VIBRATION TESTS OF A MULTISTORY BUILDING

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ABSTRACT

Vibration tests were performed on a 9 story reinforced concrete building with basement, in order to investigate its dynamical characteristics, by exciting the building with 2 vibration generators installed on its 9^{th} floor.

The natural periods of vibration, the value of the damping, and the mode shapes, in the N-S and E-W directions and in torsion, were determined by measurement. Before the main part of the testing was carried out, some preliminary tests were made to check the correctness of some assumptions which would simplify the main test procedure.

It was possible to investigate in detail only the first mode of each type of motion, because of the relatively high rigidity of the building and a limitation on the maximum frequency at which the shakers could be driven.

The periods measured were quite short for a 9 story building, 0.505 sec in the N-S direction, 0.662 sec in the E-W direction, and 0.346 sec in torsion, and their values increased by about 3 per cent when the tests were performed at the highest force levels.

The damping, which consistently increased as the exciting force increased, varied between 0.70 and 2.00 per cent of the critical viscous damping. The periods and damping values were also determined at very low force levels by exciting the building with a rhythmical movement of the operator's body. The periods measured in this way were slightly smaller than those found using the shakers, and the damping varied between 0.6 and 0.9 per cent of the critical

viscous damping.

The mode shape did not seem to be well defined for the lower force levels, but after the force level reached a certain minimum value, the normalized mode shape remained unchanged, both with further increases in the forces, and with changes in the frequency of excitation. However, in both the N-S direction and in torsion, the horizontal displacements of the first and basement floors consistently increased, on the order of 3 per cent with respect to the displacements of the upper floors as the exciting force increased.

Some aspects of the dynamical behavior of buildings, which have not been studied by other investigators in previous tests, were examined. It is a common practice in the seismic analysis of structures to assume that the floor systems act as rigid diaphragms when the building is acted upon by horizontal forces, and also to assume that the structure is fixed at the ground level. It was found that the first assumption was correct, but instead of the second it is more accurate to assume that the building is fixed at the foundation, and not at ground level.

The vibration of the ground in the vicinity of the building was also measured, together with the vibrations of the basement and first floor. It was also possible to measure the acceleration at the top of one of the units of the air conditioning equipment located on the roof. The acceleration at the top of this unit was about 8.5 times that of the roof.

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I. INTRODUCTION

At the present time, full scale vibration tests are one of the most effective methods for investigating the dynamical characteristics of a structure. In a reduced scale model study, it is very difficult to reproduce the structural members, joint conditions, and the foundation-ground interaction, and in any theoretical study, the idealization implies simplifying assumptions that could lead to significant errors. Further, the only method for determining the damping is by measuring it directly from the structure.

On the other hand, if satisfactory mathematical models have been developed, and the study is made with the aid of a digital or analog computer, it is quite easy to change the values of the parameters involved, so that a general study may be made in a relatively short time. Moreover a full scale vibration test only applies to a particular structure, so it is possible to arrive at general conclusions only if there is sufficient data available from vibration tests performed on a representative range of structures.

Since 1961, when a new vibration generator(1)* was developed at Caltech, researchers in the U.S.A. have used this improved exciter to test actual structures of different types: a tower,(2) dams,(3,4) an atomic reactor,(5) buildings(6,7,8) and other structures.(9,10)

In Japan, an extensive program of full scale vibration tests

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*Superscript numbers in parentheses indicate references at the end of this report.

has been carried out.(11,12,13,14) Relatively low buildings with high rigidity, resting in most cases upon soft ground, have been tested, so it is not surprising that important horizontal displacements of the buildings due to swaying and rocking were measured. It was estimated that a significant part of the energy input from the vibrator was radiated into the ground through the foundations. The results of experiments made in the U.S.A. show that for typical conditions on the West Coast, the contrary is true. The results reported here confirm this, in spite of the fact that the rigidity of the Millikan Library Building in the N-S direction is more than twice as great as the rigidity of a typical building in California.

For the reasons noted above, the results of the Japanese building tests can not be used directly in the study of the dynamical characteristics of buildings located on the West Coast of the U.S.A.

The purposes of the present tests are twofold: first, to study the dynamical characteristics of the Millikan Library Building, (which is somewhat different from other buildings which have previously been tested using the same vibration generators), so that data on the dynamical properties of buildings of this type will be available for immediate application or further study (i.e. strong motion accelerographs are going to be installed in the building, one on the roof and another in the basement. See Fig. AV-15); and second, to investigate other aspects of the structural dynamics problem not covered in prior tests. These aspects have been briefly indicated in the Abstract.

1.1 The Building

The Robert Millikan Memorial Library Building, located at the campus of the California Institute of Technology, was built during 1966-67. It is a 9 story reinforced concrete building, with an inter-story height of 16' between the first two floors and 14' between the eight upper floors. The roof is surrounded by a wall 16' high, and the total height from ground level to the top of the building is 144'. It has a basement, with exterior walls of reinforced concrete 12" thick, and floor 14' below the ground level.

In plan the building is a 75' × 69' rectangle, with the longer dimension oriented E-W (see Fig. 1.1). There is an additional 8' × 23' area at the East side, and a 14' × 29' staircase at the West side. Both of these end in curved walls, and both are symmetric about the E-W centerline. Thus, the entire building is symmetric about the E-W centerline, but not about a N-S centerline. At the center of the building, there is a box, 23'6" × 26'8" in plan, containing the elevator and a service stairway and there are partition walls in the basement and between the second and third floors. The floor systems consist of 9" thick slabs of lightweight concrete, reinforced in two directions and supported by 36" × 24" beams.

The building is designed to resist most of the horizontal seismic forces in the N-S direction with the East and West shear walls, and most of the E-W forces with the elevator box. All these resisting elements have uniform thickness of 12" from top to bottom, but due to an increase in the percentage of reinforcing steel toward

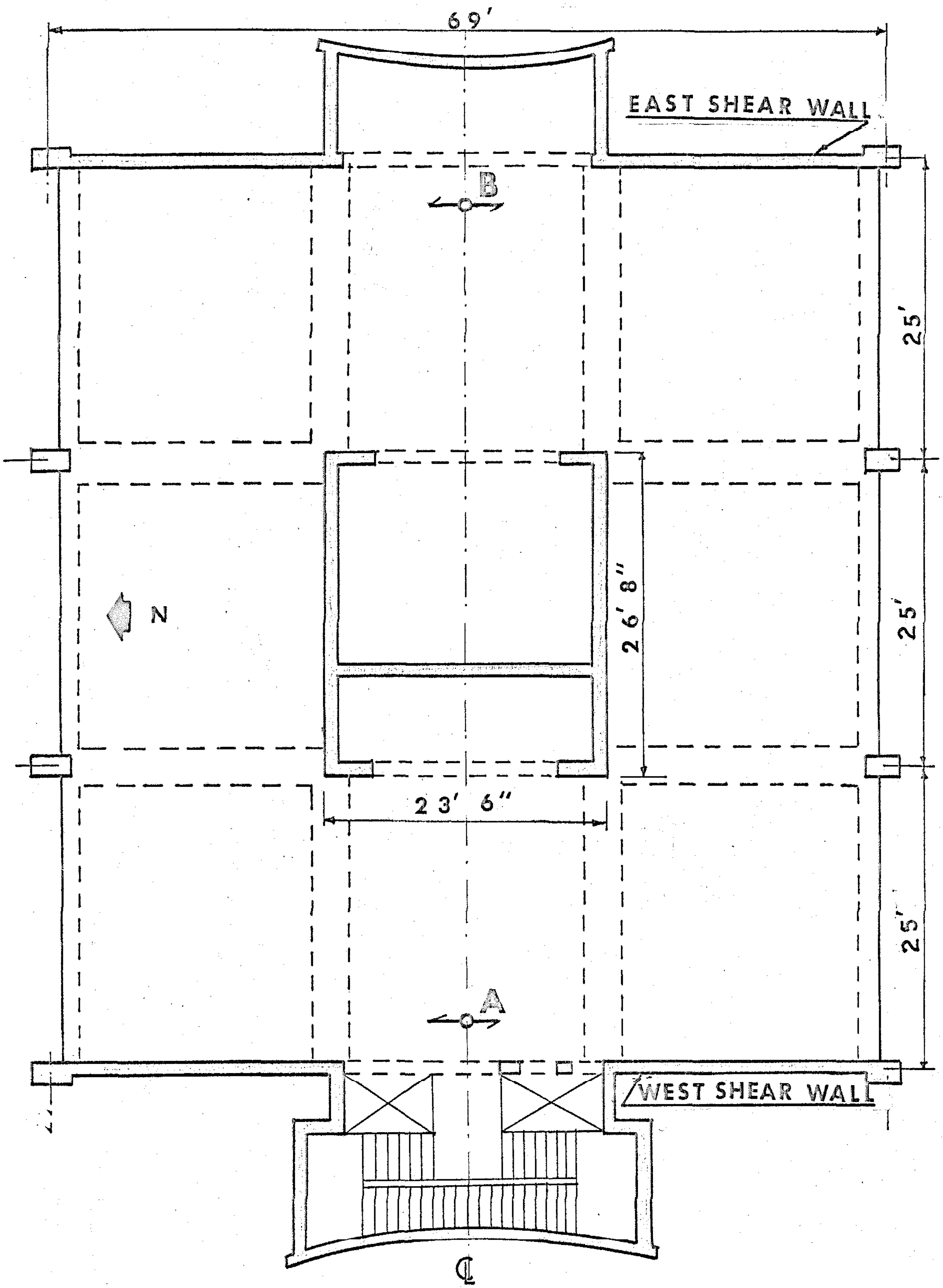


Fig. 1.1 TYPICAL FLOOR PLAN SHOWING LOCATION OF THE VIBRATION GENERATORS

the bottom, the lower portions have moments of inertia approximately 6 per cent higher than the upper portions.

The North and South side facades consist of precast concrete window wall panels weighing 11 tons each, which were lifted to their locations with a crane (see Fig. AV-7,8). Steel plates cast in the wall panels were bolted to the columns and beams to connect the panels to the structure.

The second floor beams running East-West are of variable section, in order to harmonize with the appearance of the arcs of the surrounding buildings (see Fig. AV-9).

The foundations consist of a central pad 32' wide and 4' deep which extends from the East curved wall to the West curved wall, and 2 foundation beams (9' × 4') located to the North and South of this pad and parallel to it (see Fig. 1.2). The beams end at the East and West shear walls. Beneath both the East and West shear walls is a beam (10' × 2') running N-S which connects the foundation beams to the foundation pad. These connecting beams are stepped, because the bottom of the central foundation pad is 5'4" below the bottom of the North and South foundation beams.

1.2 Condition of the Building During the Tests

At the time the tests were started the main structure had been completed for about 2 months and the partition walls of the basement and second floor were finished, but the North and South side facades were not in place. Period measurements with the lunar seismometer were made with the structure in this condition.

