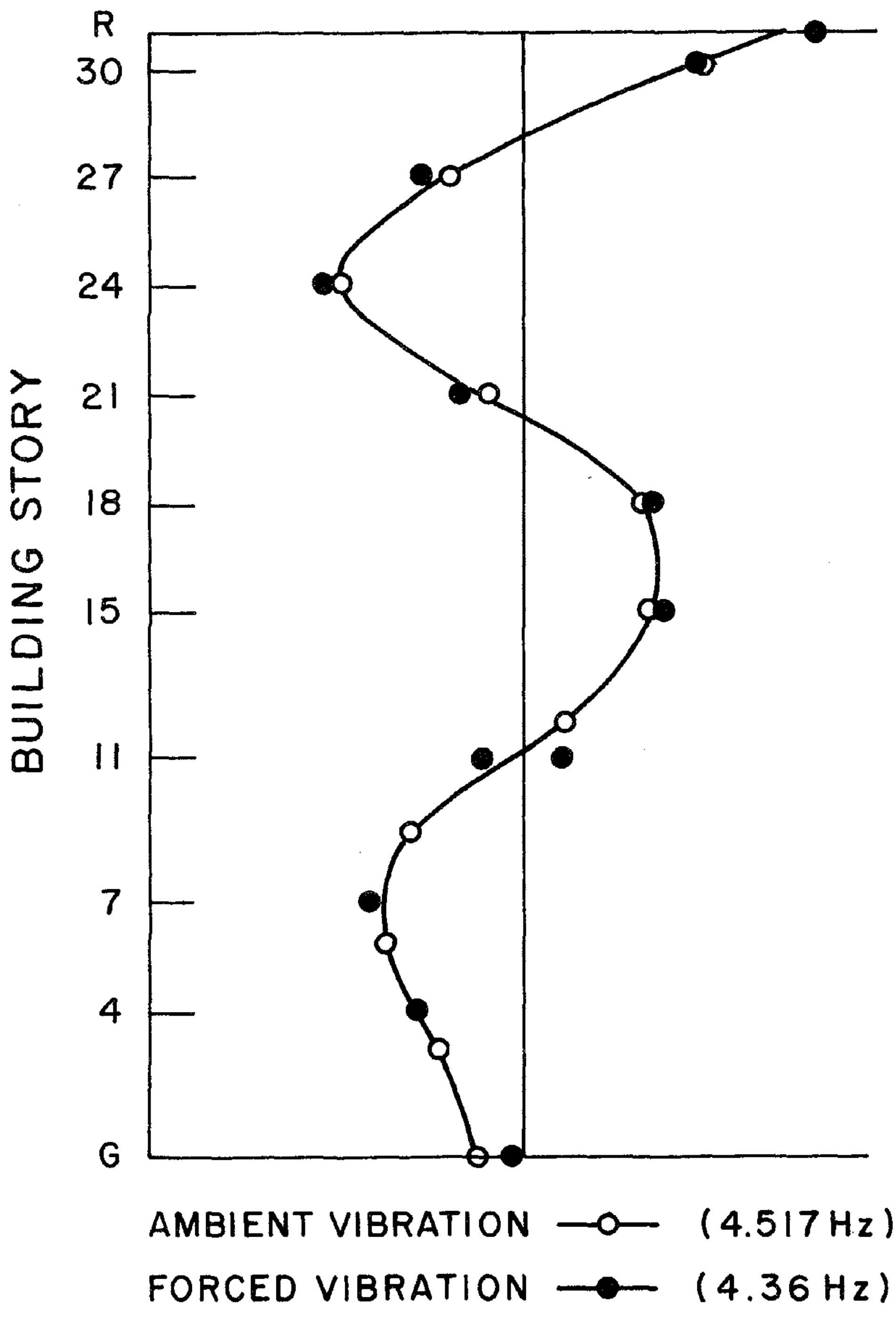


FIG. 4.10 N-S VERTICAL MODE SHAPE

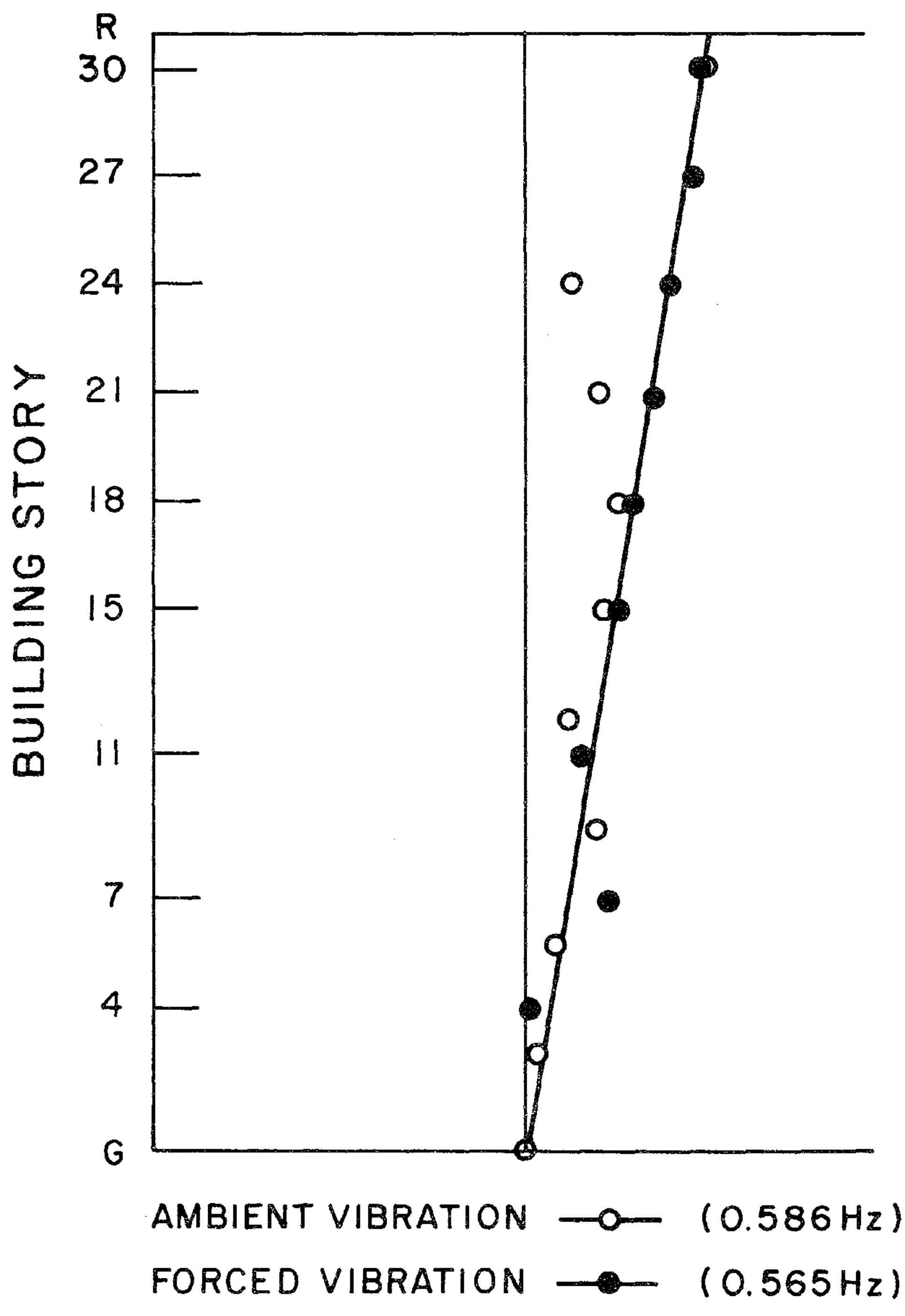


FIG. 4.11 TORSIONAL VERTICAL MODE SHAPE

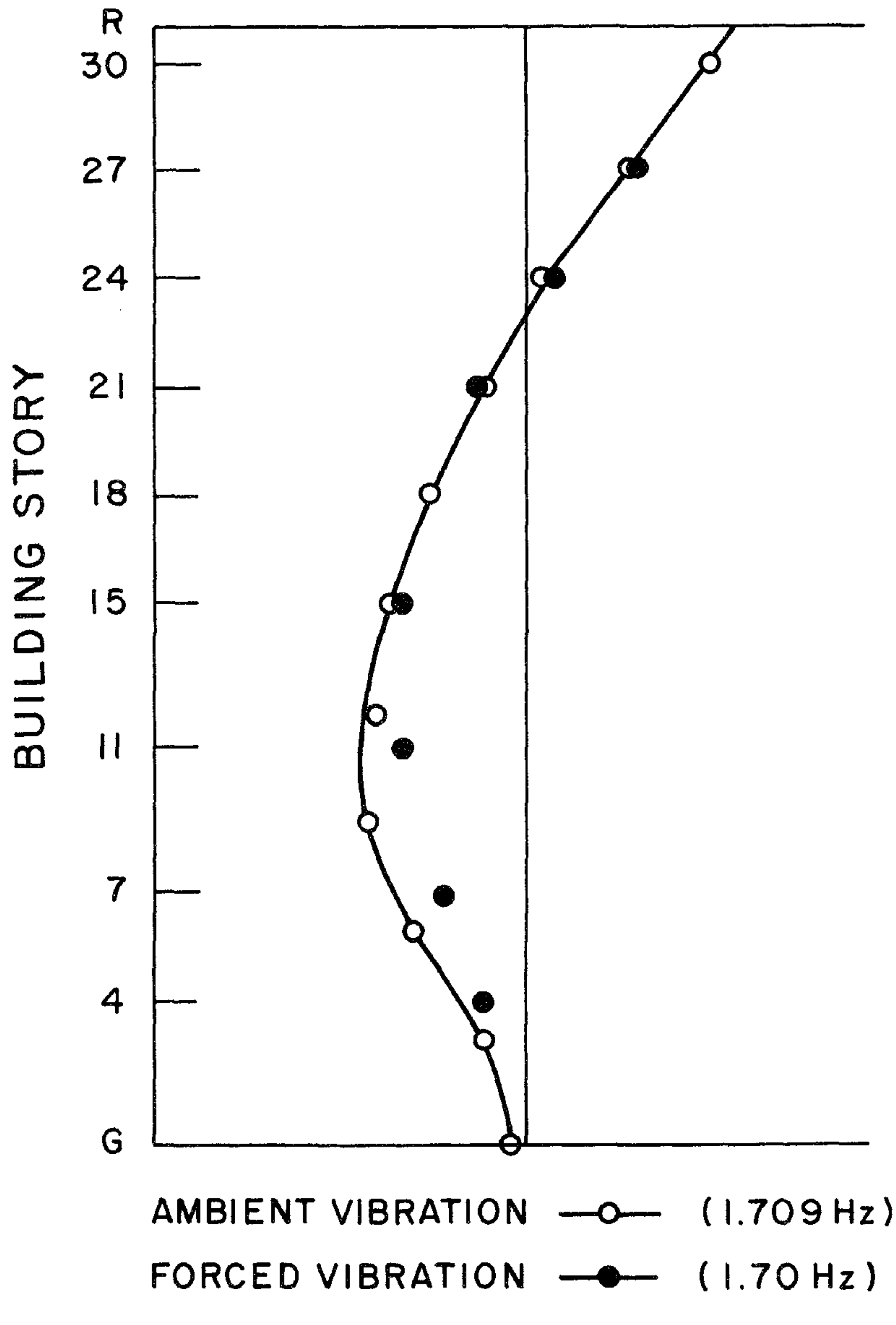


FIG. 4.12 TORSIONAL VERTICAL MODE SHAPE

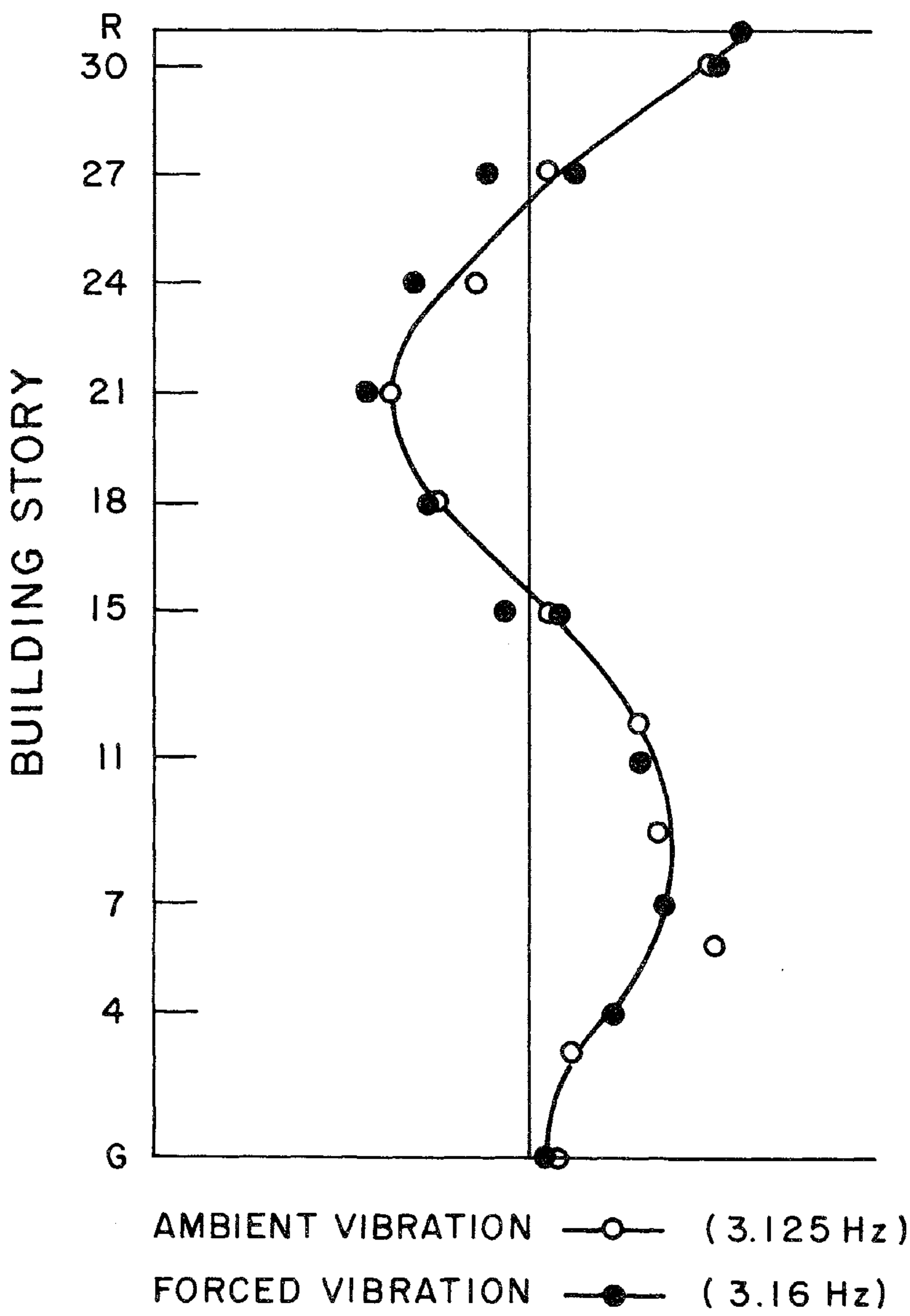


FIG. 4.13 TORSIONAL VERTICAL MODE SHAPE

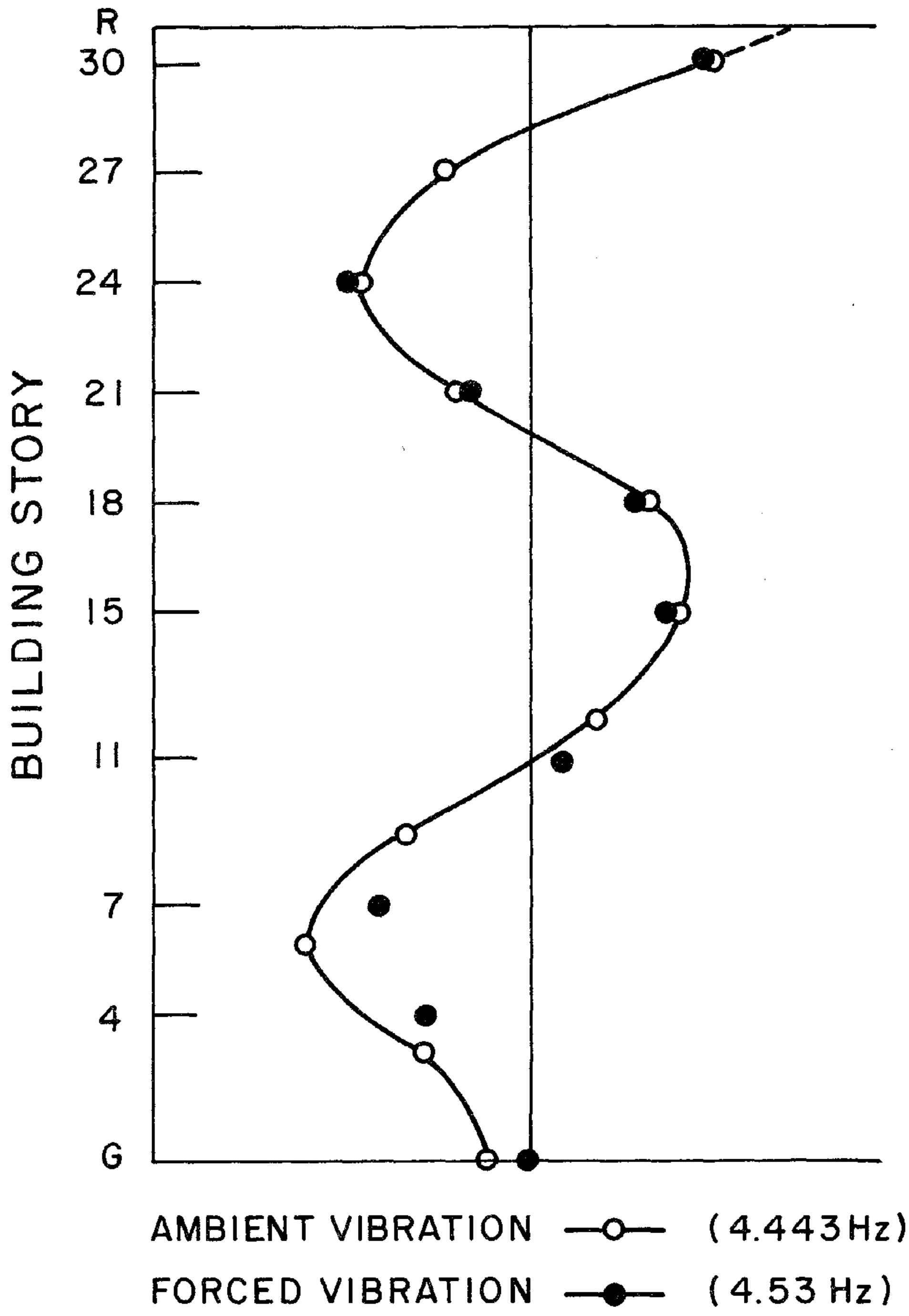


FIG. 4.14 TORSIONAL VERTICAL MODE SHAPE

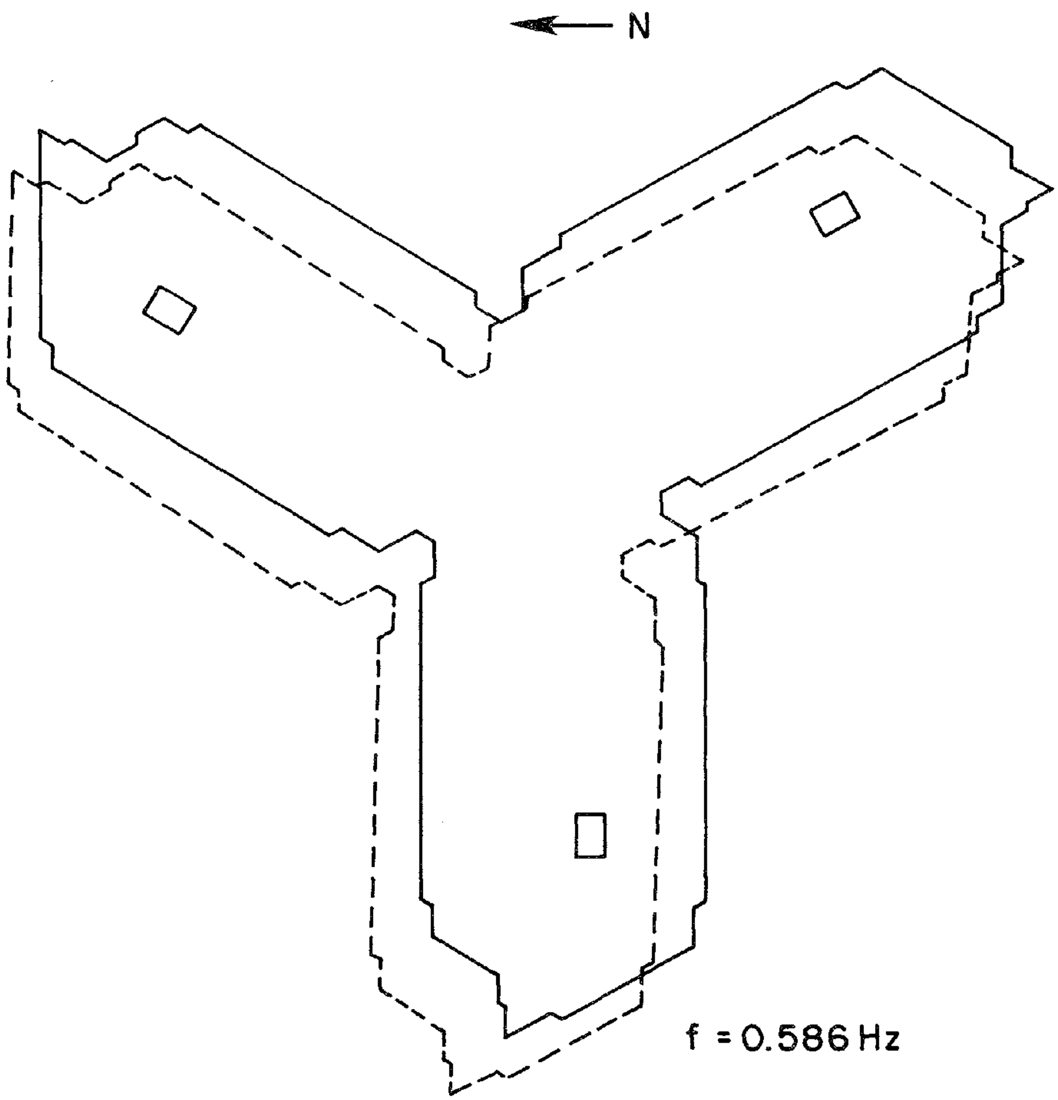


FIG. 4.15 30TH FLOOR MODE SHAPE AMBIENT VIBRATION

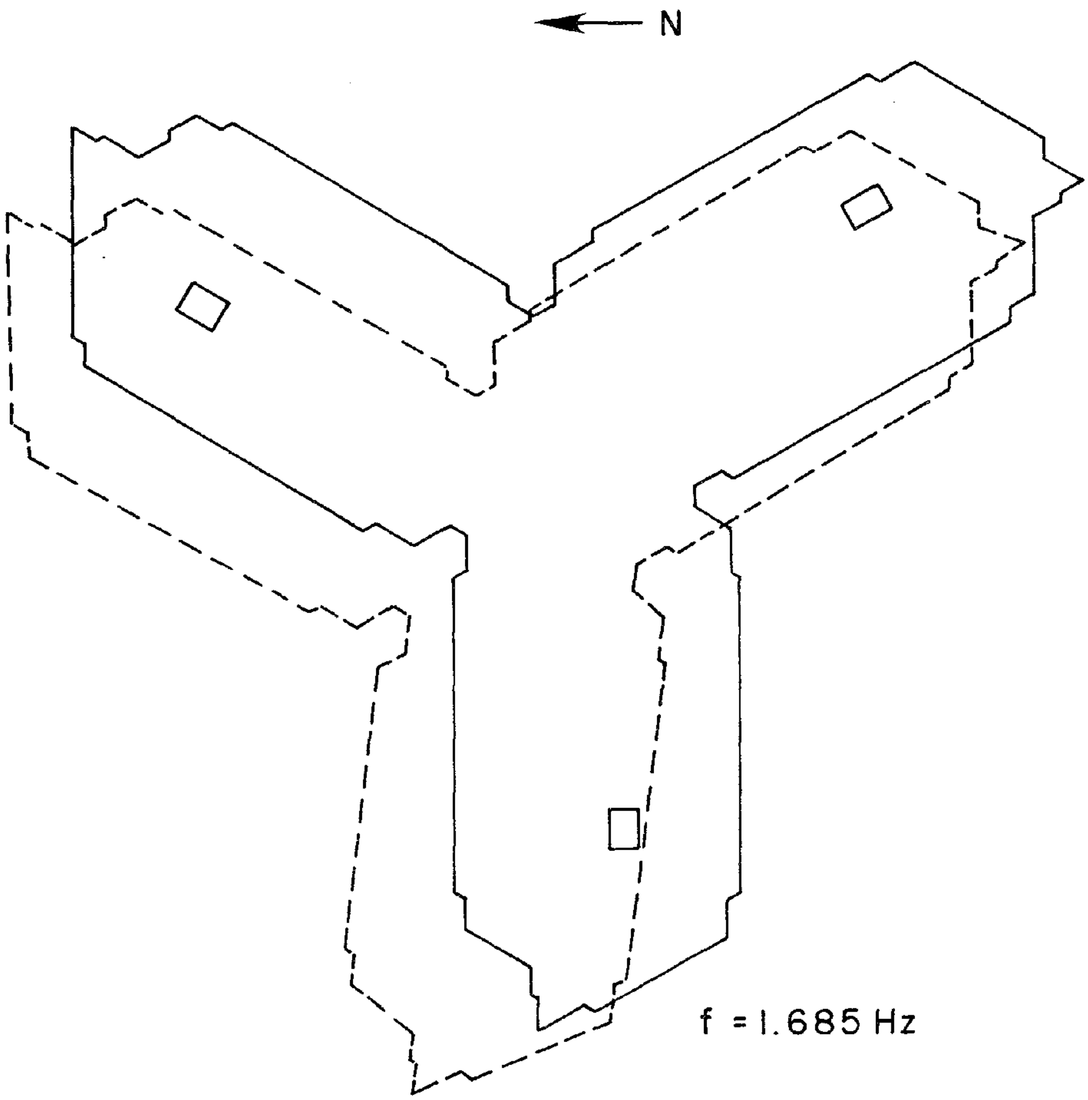


FIG. 4.16 30TH FLOOR MODE SHAPE AMBIENT VIBRATION

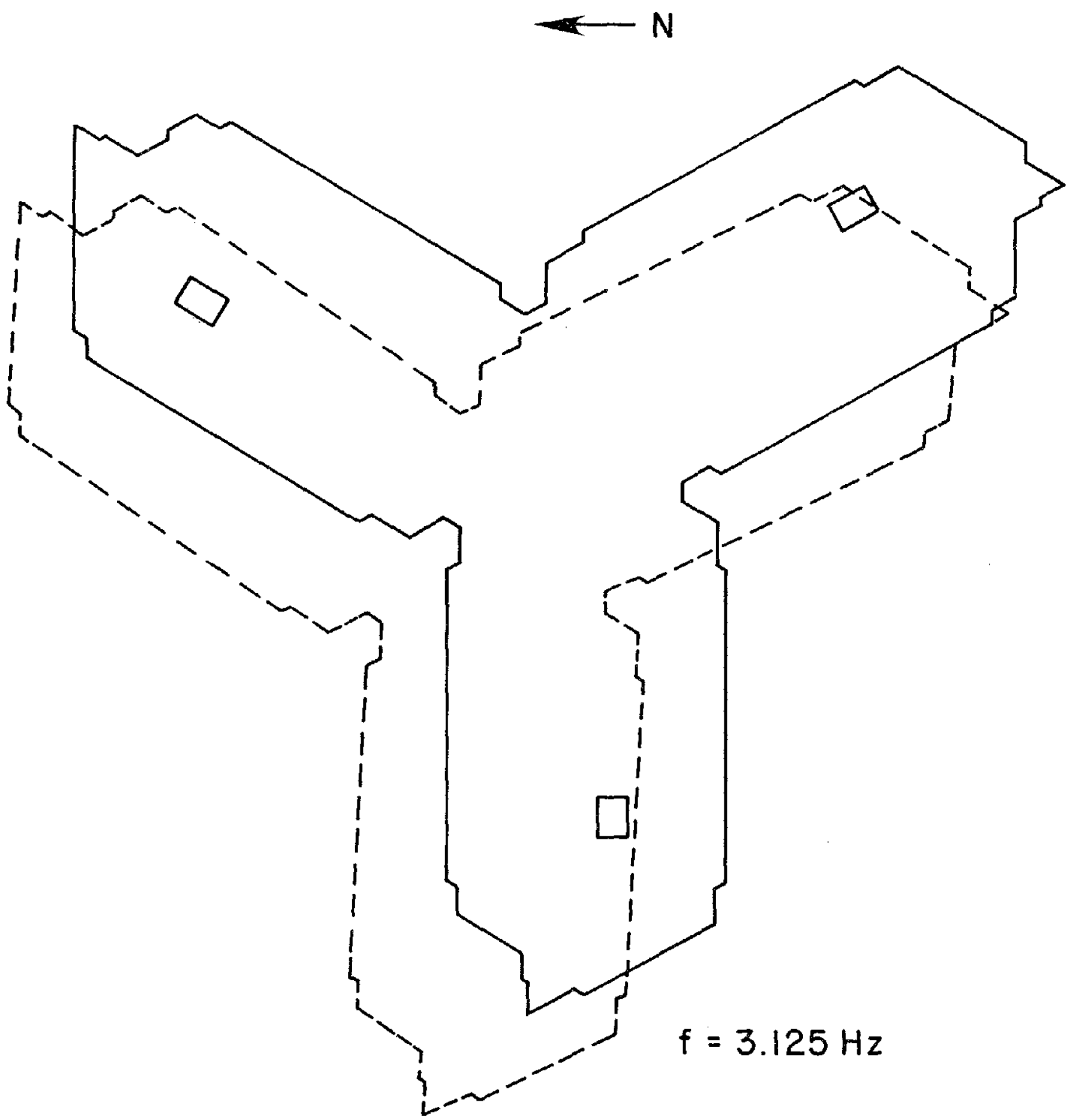


FIG. 4.17 30TH FLOOR MODE SHAPE AMBIENT VIBRATION

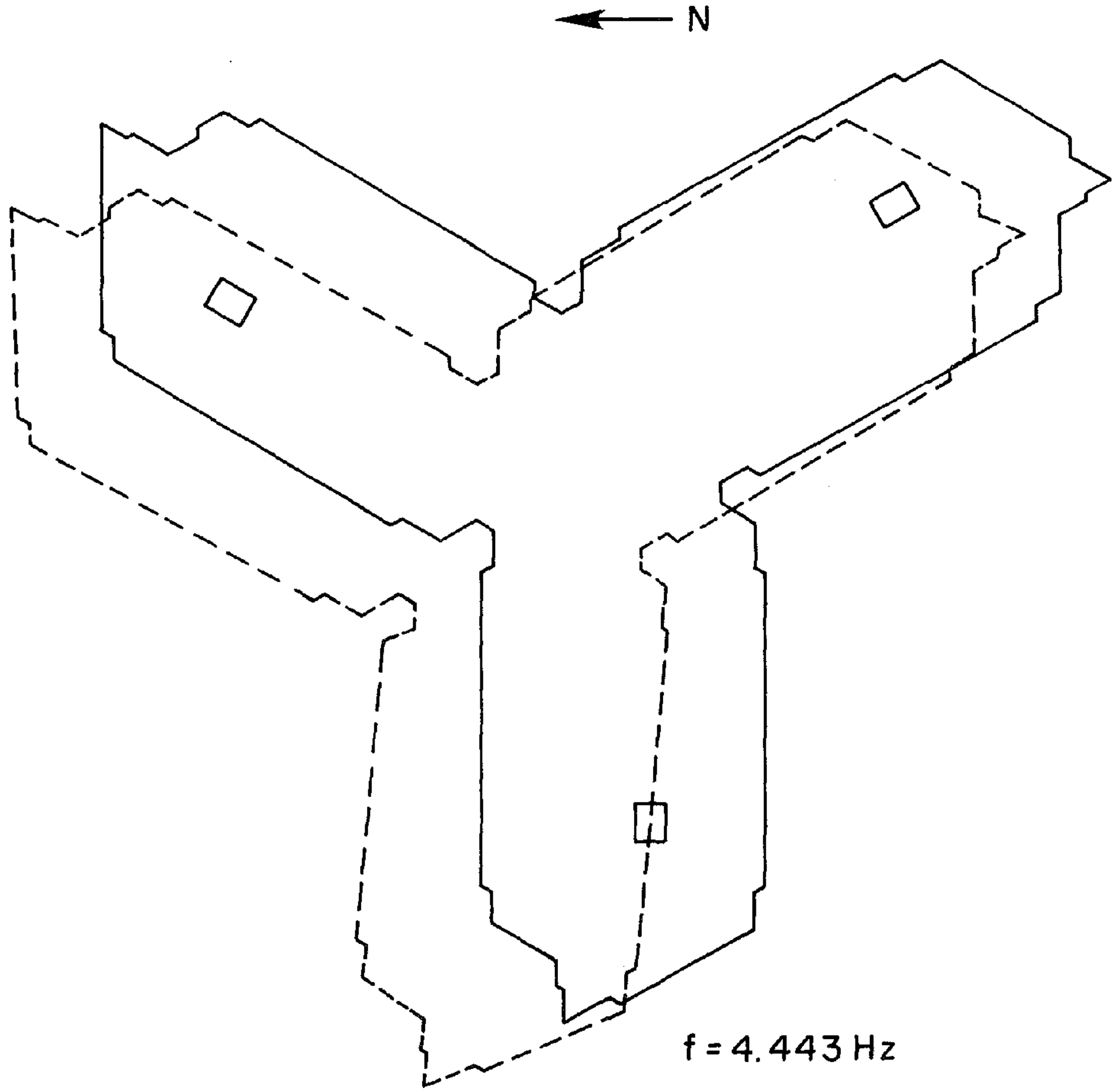


FIG. 4.18 30TH FLOOR MODE SHAPE AMBIENT VIBRATION

5. FORMULATION OF MATHEMATICAL MODEL

5.1 General

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

5.2 Computer Program

A general computer program, TABS, was used to calculate the frequencies and mode shapes of the building. The program operates as a segment within the SAP-80 Series of Programs. The program is described in reference (16).

The analysis in this investigation was performed on a microcomputer using the TABS program. The program considers the floors to be rigid in their own planes and to have zero transverse stiffness. All elements are assembled initially into planar frames. The internal stiffness at such frame is then transformed to the three degrees of freedom (2 translational, 1 rotational) at the center of mass at story level. The mass was then lumped at each second level and the mode shapes and frequencies were then determined.

5.3 Modeling

The basic model of the building was formulated as a system of frames interconnected by floor diaphragms which are rigid in their own planes and fixed at the first floor level. A plan layout of the frames is shown in Fig. 5.1. Because of the irregular configuration of the building, it could not be modeled exactly using plane frames.

An approximation of the layout geometry was necessary and Fig. 5.1 shows how the frames were chosen. The indicated frames were modeled in-plane.

In general the frames are perpendicular to each other and the three dimensional properties of each column are captured by both the intersecting frames. The total number of column lines in all the frames is therefore exactly twice the actual number. (Plane frames only model in-plane properties).

The structural joints where columns and beams intersect were assumed to be rigid and were represented in the computer input as rigid member ends. The rigid ends were measured from the intersecting member center line to the face of that member.

Generally the beam and column center lines in the actual structure do not intersect at a common node. This is caused by the varying member sizes in different floors and bays. The structure was modeled so that the beam center lines of a floor were all in the same plane and the column lines all unstaggered throughout the building.

The columns in the core of the building were restrained axially to avoid discontinuity of vertical deformation where the frames meet. The use of seperate frames without such a measure makes the structure too flexible because the structure does not remain vertically compatible under horizontal loads.

In frames 1 and 3 the end columns (in the core of the building) were omitted because of inclusion in other frames. In these positions continuity was modeled by constraining the in-plane rotations of the two end columns. The vertical displacements at the end column positions were also restrained for the reason given in the previous paragraph.