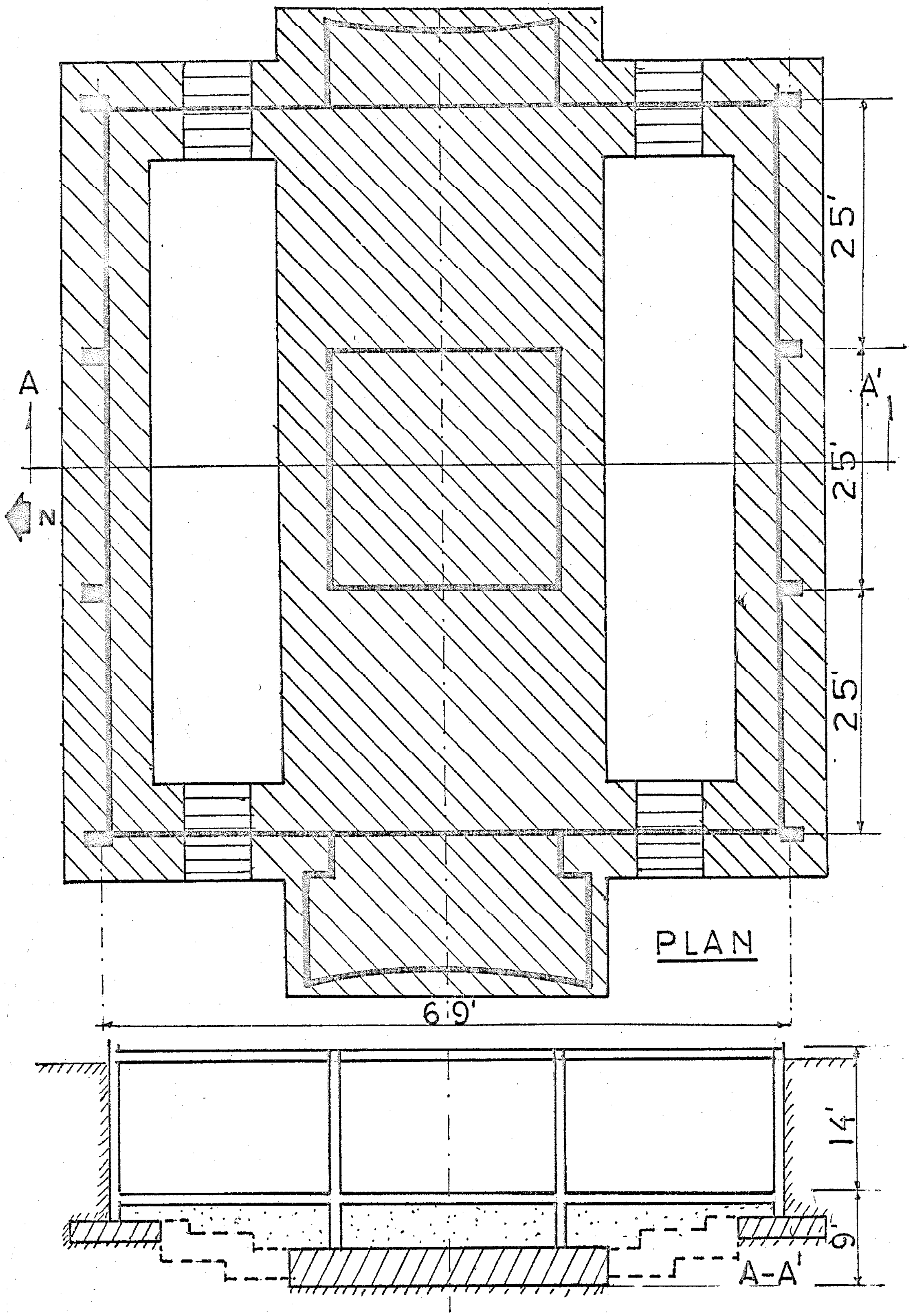


Fig. 1.2 FOUNDATION PLAN AND A N-S SECTION

When the first part of the preliminary steady state tests were made only the North side window panels had been installed. However, the rest of the tests were carried out after the window panels on both sides had been placed, and the finishing work was being done.

The mass of the building, assumed lumped at each floor level, was estimated from the construction drawings, and the weight of the air conditioning and elevator equipment already placed on the roof was taken from the manufacturer's catalog. The masses, in kips, at the time that the steady state tests were performed, were from top to bottom: $M_{10} = 2600$, $M_9 - M_3 = 1950$, $M_2 = 2433$ and $M_1 = 2280$.

1.3 The Tests

The vibration tests performed consist basically of a determination of the steady state response of the building to a given sinusoidal exciting force. Appendix I contains a listing and a brief description of all the tests that were made.

The testing method used may be summarized as follows:

1) The vibration generators were installed on the 9^{th} floor of the building at the locations A and B shown in Fig. 1.1. The exciting forces could easily be made to act in either the N-S or the E-W direction.

2) The recording system was calibrated at the beginning of each working day.

3) The accelerometers were installed, properly oriented, at previously selected locations.

4) After the test force level had been decided, the appropriate

weight combination was placed in the baskets of the vibration generators (see Table AI-2 of Appendix I for the weight combination selected for each test).

5) The exciter was driven at the resonant frequency, when the amplitude became a maximum. The amplifiers were then adjusted to obtain the best use of the recording paper.

6) The frequency of excitation was decreased to an amplitude of about 1/10 to 1/15 of that at resonance, and the reading of the frequencies and recording of the vibrations was begun.

7) The frequency of excitation was increased in appropriate steps. The steady state response was reached after several cycles of excitation, with the waiting time becoming longer for the higher force levels. The entire operation was controlled by an auxiliary accelerometer and recording system which gave a continuous reading of the ninth floor response.

8) After the resonant peak was passed and the response curve became horizontal, the frequency was decreased, and the reading was stopped when the frequency reached the starting frequency. The vibration generators were then stopped.

9) The system was made ready for the start of a new test by changing the weight in the baskets of the exciter, and the locations of the accelerometers, if necessary.

10) After the last test of the day was finished, the recording system was recalibrated to check that it had performed properly throughout the day.

1.4 Instrumentation

The building was excited with two vibration generators[1] installed on the ninth floor at locations A and B of Fig. 1.1. Each unit can generate a unidirectional sinusoidally varying inertial force which can reach up to 5000 lbs, and can be driven up to a maximum speed of 10 c.p.s. The vibration generators can be operated independently or synchronously. The response of the building and nearby ground was recorded with a six channel accelerometer-amplifier-recorder system. More details of the organization,and description of the instruments involved in the tests are given in Appendix II.

1.5 Analysis of the Data

The methods used for the computation of the results are described here. Additional explanations are included with the results of specific tests where necessary.

The natural periods of vibration were computed from the peak frequencies of the acceleration response curves. Theoretical analysis has shown[14] that the resonant frequency, n_d, found from a displacement response curve, is related to the resonant frequency, n_a, found from the corresponding acceleration response, by the formula

$$n_d = n_a(1 - 2\xi^2) \tag{1.1}$$

where ξ = fraction of critical viscous damping.

For the Millikan building, ξ was found to be less than 0.02,

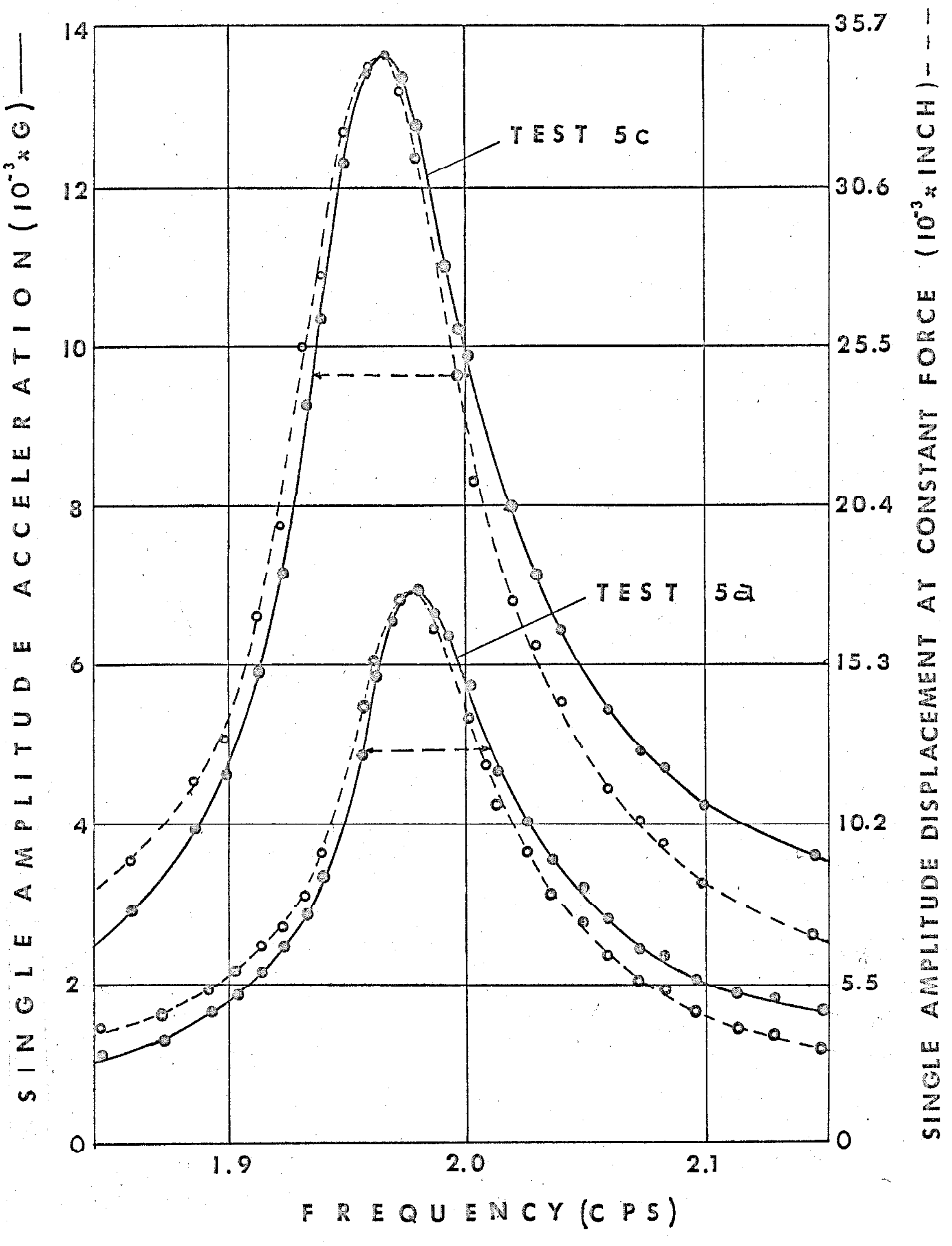


Fig. 1.3 ACCELERATION RESPONSE CURVES AND DISPLACEMENT RESPONSE CURVES FOR CONSTANT EXCITING FORCE

so the difference between the two peak frequencies n_d and n_a would not exceed 0.1 per cent, and either value could be used.

The damping was estimated by the methods described below. The derivations of the formulas are not included, but references to them are given.

1) Standard or half power method

This method consists of measuring the width, Δn, of the displacement response curve at an amplitude of $\sqrt{2}/2$ times the resonant amplitude. The resonant frequency (n_d) is found from a steady state test. For a single degree of freedom system excited with constant amplitude sinusoidal force, the damping value is then given by the well known relation:

$$\xi = \frac{\Delta n}{2 n_d} \qquad (1.2)$$

The Millikan building was excited with sinusoidal forces that varied as the square of the frequency of excitation, and accelerations, not displacements, were the quantities measured. Thus this method could not be applied directly. The acceleration response curves of tests 5a and 5c were reduced to displacement response curves by assuming that for each test the force at resonance was kept constant throughout the test. The acceleration and displacement response curves are shown in Fig. 1.3 with the vertical scale chosen so that the peaks coincide. It can be seen in the figure that the basic effect of the reduction is that the curves are rotated clockwise about the peaks, and neither the width of the response curves at $\sqrt{2}/2$ times the maximum amplitude, nor the resonant frequencies, change by a measurable amount. Thus the damping may be computed directly from the acceleration response curve, with no further data reduction.

2) Using the mode shape

It is shown in part II that the first nodes in the N-S and in the E-W directions were almost pure translational modes, and that the first torsional mode was well separated from these. Because this was so, the damping for the first modes could be computed by using the relation[6]

$$\xi = \frac{\psi_k^{(1)} - \psi_m^{(1)} F_m}{2\,|\ddot{X}_k|} \tag{1.3}$$

where

$\psi_k^{(1)}$ = normalized deflection at "k^{th}" floor in the first mode

$\psi_m^{(1)}$ = normalized deflection at the "m^{th}" floor in the first mode

F_m = force applied at the "m^{th}" floor

$\ddot{X}_k$ = acceleration at the "k^{th}" floor

and the mode shape is normalized by using the formula

$$\sum_{j=1}^{n} m_j(\psi_j^!)^2 = 1 \tag{1.4}$$

where

m_j = mass at level j

3) Hudson's method

Since the damping of the structure is small, and the first modes are well separated, the assumptions made in arriving at the following expression[6] for ξ are valid.

$$\xi = \frac{1}{8} \frac{A_2}{A_1} \tag{1.5}$$

where

A_1 = peak amplitude of the acceleration response curve

A_2 = the value of acceleration at which the response curve is horizontal

The value A_2 was determined during testing, but generally is not within the range of data plotted in the figures.

4) Free vibration decay test

When a vibration generator is not available, a damping value can be found by measuring the amplitude of two successive oscillations (X_n, X_{n+1}) of a free vibration test. The formula giving the damping[16] is

$$\log\left(\frac{X_n}{X_{n+1}}\right) = \frac{2\pi\xi}{\sqrt{1 - \xi^2}} \tag{1.6}$$

This method was used in part VII of this report to compute damping values, and the results are compared with the values found by the other methods.

The values of the displacement amplitudes at resonance were calculated from the values of the acceleration using the relation

$$D = \frac{386\,\alpha G}{(2\pi n)^2} \tag{1.7}$$

where

D = displacement in inches

αG = acceleration as a fraction of gravity

n = frequency of excitation in c.p.s.

The <u>mode shape</u> was normalized with respect to the value of the component having the maximum amplitude.

II. PRELIMINARY TESTS

A. Identification of Resonant Frequencies by Steady State Test

With no load in the baskets of the shaking machines, it is possible to drive the vibration generators up to their maximum speed, producing a relatively small force as a result of the eccentricity of the baskets themselves. (The counter-balances were removed for this test.) In this way, the frequencies of all the modes within the range of the exciter, which can be driven up to about 10 c.p.s., can be identified. Knowledge of all the frequencies to be encountered in the tests permits careful planning of the complete experiment, with consequent advantages.

The exploratory tests were made in the N-S and E-W directions and in torsion, using only one exciter at a time. With the exciter located at the East side of the 9^{th} floor (location B, Fig. 1.1) and the baskets oriented so that the exciting forces were acting in the N-S direction, two resonant peaks were found. The first at 2.04 c.p.s. or $T = 0.491$ sec (first mode, N-S) and the second at 2.96 c.p.s. or $T = 0.338$ sec (first torsional mode).

For the identification of the frequencies in the E-W direction, the exciter on the East side was used with the exciting force oriented in that direction. Note in Fig. 1.1 that the resulting exciting forces act along the E-W center line of the building.

The first mode in this direction was found at 1.49 c.p.s. ($T = 0.671$ sec) and the second mode at 6.45 c.p.s. ($T = 0.155$ sec). At the time these tests were made the North side concrete facade was already placed. The influence of the precast concrete window panels on the dynamical properties of the building is discussed in part VII, section A.

B. Check of the Assumption That Each Floor Was Moving as a Rigid Body

For dynamic tests of large structures, such as a multistory building, the vibration pick-ups have to be placed at a number of selected points, limited by practical considerations; and it is necessary to assume that the vibrations measured at each point are the same as those occurring in adjacent portions of the building.

Specifically, in the Millikan Library Building test, one accelerometer was placed on every other floor, and it was assumed that the acceleration at the point where the accelerometer was placed would be the same as that of the entire floor. In order to check the validity of this assumption six accelerometers were placed on the 9th floor, located as shown in Fig. 2.1, five measuring vibration in the N-S direction, and one in the E-W direction. The two vibration generators were synchronized to produce equal forces acting in phase in the N-S direction.

An additional reason for performing this test was to verify the usual assumption made during a building seismic analysis, that every floor system acts as a rigid diaphragm, distributing the shear forces to the vertical elements in proportion to their rigidities. The East and West shear walls of the Millikan building are very rigid compared to the other structural elements, so it is interesting to

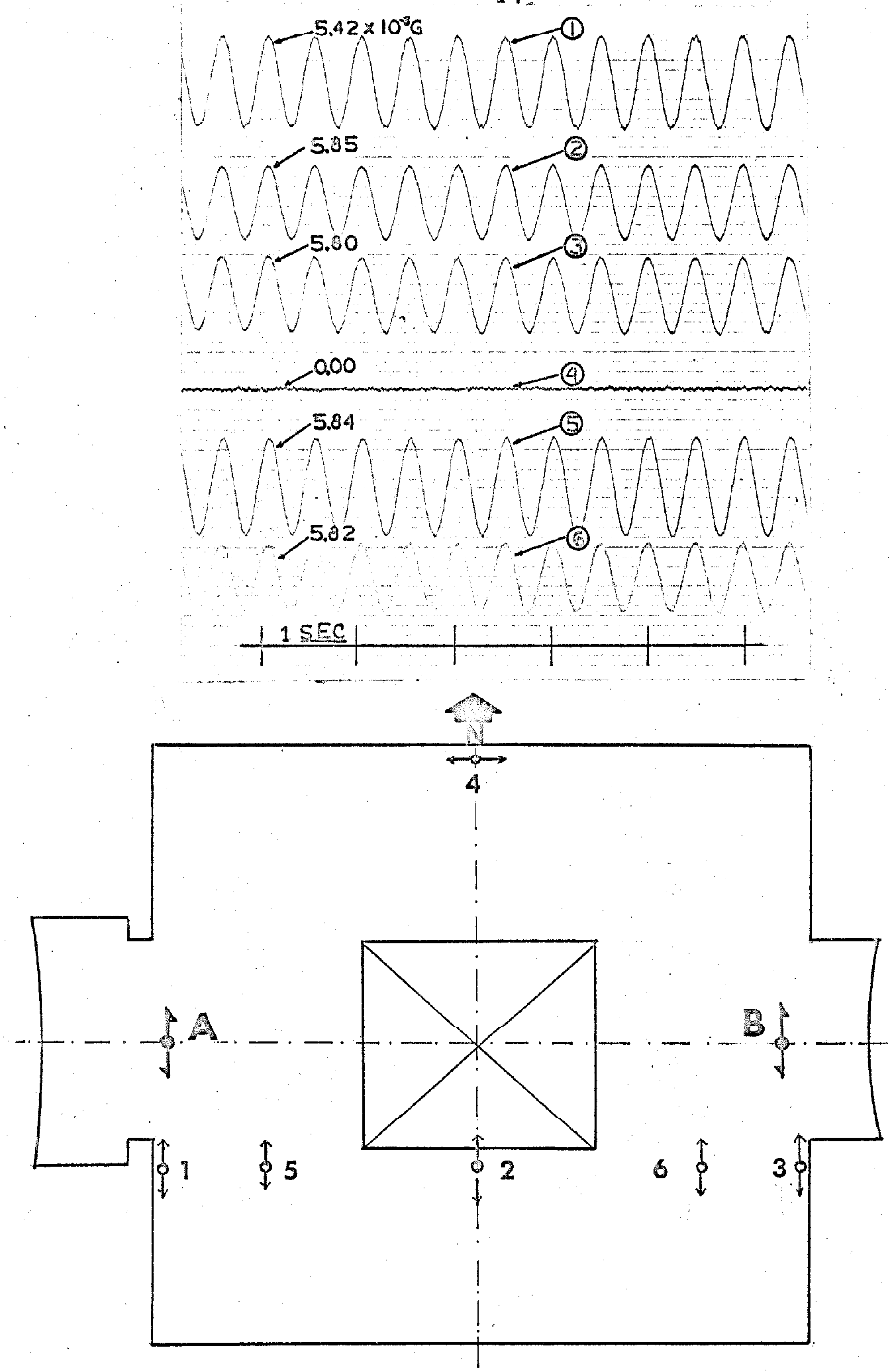


Fig. 2.1 9^{th} FLOOR RESPONSE IN THE FIRST MODE, N-S DIRECTION. ABOVE: SAMPLE RECORDED DURING THE TEST, BELOW: LOCATION OF THE ACCELEROMETERS.

check if this assumption still holds in this case. During this test the North and South side facades were in place.

Test results

The values of the acceleration recorded at each location were:

Acc. No.	Single Amplitude Acceleration ($g \times 10^{-3}$)	Orientation	Location*
1	5.42	N-S	One foot E of the West wall
2	5.85	N-S	One foot S of the center of the elevator box
3	5.80	N-S	One foot W of the East wall
4	0	E-W	One foot S of the center of the N facade
5	5.84	N-S	Between 1 and 2
6	5.82	N-S	Between 2 and 3

*The accelerometers oriented in the N-S direction were placed on a line running E-W, passing 1' South of the elevator box.

From the test results, it may be concluded that:

1) The floor system is stiff enough to move as a rigid body, so the assumptions discussed above are correct.

2) The first mode in the N-S direction is essentially a pure translational mode, with no component in the E-W direction.

3) The values of the acceleration in the N-S direction are the same at every point on the floor, except near the West shear wall, where the value of the acceleration is 6.9 per cent smaller than at the other locations. This is explained by the high rigidity of the West

side resisting elements (see Fig. 1.1).

4) This was a test of the whole instrumentation, under working conditions, and the results show that the synchronization of the exciters was precise and that the recording system was working correctly.

5) A similar test performed in the E-W direction confirmed that conclusions (1) and (2) hold for the E-W direction.

C. Determination of the Center of Torsion

For horizontal forces in the E-W direction, the resisting elements at every floor are symmetrically arranged with respect to a vertical plane through the E-W center line (see Fig. 1.1). Thus, the center of torsion can be expected to lie in that plane. But in the N-S direction the resisting elements on the West side are more rigid than those on the East side, so the center of torsion will be closer to the West shear wall than to the East.