The west wing base level is lower than the level of the core and other wings. Because the frame assembly was repeated exactly in each wing, a uniform level over the entire base area was adopted. The higher level was used because it covers most of the area. Shear walls are only contained between the first and second floors and were included in the analysis as stiff columns.

The total mass of each floor was uniformly distributed over its area to calculate the radius of gyration (82.0 feet). The structural mass was lumped at each second level and consisted of the slab and beam column masses for each two stories. A concrete weight density of 150 lb/cubic foot was used for the mass computation.

Beams constructed monolithically with slabs were modeled as T-beams. A flange width of 25% x distance between center lines of supporting columns, was used.

The value of the elastic modulus E is taken as 57,000 $\sqrt{f'c}$ (17). This is 580,000 ksf for 5000 psi concrete and 670,000 ksf for 6500 psi concrete. Twelve percent was added to the strength to compensate for the tested average being higher than the specified strength. This increased the respective E values to 615,000 and 710,000 ksf.

The contribution of reinforcing steel to the cross-sectional properties is substantial (+-35% increase in cross-sectional steel). This was not added since the extent of cracking and resulting loss of concrete area is uncertain. Therefore a uniform adjustment of +11% was applied to the elastic moduli calculated in the previous paragraph. This number was arrived at by adjusting the elastic moduli uniformly until the computed fundamental frequency equaled its experimental value.

The foundation was modeled by supporting the foundation mass laterally and torsionally by springs. Actual site properties were not used but the spring stiffnesses were obtained by setting the theoretical mode shape ordinate at the base equal to the experimental ordinate. A rigid base analysis was also done.

5.4 Analytical Results

The analytical results are presented in Table 5.1. Results for a rigid and flexible base are given opposite the experimental results.

TABLE 5.1 MODAL FREQUENCIES (Hz)

No.	(3) fe	Dir. of Forcing	(4) fr (Rigid Foundation)	fe/fr-1 (%)	(5) ff (Flexible Foundation)	(1) fe/ff-1 (%)
1	0.565	T	0.565	0.0 T	0.562	+0.6 T
2	0.590	NS	0.596	-0.5	0.595	-0.3
3	0.595	EW	0.596	-0.2	0.595	0.0