To determine the location of the center of torsion, the building was excited with both shakers generating forces in the N-S direction, but 180° out of phase. Four accelerometers were placed on the 9^{th} floor and two on the 8^{th} floor, as shown in Fig. 2.2. All were oriented so that a clockwise acceleration of the building was recorded as positive.

The results are shown in Fig. 2.2 in which the location of four of the accelerometers is indicated also. The other two were placed on the 9^{th} floor, both 11'9" south from the E-W center line, one 8" from the East shear wall, and the other 8" from the West wall.

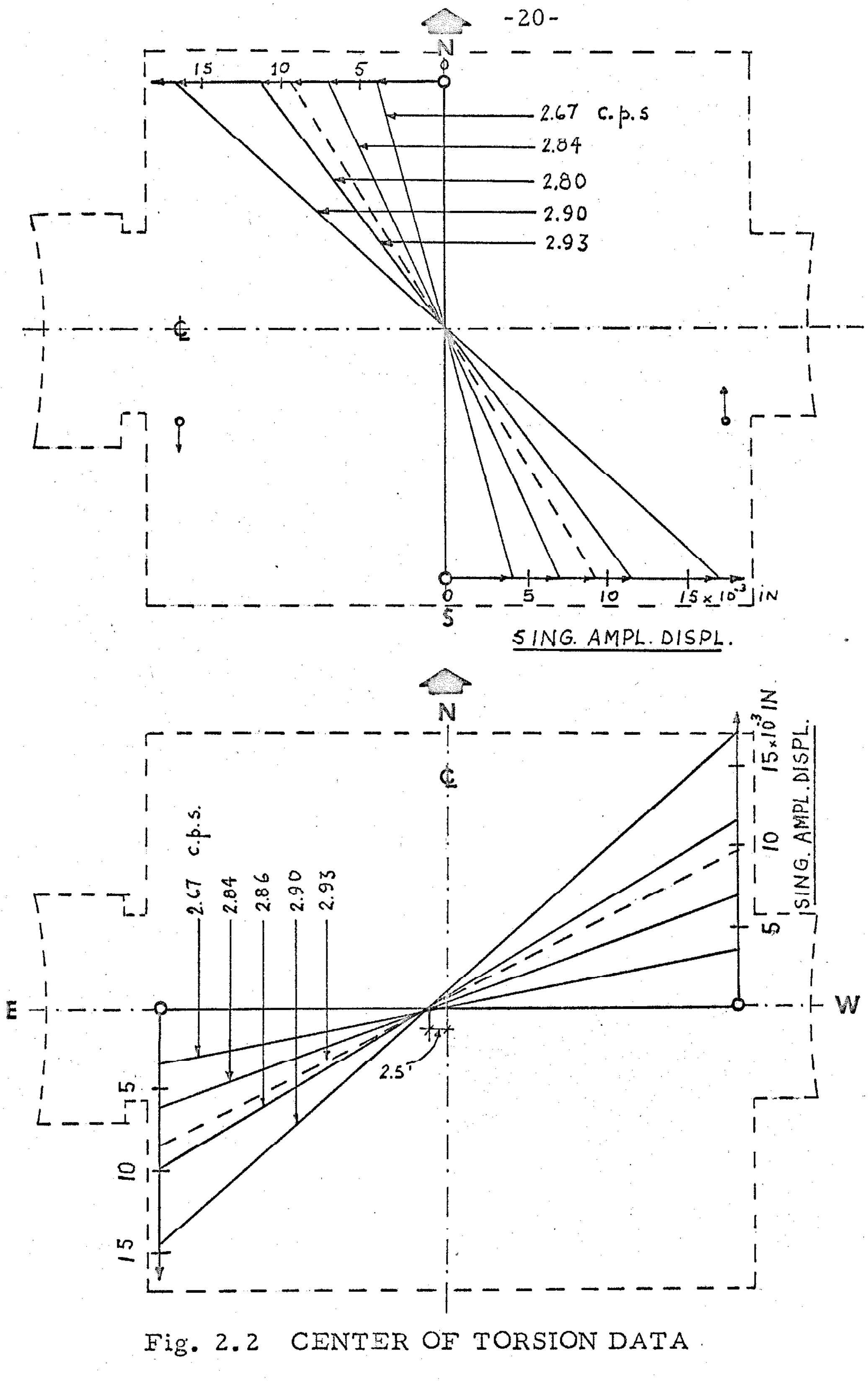


Fig. 2.2 CENTER OF TORSION DATA

From this test, the center of torsion was found to be on the E-W center line and 2'6" West of the geometrical center of the rectangular plan. This Westerly deviation tends to decrease toward ground level. The results also show that for this force level (1420 lbs at resonance) there is no accidental torsion due to difference in quality between the material used North and South of the vertical E-W plane of symmetry.

III. TRANSLATIONAL MODE IN THE N-S DIRECTION

Preliminary steady state tests indicated that in the N-S direction it was possible to measure the first mode only, that each floor was moving as a rigid body, and that the building was vibrating in a pure translational mode. Thus one accelerometer on each floor was sufficient to measure the vibration of the entire floor. The accelerometers were placed on the roof (10^{th}), 8^{th}, 6^{th}, 4^{th}, 2^{nd}, and 1^{st} floors for the three lower force levels. For the next three tests, the first floor accelerometer was moved to the basement; the others were left at the same locations. Each accelerometer was placed 1 foot North of the elevator box and 2-1/2 feet West of the N-S center line.

The vibration generators were located on the 9^{th} floor, on the E-W center line and 4 feet inward from the interior faces of the East and West shear walls, as shown in Fig. 1.1. The baskets were so arranged that the sinusoidal exciting forces acted in the N-S direction.

Test Results

a) Periods of Vibration

The periods increased slightly as the force levels were increased. This is a characteristic of a system with a nonlinear softening spring. The periods measured are given in Table 3.1 and the acceleration response curves in Fig. 3.1.

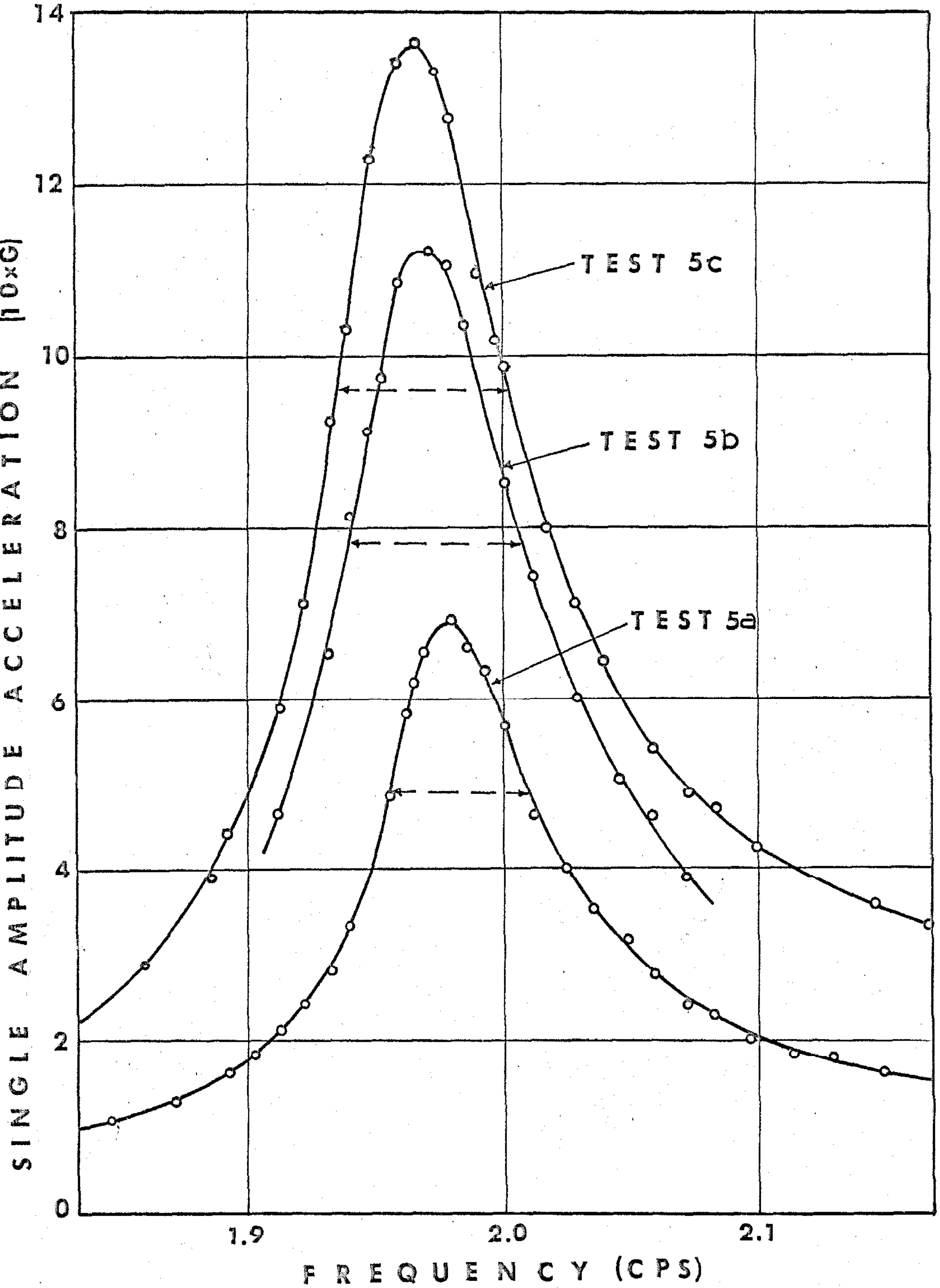


Fig. 3.1 ROOF RESPONSE FOR FIRST MODE, N-S DIRECTION

Table 3.1

First Mode Periods in the N-S Direction

Test No.	Weight Combination †	Force at Resonance (lbs)	Peak Freq. (c.p.s.)	Periods (sec)
5a	F_1	1448	1.98	.505
5b	F_2	2620	1.97	.507
5c	F_3	3320	1.96	.510
8a	F_4	4200	1.91	.523*
8b	F_5	5430	1.89	.530*
8c	F_1	1382	1.94	.515*

†Details are given in Table AI-2 of Appendix I.

*See text.

Note, in Table 3.1, how the natural period of the building increases with increases in the inertial force at resonance. The "jump" in the period values between tests 5c and 8a, in addition to showing the influence of the nonlinear characteristics of the building also shows the effects of the mass added to the building (mainly finishing material: plaster, air conditioning pipes, ceilings, etc.) during the week which elapsed between the two tests. The three periods marked with asterisks (*) were measured a week later than the preceeding three.

The last test (8c), gave a period 2 per cent greater than that measured one week earlier (5a), with the same load in the exciter baskets. The difference between the periods, measured with the maximum force level (test 8b), and that measured with the minimum (test 8c), was 2.9 per cent. Tests 8b and 8c were both made on the same day.

b) Acceleration and Displacement of Different Floors at Resonance

The values of the acceleration were computed directly from the recorded data, and the displacements were found by using the values of the acceleration and formula 1.7. The results are shown in Table 3.2.

Table 3.2

Resonant Acceleration and Displacement Amplitudes at Different Floors (1^{st} Mode N-S Direction)

Test	Force at Resonance (lbs)		Roof	8^{th}	6^{th}	4^{th}	2^{nd}	1^{st}	Basement
5a	1448	A*	6.95	5.32	3.51	2.33	1.30	.31	**
		D*	17.40	13.30	8.80	5.84	3.16	.78	
5b	2620	A	11.23	8.80	6.39	4.13	1.98	.64	
		D	28.40	22.20	16.10	10.40	5.00	1.62	
5c	3320	A	13.64	11.04	8.05	5.23	2.57	.96	
		D	34.70	28.10	20.50	13.30	6.55	2.44	
8a	4200	A	16.10	13.10	9.40	6.40	2.97	**	.38
		D	43.10	35.20	25.20	17.15	7.96		1.02
8b	5400	A	19.40	15.35	11.00	7.50	3.72		.47
		D	53.35	42.20	30.30	20.60	10.20		1.29
8c	1582	A	5.38	4.21	2.75	1.82	1.03		.057
		D	14.10	11.0	7.26	4.76	2.69		.149

A* 1^{st} row, single amplitude acceleration in "$g \times 10^{-3}$"

D* 2^{nd} row, single amplitude displacements in "inches $\times 10^{-3}$"

** no measurements were made at this location

c) Mode Shape

A mode shape was computed for each test. A number of normalized mode shapes were first found for different speeds of the rotating baskets. Average values of the normalized deflections found for each of the six floors considered were then computed, and these gave an average normalized mode shape for the test.

For each test the variation in the values of the normalized deflections for each floor was small, with the maximum deviation from the average being about 3 per cent. These variations were randomly distributed, and are thought to be within the range of the experimental error. The mode shape remained nearly constant during any one test, and was not affected by changes in the frequency of excitation.

Table 3.3

Mode Shape at Different Force Levels

Test	Force at Resonance (lbs)	Roof	8^{th}	6^{th}	4^{th}	2^{nd}	1^{st}	Basement
5a	1448	1.00	.78	.51	.34	.18	.045	*
5b	2620	1.00	.78	.55	.37	.18	.055	
5c	3320	1.00	.79	.57	.37	.18	.068	
8a	4200	1.00	.80	.57	.39	.18	*	.022
8b	5400	1.00	.79	.57	.39	.19		.025
8c	1582	1.00	.76	.51	.35	.19		.011

*No measurements were made at this location.

Table 3.3 shows that the mode shape from the 2nd floor upward is relatively stable. However, the relative displacements of the 1st floor for the first 3 tests and then of the basement for the last three tests show a consistent increase with increasing force level.

d) Damping

The damping was estimated in three ways: by the standard method; by using the mode shape; and by the simplified method proposed by Hudson.

The computed values of the damping at the roof are presented in Table 3.4.

Table 3.4

Damping in Percentage of Critical Damping

Test No.	Displacement Amplitude Ratio	Standard Method	Using Mode Shape	Hudson's Method
5a	1	1.16	1.24	1.44
5b	1.62	1.45	1.35	*
5c	1.97	1.48	1.39	1.51
8a	2.45	1.75**	1.49	1.88
8b	3.02	1.64	1.60	1.93
8c	0.79	1.54	1.47	1.74

*Only the response of the peak was measured, so it is not possible to apply this method.

**See text.

The damping values in the above table show a consistent increase with increasing force level.

It is interesting that before the data of test 8a was recorded, the building had been continuously excited at resonance, with the test 8a force level, for more than 2 hours. In each of the other tests, except 8c, the building was excited at resonance, with the respective weight combination, for only 50 to 100 cycles before the data were recorded, and no other excitation with the same or heavier weight combinations had previously been applied. Comparing the results of tests 8a and 8b shows that the response curves of test 8a are noticeably wider than those of test 8b, and this is clearly seen from the damping values calculated by the standard method, which gives values for test 8a bigger than those for test 8b (the next higher force level). The other two methods, which use a relation between amplitudes but do not take into consideration the width of the response curve, do not show these bigger damping values.

The response curve of test 8b shows the sharpness typical of the curves for tests 5a, 5b and 5c, in spite of the fact that in test 8a the building was excited at resonance, at 0.8 of the test 8b force level, for more than 7000 cycles. This result suggests that the damping value for a given amplitude remains unchanged by any previous vibrations at lower amplitudes. However, more research is needed to confirm this statement.

On the other hand, damping values from test 8c, which was performed at the same force level as test 5a, but after all the intermediate tests had been carried out (during which the building had experienced amplitudes 3 times bigger than those of test 5a) are considerably greater than the values from 5a. The 8c values computed by the 3 different methods were 23, 18.5 and 21 per cent greater than the corresponding values from test 5a.

Damping Computed at Different Floors

For test 8a the damping values corresponding to the floors where the accelerometers were located are given in Table 3.5, and the response curves for these floors are shown in Fig. 3.2.

Table 3.5

Damping at Different Floor Levels
(Test 8a, Force at Resonance: 4200 lbs)

Floor / Method	10th	8th	6th	4th	2nd	Basement
Standard	1.75	1.75	1.76	1.68	1.68	1.75
Mode shape	1.49	1.45	1.43	1.39	1.49	
Hudson's	1.88	1.99	2.00	1.99	2.00	

The damping was also computed at different floors for the other tests, and the results agree with the results in Table 3.5: that is, for each method, damping values are approximately the same for all floors. But, although values may be calculated for any floor and estimated for all the others, it is preferable to use the data of the upper floors because the relative error in the readings is then smaller.

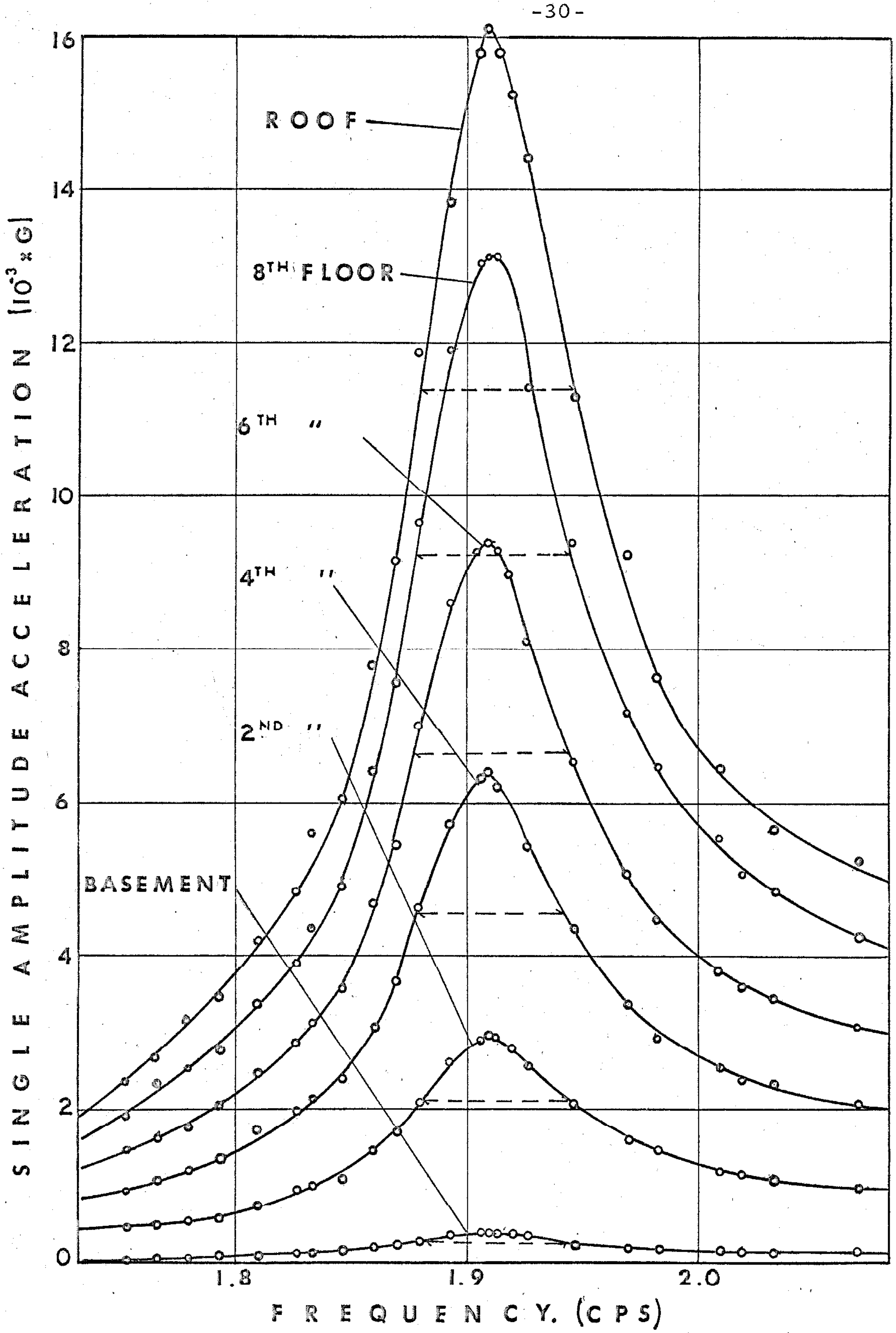


Fig. 3.2 RESPONSE CURVES FROM TEST 8a, RECORDED AT DIFFERENT FLOORS

IV. TRANSLATIONAL MODE IN THE E-W DIRECTION

In the E-W direction it was possible to excite both the first mode, which had no component in the N-S direction, and the second mode, which had a component in the N-S direction of about 1/3 its value. The shape of the second mode changed with the frequency of excitation. The ratio of the frequencies of the 2 lowest modes was 1 to 4.2, indicating that the building deformation due to bending was important in this direction.

The accelerometers were placed on the roof, 8^{th}, 6^{th}, 4^{th}, 2^{nd}, and basement floors, on the E-W center line in each case, 8 feet from the East elevator wall. The shakers were reset to act in the E-W direction.

Test Results

Periods of vibration

The periods corresponding to the first mode, and the exciting forces at resonance, are indicated in Table 4.1. The resonance curves are given in Fig. 4.1.

As in the N-S direction, the period in the E-W direction increased as the excitation force increased. However, in the E-W direction all the tests were performed during the same weekend, so no mass was added between tests. There was an increase of 2.72 per cent in the resonant frequency between the lowest and highest levels of excitation.