4	1.660	NS	1.666	-0.3	1.650	+0.6
5	1.675	EW	1.666	+0.6	1.650	+1.5
6	1.70	T	1.711	-1.2 T	1.686	+0.2 T
7	3.09	NS	3.115	-0.8	3.081	+0.2
8	3.12	EW	3.115	+0.2	3.081	+1.1
9	3.16	T	3.275	-3.7	3.220	-1.7 T
10	4.36	NS	4.344	0.4	4.292	+1.7
11	4.39	EW	4.344	+1.1	4.292	+2.2
12	4.53	T	4.644	-2.6 T	4.556	-0.7 T
13			5.855		5.744	
14			5.855		5.744	
15	6.19/	EW				
	6.24	NS	6.274	(-0.5)T	6.072	+1.7 T (2)
16			7.205		6.998	
17			7.205		6.998	
18	7.45	NS/EW	7.788	(-4.5)T	7.386	+1.0 T (2)

(1) Kx = 2.5E06 k/ft.
Ky = 2.5E06 k/ft.
Dz = 1.6E10 k-ft.

(2) Matches better with torsional mode although forcing was translational.

(3) Experimental frequencies (hz).

(4) Theoretical frequency for rigid foundation. (fr) T : Torsional.

(5) Theoretical frequency for flexible foundation. (ff) T : Torsional.

Normalization of the elastic modulus with respect to the fundamental mode yields good consistency of modal frequencies. This may be interpreted as a sign that there are no significant modeling deficiencies. In the rigid base case there is, however, a small inconsistency in the torsional frequencies. The error increases from (the selected) zero in the first mode to -3.7 and -2.6% in the 3rd and 4th torsional modes, respectively.