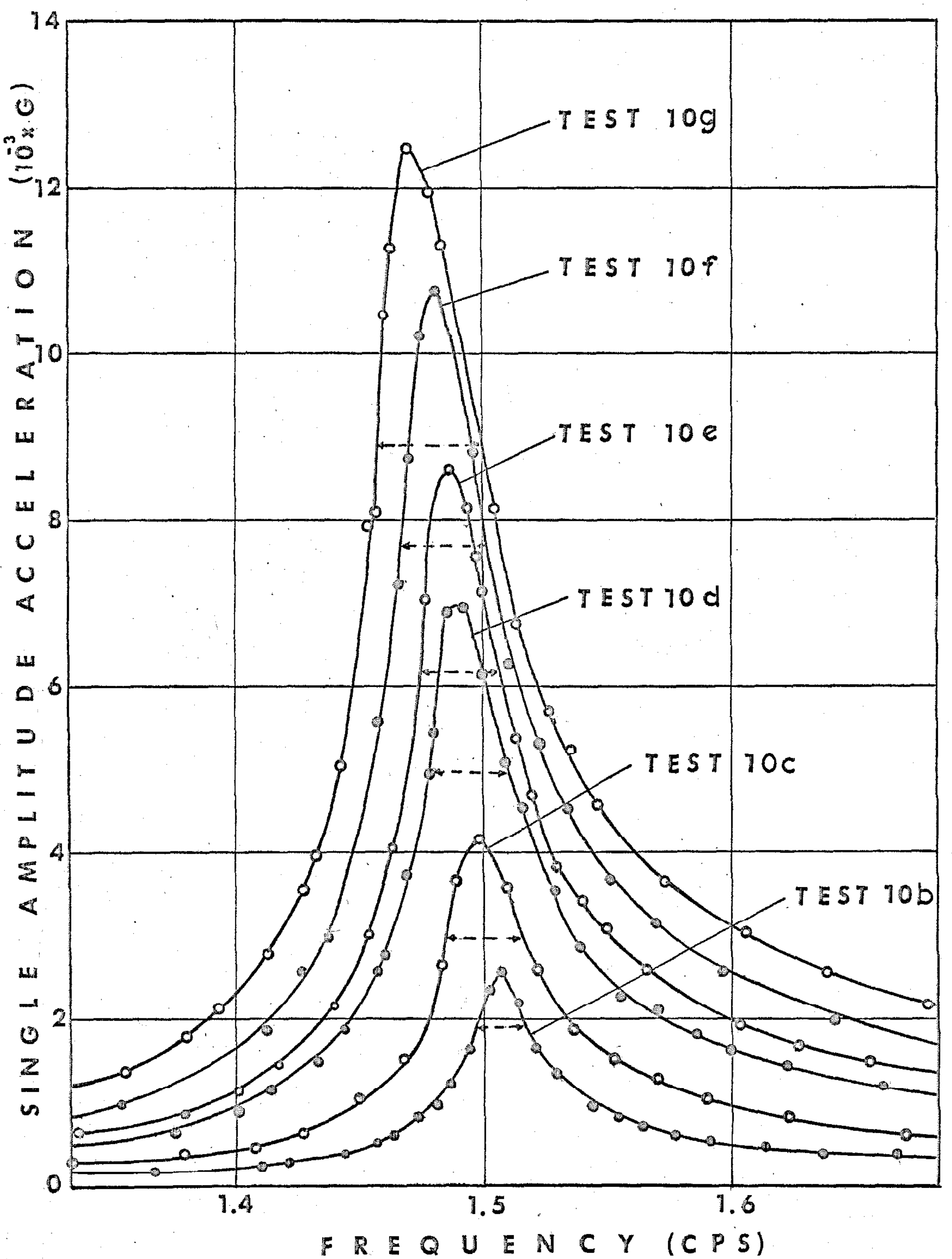


Fig. 4.1 RESPONSE OF THE 8th FLOOR, LOWEST TRANSLATIONAL MODE, E-W DIRECTION.

Table 4.1

First Mode Periods in the E-W Direction

Test No.	Weight Combination	Force at Resonance (lbs)	Peak Frequency (c.p.s.)	Period (sec)
10b	O_2	432	1.510	.662
10c	F_1	834	1.500	.666
10d	F_2	1495	1.490	.671
10e	F_3	1890	1.485	.673
10f	F_4	2620	1.480	.676
10g	F_5	3290	1.470	.680
10g*	F_5	3280	1.466	.682

*With the frequency of excitation decreasing.

The resonant periods are plotted versus the inertial forces at resonance in Fig. 4.2. Note in that figure that the period increases faster with respect to the exciting force at low force levels than it does at high force levels. The maximum relative displacement between adjacent floors was at the 2 upper stories (8^{th} and 9^{th}), and ranged between .002" and .010" for tests performed at the lowest and highest force levels, respectively.

From the results of other tests, and by considering the likely behavior of buildings near collapse, it is possible to sketch a complete curve for resonant frequency vs. inertial force. Increases in periods of buildings measured for structures excited by small disturbances (vehicular traffic, low velocity wind or rhythmical

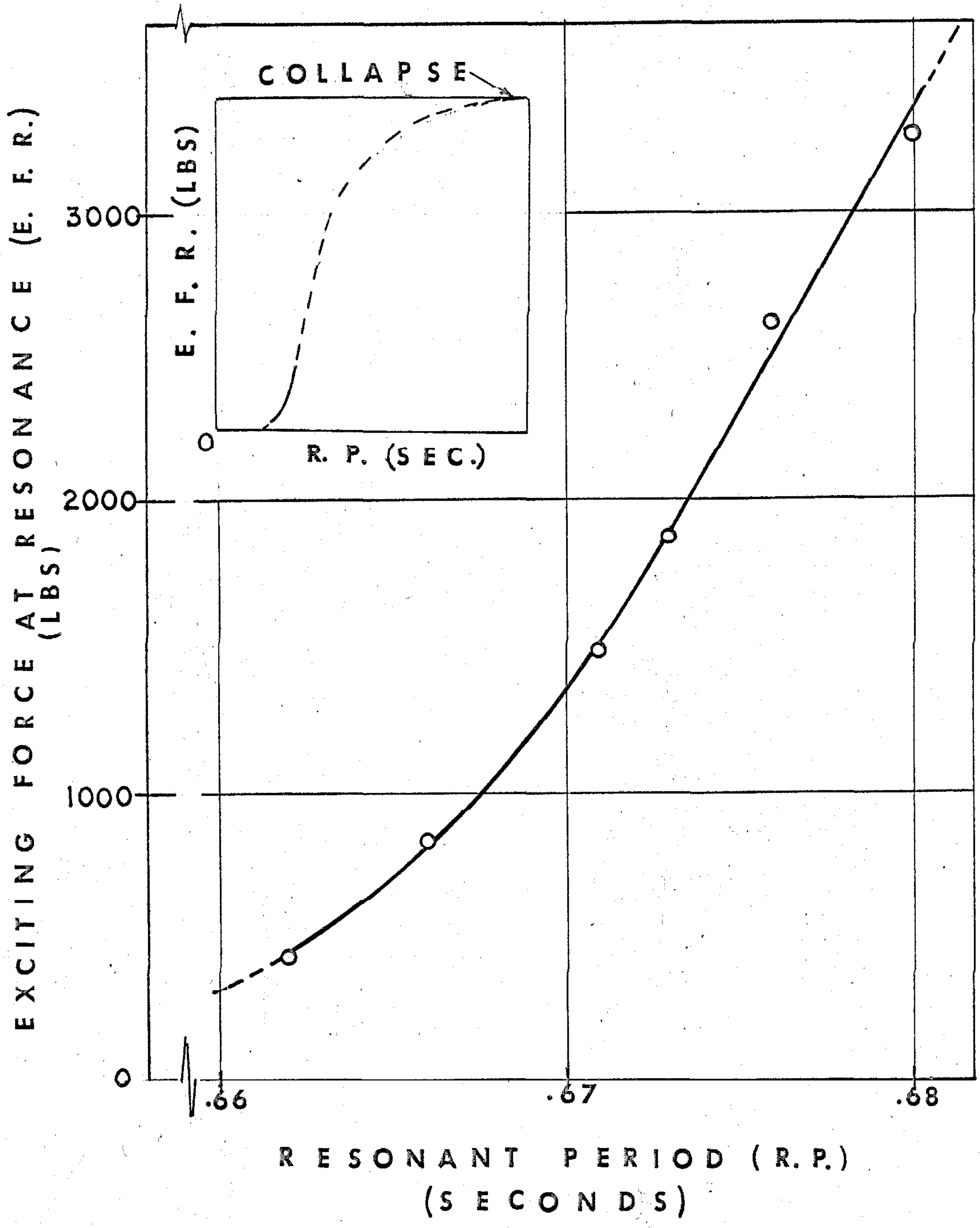


Fig. 4.2 RESONANT PERIODS VERSUS INERTIAL FORCES AT RESONANCE, LOWEST TRANSLATIONAL MODE, E-W DIRECTION

movement of the operator's body) have been no more than 10 per cent of the periods found by steady state tests within the elastic range of the structure. On the other hand, some Japanese tests performed on a relatively small frame type reinforced concrete structure (see ref. 12), for which the influence of the ground on the dynamical response was found to be much less important than for a typical Japanese building, and for which the stresses developed were high enough to cause fracture of the structural elements, have shown increases of 200 per cent or more in the values of the periods as a result of increases in the exciting force. In addition, if the exciting forces continue to increase after yielding, tangents to the curve would tend to become horizontal because of the ductility shown by reinforced concrete, and the period would tend to infinity if the structure collapsed in a ductile manner. The appropriate curve would then be like the dotted curve shown in the upper left of Fig. 4.2.

This emphasizes the fact that computing the periods of vibration of structures is a complicated problem, because the periods increase at an unknown rate with increases in the exciting force. It also shows that values of the periods would be better defined if they were related to corresponding levels of stress in the structure. However, at the present time it is very difficult to compute the period of a structure for a given level of stress.

The period corresponding to the second mode was not well defined. The maximum acceleration at different floors did not occur at the same instant, and the peak responses of the different floors did not occur at the same frequency because of coupling with vibration in the N-S direction. However the peak responses of the 6^{th}, 4^{th}, and 2^{nd} floors were at the same frequency, as is shown in Fig. 4.3. For these floors the measured period was 0.161 sec when the force at resonance was 3880 lbs, and 0.162 sec when the force at resonance was 7270 lbs.

<u>Acceleration and Displacement Amplitudes at Resonance</u>

These values at the different floors were computed in the same way as the corresponding values in the N-S direction. The results for the first mode in the E-W direction are given in Table 4.2.