In the flexible foundation case, with springs provided as explained previously, the consistency is improved upon so that an error of -1.7% is found for the 3rd torsional mode.

The 15th and 18th modes have experimental values which are very close to the theoretical torsional frequencies although forcing was applied translationally. In the table the values are compared as such and the errors shown in brackets.

The modal frequency values in the table are arranged in order of increasing magnitude.

The mode shapes representing the flexible foundation case are shown in Figs. 5.2 and 5.3. The experimental mode shapes were compared to both the free vibration theoretical mode shapes and to the forced vibration shapes. A forced shape in this context is the computed structural shape of the building when forced on the 30th floor at a resonant frequency. Because this loading has components in each mode, the forced shapes differ slightly from the free mode shapes. Experimental forced shape discrepancies in higher modes probably result from significant in-plane floor deformations at high frequencies. (See 30th floor motion at 6.19 Hz.)

The overall consistency of modal frequencies, when foundation motion and fundamental frequency are used as norms to adjust the elastic properties uniformly, proves to be excellent through the 12th mode.

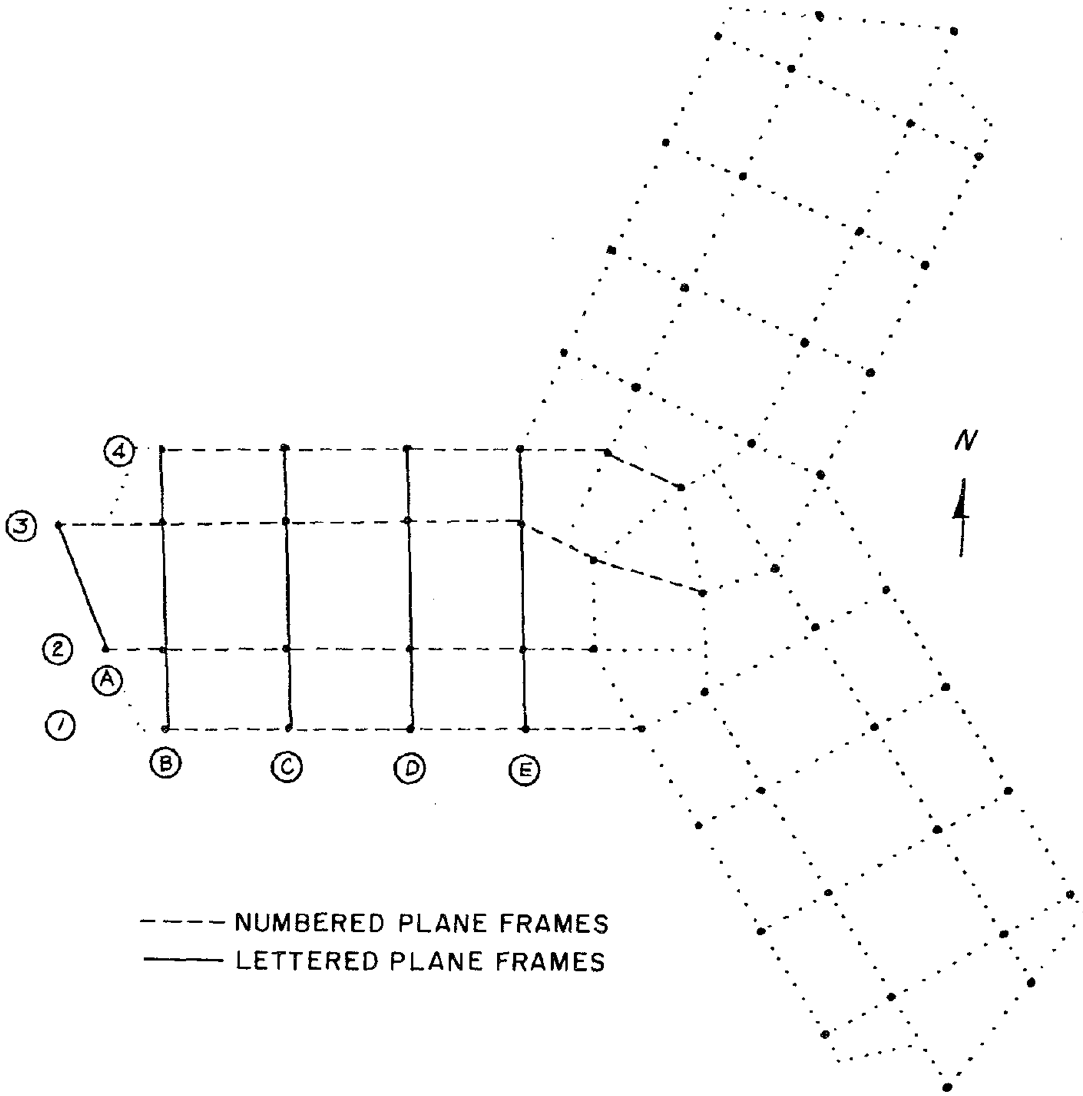


FIG. 5.1 PLAN LAYOUT OF THE FRAMES

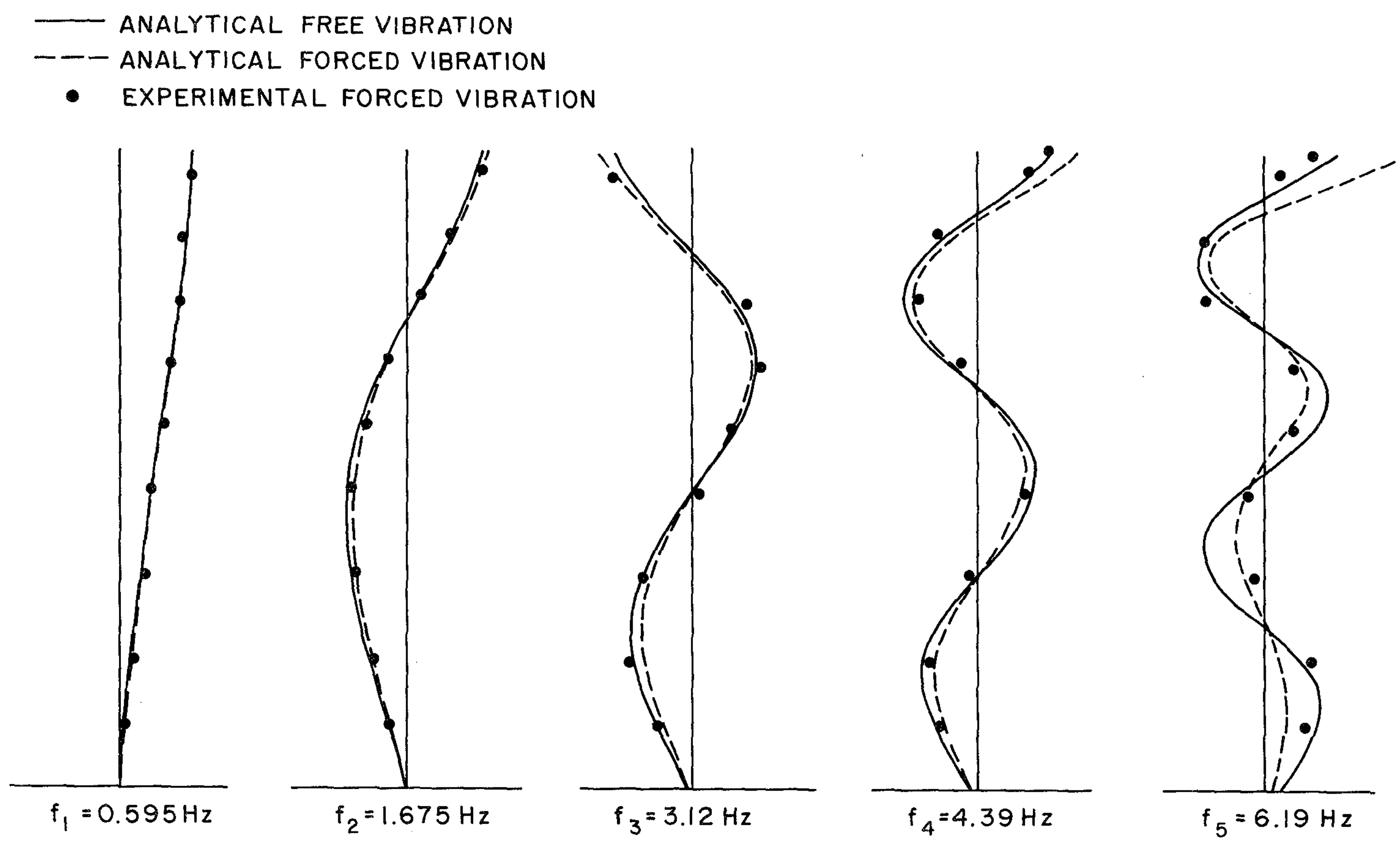


FIG. 5.2 MODE SHAPES, TRANSLATION E-W

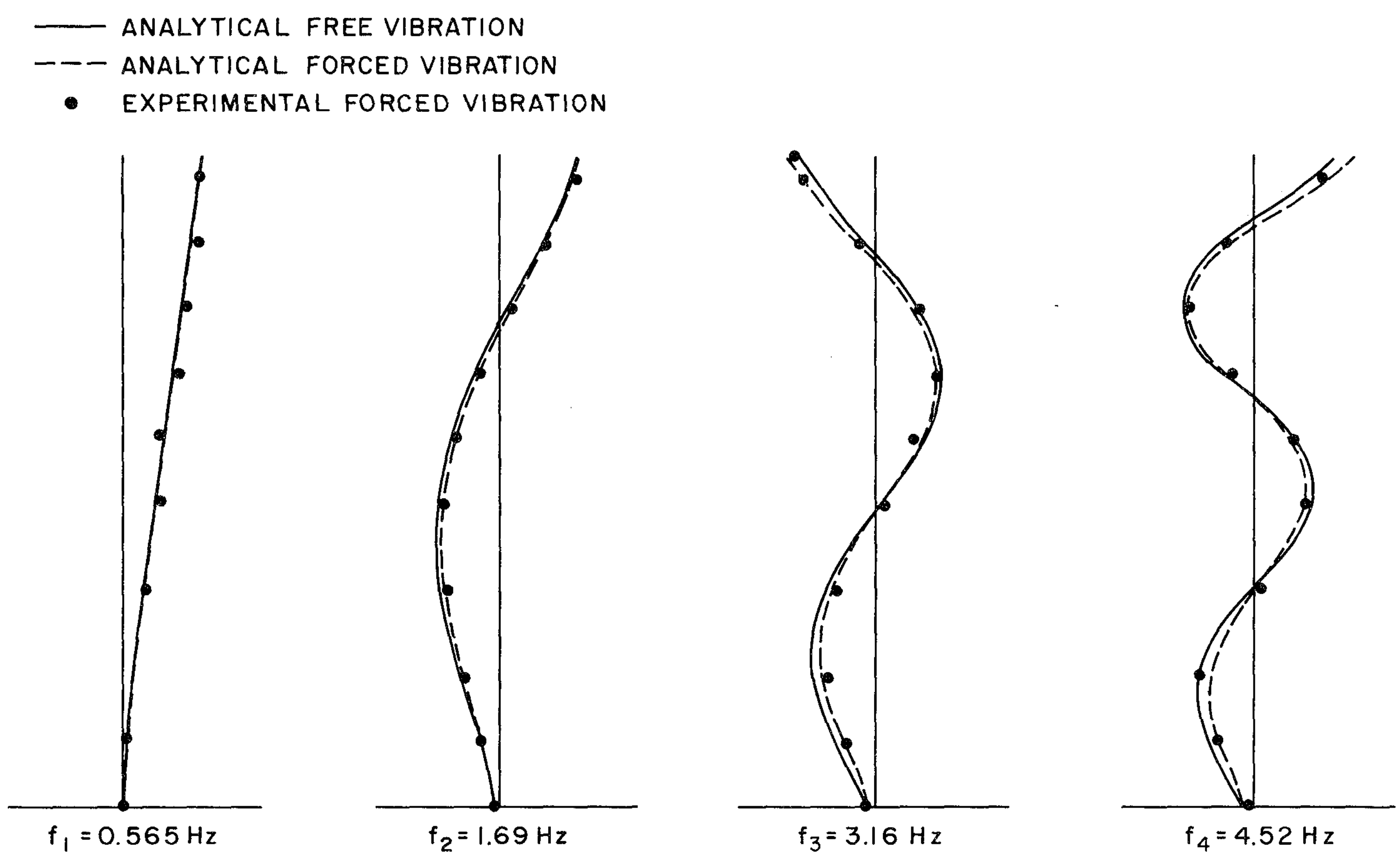


FIG. 5.3 MODE SHAPES, TORSIONAL

6. 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 7.5 Hz, sixteen forced vibration modal frequencies were identified. The resonant frequencies for the E-W, N-S translational and torsional direction for each particular mode were very similar. This clearly indicates the uniform stiffness of the structure. The ratios of the observed higher mode frequencies with respect to the fundamental one from the forced vibration study indicate that the overall structural response is predominantly of the shear type. In the higher modes significant in-plane floor deformations were noted.

The analytical study was carried out using a fixed as well as a flexible foundation. There was very little difference noted between these two studies. The flexible foundation model did show better agreement with the forced vibration study, however, and this model was used for mode shape comparisons. A comparison of the analytical results with the experimental results show very good agreement even into the higher modes of vibration.

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Rigid floor diaphragm action was noted at higher frequencies and very little structure foundation interaction was observed.

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