Table 4.2

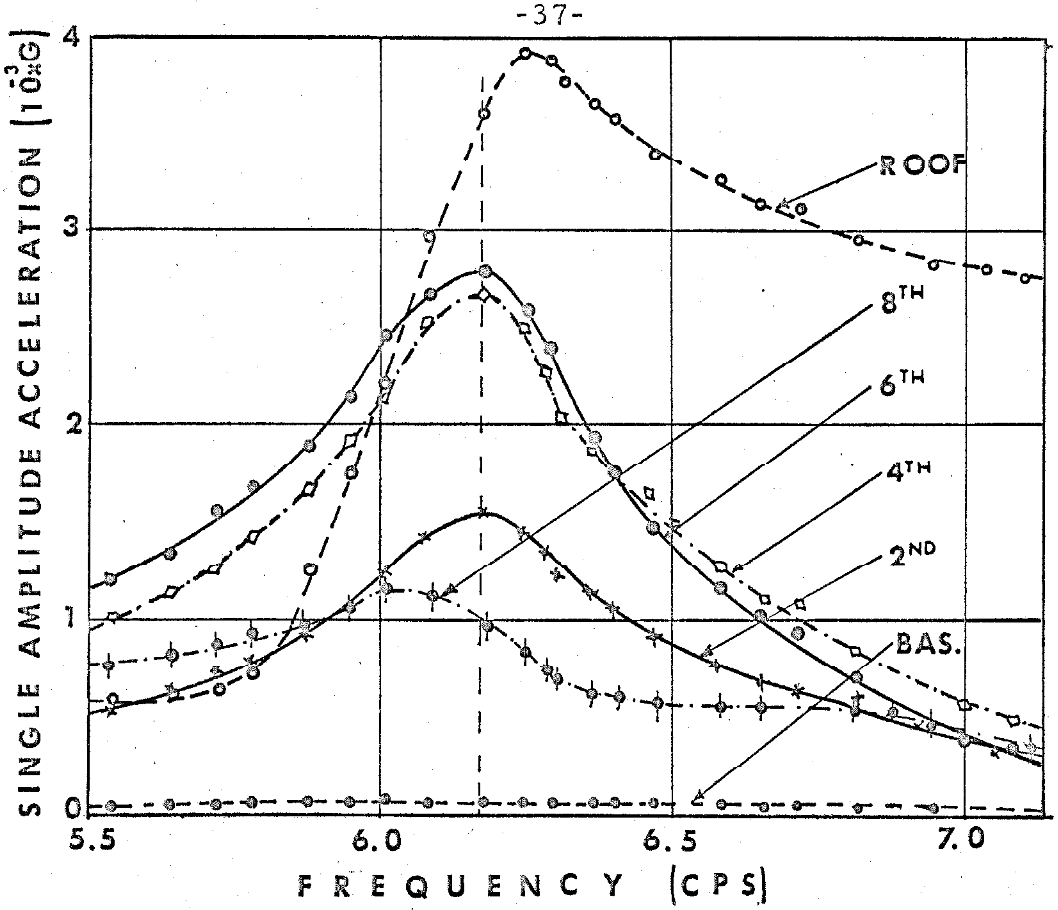
Resonant Acceleration and Displacement at Different Floors (1st Mode E-W Direction)

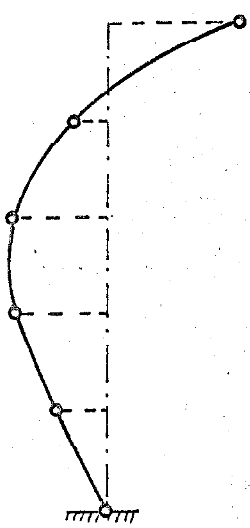
Test	Force at Resonance (lbs)		Roof	8^{th}	6^{th}	4^{th}	2^{nd}	Basement
10b	432	A*	3.52	2.34	1.59	.95	.36	0.011
		D*	14.90	9.92	6.75	4.13	1.53	0.047
10c	834	A	5.94	4.17	2.79	1.67	.68	0.018
		D	25.50	17.90	11.95	7.16	2.92	0.077
10d	1495	A	9.36	6.96	4.25	2.72	1.05	0.029
		D	40.80	30.40	18.55	11.90	4.57	0.129
10e	1890	A	11.55	8.61	5.30	3.30	1.24	0.038
		D	51.30	38.20	23.50	14.65	5.50	0.168
10f	2620	A	14.50	10.71	6.70	4.10	1.58	0.032+
		D	65.20	48.20	30.20	18.45	7.10	0.144
10g	3290	A	16.70	12.50	7.70	4.62	1.78	0.054
		D	75.20	56.60	34.90	21.00	8.07	0.245

A* First row, single amplitude acceleration in "$g \times 10^{-3}$"

D* Second row, single amplitude displacement in "$in \times 10^{-3}$"

\+ Malfunction of the amplifier.





FLOOR	ACCEL.	M. SHAPE
10	+ 3.60 10^{-3} G	+ 1.00
8	- .97	- .27
6	- 2.77	- .77
4	- 2.67	- .74
2	- 1.55	- .43
BASE.	.08	- .02

Fig. 4.3 a) RESPONSE AT DIFFERENT FLOORS, SECOND MODE E-W DIRECTION. b) MODE SHAPE AT 6.2 c.p.s. (TEST 10b).

Mode Shape

The mode shape for the lowest force level, 432 lbs at resonance, changed with the frequency of excitation as shown in Table 4.3.

Table 4.3

Shape of the First Mode for the Lowest Excitation (E-W Direction)

Frequency	10th	8th	6th	4th	2nd	Basement
1.44	1.00	.59	.46	.29	.14	.0046
1.47	1.00	.65	.48	.28	.12	.0039
1.50*	1.00	.66	.46	.28	.10	.0032
1.54	1.00	.72	.49	.30	.12	.0034
1.55	1.00	.66	.45	.28	.11	.0034
1.58	1.00	.55	.39	.22	.09	.0033

*Resonant frequency

The mode shape for the force levels of tests 10c to 10g does not vary with either change of frequency or increase in the force level. The values of Table 4.4, except for the results of test 10b where the mode shape corresponds to the resonant frequency, are the average of the values at different frequencies of excitation. The small discrepancies are within the range of the experimental errors.

Table 4.4

Mode Shape at Different Force Levels

Test	Force at Resonance (lbs)	10^{th}	8^{th}	6^{th}	4^{th}	2^{nd}	Basement
10b	432	1.00	.66	.46	.28	.10	.0032
10c	834	1.00	.71	.48	.28	.11	.0031
10d	1495	1.00	.75	.48	.28	.11	.0032
10e	1890	1.00	.74	.48*	.29	.11	.0033
10f	2620	1.00	.75	.48*	.29	.11	.0030
10g	3290	1.00	.75	.48*	.28	.11	.0033

*The accelerometer was found to be rotated about 40° from the E-W direction, and the necessary corrections have been made.

The shape of the second mode does change with frequency. The frequency at which the peak response of the 6^{th}, 4^{th}, and 2^{nd} floors occurred is given in Fig. 4.3.

Damping

The damping values for first mode excitation in the E-W direction, calculated from data recorded at roof level, are given in Table 4.5.

Table 4.5

Damping in Percentage of Critical Damping

Test No.	Displacement Amplitude Ratio	Standard Method	Using Mode Shape	Hudson's Method
10b	1.00	.69	.80	.97
10c	1.71	1.06	.89	.96
10d	2.74	1.12	.97	1.17
10e	3.44	1.12	1.01	1.15
10f	4.38	1.21	1.16	*
10g	5.08	1.48	1.20	1.50
10g†	5.20	1.70	1.16	1.46

*Only frequencies close to the peak were measured, so it is not possible to apply this method.

†See text.

Note in Table 4.5 that the damping consistently increases as the amplitude of vibration increases. All the damping values were computed from data recorded with increasing frequency. They were found to be smaller than the damping values computed for decreasing frequency (e.g., test 10g†, Table 4.5). The latter type of tests gives a wider curve, but a smaller resonant frequency than the former.

The last two methods in Table 4.5 gave slightly smaller damping values in test 10g† than in test 10g because the peak acceleration was slightly larger in the decreasing frequency curve, which was not the usual case.

V. TORSIONAL MODE

Only the first torsional mode was studied. The second mode was just out of the effective range of the vibration generators. During the preliminary tests it was found that one shaker was enough to excite the building in torsion. So for the torsional tests, only the shaker located on the East side was used, generating exciting forces in the N-S direction. Because of the relatively high value of the first torsional natural frequency, the maximum load that could be used safely was the weight combination F_3.

The accelerometers were located over the E-W center line, 8 feet from the East shear wall, on the roof, 8th, 6th, 4th, 2nd, and basement floors.

Test Results

a) Periods of Vibration

The periods corresponding to 3 force levels were measured and are indicated in Table 5.1.

Table 5.1

First Torsional Mode Periods

Test No.	Weight Combination	Force at Resonance (lbs)	Peak Frequency (c.p.s.)	Period (sec)
9a	F_1	1535	2.885	.346
9b	F_2	2775	2.865	.349
9c	F_3	3525	2.860	.350

In the torsional mode, the natural periods increased by 1.2 per cent when the exciting force was increased 2.3 times.

b) Acceleration and Displacement Amplitudes at Resonance

The values of the acceleration and displacements at the location indicated above are given in Table 5.2.

Table 5.2

Acceleration and Displacement at Different Floors at Resonance (Lowest Torsional Mode)

Test No.	Force at Resonance (lbs)		Roof	8^{th}	6^{th}	4^{th}	2^{nd}	Basement
9a	1535	A*	9.80	7.64	5.30	1.63	1.36	.24
		D*	11.55	9.00	6.25	1.92	1.61	.28
9b	2775	A	16.50	13.50	10.15	3.62	2.77	.42
		D	19.63	16.05	12.10	4.30	3.30	.50
9c	3525	A	19.70	15.95	11.90	4.20	3.27	.50
		D	23.60	19.10	14.30	5.03	3.92	.60

A* Single amplitude acceleration in "$g \times 10^{-3}$"

D* Single amplitude displacement in "$in \times 10^{-3}$"

c) Mode Shape

The mode shape changed with the frequency of excitation in test 9a, but remained stable during tests 9b and 9c. The relative value of the horizontal displacement of the basement, as in the N-S direction tests, increased with respect to the values of the other

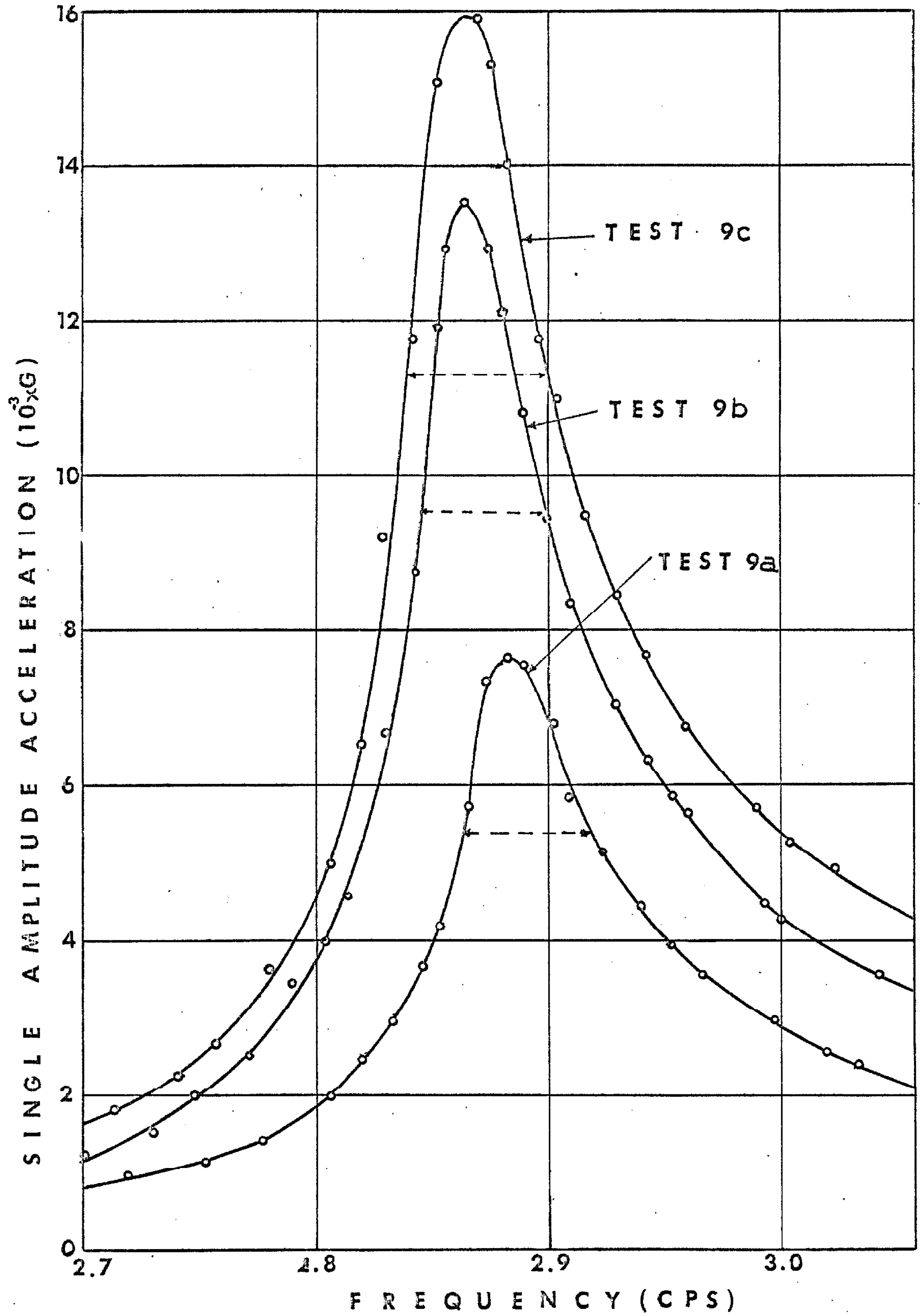


Fig. 5.1 8th FLOOR RESPONSE, LOWEST TORSIONAL MODE

components as the force levels were increased, as shown in Table 5.3. The mode shapes were computed using the values of the accelerations recorded at the locations indicated in the introduction of this section, but since the accelerometers were located on a vertical line the mode shape computed using the angular rotation at each floor would give the same results.

Table 5.3

Lowest Torsional Mode Shape

Test No.	Force at Resonance (lbs)	10^{th}	8^{th}	6^{th}	4^{th}	2^{nd}	Basement
9a	1535	1.00	.80	.54	.17	.14	.023
9b	2775	1.00	.82	.60	.22	.16	.025
9c	3525	1.00	.81	.60	.22	.16	.026

d) Damping

In torsion, as in the N-S and E-W direction tests, the damping value increased as the amplitude of vibration increased. The results are given in Table 5.4.

Table 5.4

Damping in Percentage of Critical Damping

Test No.	Displacement Amplitude Ratio	Standard Method	Using Mode Shape	Hudson's Method
9a	1.00	.93	.98	1.51
9b	1.70	.96	.99	1.40
9c	2.04	1.05	1.06	1.65

VI. BUILDING-GROUND COUPLING

A. Basement Motion Investigation

When a simplified seismic structural analysis of a building is made, for the common type of soil encountered on the West Coast of the USA, it is supposed that the building is fixed at the ground level. For a more accurate analysis, especially during a research study, the foundation-ground interaction is assumed to satisfy certain analytic relations, so the problem can be resolved, usually with the aid of a computer. Several mathematical models have been proposed[17,18,19] and the building ground coupling problem has been studied by various researchers[20,21,22,23,24], but the horizontal displacement of that part of the building below the ground level has not been specifically investigated in this country during a building vibration test, and there is a little data available on the influence of embedment.

It was thought that the Millikan Library Building, which is more rigid in the N-S direction than in the E-W direction, and which has a basement, would be a good building to use to investigate the adequacy of the assumption that the building is fixed at ground level.

The horizontal displacement at the ground floor and the basement floor, and the rocking vibration of the building on its base, were measured in the N-S and E-W directions and the soil characteristics at the construction site were studied, so significant comparisons with other cases may be made.

During the tests in each direction, horizontal accelerometers were placed on the roof, 6^{th}, ground level and basement floors exactly as for the translational tests. Two vertical accelerometers were placed on the basement floor, at opposite ends of the N-S center line for the N-S tests, and at opposite ends of the E-W center line for the E-W tests.

Each test was made at the first mode resonant frequency of that direction, and the resonant exciting forces were 5430 lbs in the N-S direction and 2740 lbs in the E-W direction.

Test results

a) The horizontal displacement at the ground floor and at the basement floor, as percentage of the horizontal displacement of the roof, were:

In the N-S direction

Ground floor	9.6 per cent
Basement	2.2 per cent

In the E-W direction

Ground floor	4.0 per cent
Basement	0.3 per cent

b) The rocking vibration of the whole structure on its base caused a horizontal displacement at the top of the building which, expressed as a percentage of the total deflection at the top, was 0.8 in the N-S direction and 0.4 in the E-W direction.

Conclusions

1) The horizontal displacements at the ground floor were important in both directions, but were larger in the N-S direction (9.6 per cent of that of the roof) than in the E-W direction (4.0 per cent). This difference may be explained by the fact that the building is about twice as stiff in the N-S direction as in the E-W direction.

2) The horizontal displacement of the basement floor with respect to the displacement at the roof was small but significant (2.2 per cent) in the N-S direction, and negligible (0.3 per cent) in the E-W direction. The reason is that the resisting elements in the N-S direction have practically constant section from the top of the building to the foundation, but the resisting elements in the E-W direction have an abrupt change of section, from frames above ground level to a pair of solid exterior walls below. These walls, which are 75' long and 12 inches thick, add 74 per cent to the shear resisting area, and result in an increase of the total moment of inertia of the resisting sections of the basement to about 9 times that for the stories above.

3) The structural characteristics of the building in both directions, as explained in 1 and 2, are reflected clearly in the mode shapes in Fig. 6.2 and Fig. 6.3.

4) Although the compaction of the backfill was accomplished to 90 to 100 per cent of the maximum density, and had been done 9 months before these tests were performed, the soil surrounding the basement was not rigid enough to prevent the horizontal displacement of part of the building below the ground level, and during tests the

building behaved much as if no backfill were present at all.

5) In a simplified seismic analysis of a building with a basement, for similar conditions to those existing at the Millikan Library Building site at the time the tests were done, it would be more realistic to consider the building fixed at the foundation than at the ground level.

It is possible to extrapolate this conclusion to most of the cases of buildings with basements if the following reasoning is used:

If a building is being acted upon by horizontal forces and the portion of the building between the foundation and ground surface is considered, the horizontal displacement increases upward from a negligible amplitude at the foundation (see Fig. 6.2 and 6.3). At the same time the passive resistance of the soil to the horizontal deflection decreases upward, as the depth of soil becomes less. Thus where the horizontal movements are greatest, the resistance of the soil to them is least. Even in the case where the soil surrounding the basement has been well compacted, and the time elapsed has been long enough to permit it to reach its state of natural consolidation, it is very doubtful that the soil could prevent the small displacement of a much more rigid element, such as a reinforced concrete wall.

6) The soils underlying the foundation, which were undisturbed, were firm enough to prevent significant rocking. Also it is important that the soils are cohesionless (see Appendix IV) so that their strength and rigidities depends on the confining pressure.

This finding is reasonable when it is realized that the soils below the foundation are under quite different conditions than the soils that surround the basement. First, the foundations are usually placed over a clean undisturbed soil; secondly, for a building with a basement, the foundation lies about 18 feet below the ground level and finally, these soils are steadily and increasingly loaded during the construction period of the building, thus at the time the building is completed these soils support a considerable load.

7) The results of the rocking vibration tests agree with the conclusions of studies made by G. W. Housner[20] comparing accelerograms recorded at the basement of a building and at a nearby parking lot, and with a theoretical research made by G. W. Merrit and G. W. Housner[17] with the aid of an analog computer in which the foundation compliance was changed through a wide range. In that investigation it was found that significant rocking effect could be expected only in exceptionally soft ground, and this is not the case at the Millikan Building site.

B. Vibration of the Ground in the Vicinity of the Building

The next question investigated was the relative displacement of the lower portion of the building and the nearby ground.

Most of the investigations of this type have been made in connection with the vibrations of machine foundations, where high speeds are predominant (800 to 3000 r.p.m.). To solve the problem analytically, essentially two approaches have been used:

a) It is assumed that the vibrating system behaves as a single mass, supported by a weightless spring and subjected to viscous damping. Tests have shown that it is necessary to consider in the analysis the effective mass of the soil which is vibrating in phase with the foundation, if the theoretical prediction is to agree with experimental results. The initial advantage of the simplicity of this method is outweighed by the difficulty of computing the effective mass. This approach has been used by Lorentz,[25] Heukelom,[26] Crockett and Hammond[27] and other investigators.

b) The foundation-ground system is reduced to a model consisting of a solid block, resting on a semi-infinite elastic space or on a layered medium. During the modeling several simplifying assumptions have to be made, and to reconcile calculated and experimental results, it has been necessary to introduce some adjustments. Barkan,[28] Sung,[29] Arnold, Bycroft and Warburton,[30] Toriumi,[31] and others have applied this method in their research. Additional bibliography may be found in references 32 and 33.

Comparative studies of seismograms recorded at the basement, and on the ground at short distances from a building, have been made by Housner and Kanai. Details of the results of those studies may be found in references 20 and 34 respectively. It is interesting to note that for the accelerograms used by Housner, which were recorded at Hollywood, USA, there is no noticeable difference between those recorded at the basement and those recorded on the ground, but for the seismograms used by Kanai, recorded at Tokyo, Japan, the maximum amplitude of the seismogram recorded on the ground, close to the building, varies between 200 and 500 per cent of the maximum amplitude of that recorded in the basement.

During a building vibration test, the handicap of not having the phenomenon of a real earthquake is compensated for by the advantage of being able to change the location of the vibration pick-ups, and the possibility that readings may be taken simultaneously at various places. It was thought that by comparing the vertical and horizontal displacements at different distances from the structure, it would be possible to get some indication of how the energy was being transferred from the building into the medium around it. During the basement motion investigation and soil vibration test in the N-S direction, when the vibration generators were continuously running at resonance for more than 2 hours, an unexpected phenomenon occurred: the elastic waves originating at the foundation of the Millikan Building were recorded 3 miles away at the Caltech Seismological Laboratory. (Subsequently, a separate investigation was carried out by members of the Caltech Staff to study the nature of the induced motion in the Pasadena area using the building excited in the N-S direction as energy source. The report is being prepared for publication.)

The tests were performed in the N-S and E-W directions by exciting the building in each direction with constant amplitude, at the resonant frequency of the lowest translational mode. A horizontal accelerometer located on the roof, and another on the ground floor, were used for control and reference, and were maintained fixed in those locations throughout the tests in each direction, while 2 horizontal and 2 vertical accelerometers were moved around outside the building. The outside accelerometers were in 2 sets, each set consisting of one vertical and one horizontal accelerometer placed on a 43 pound steel plate, which was firmly fixed on the ground while

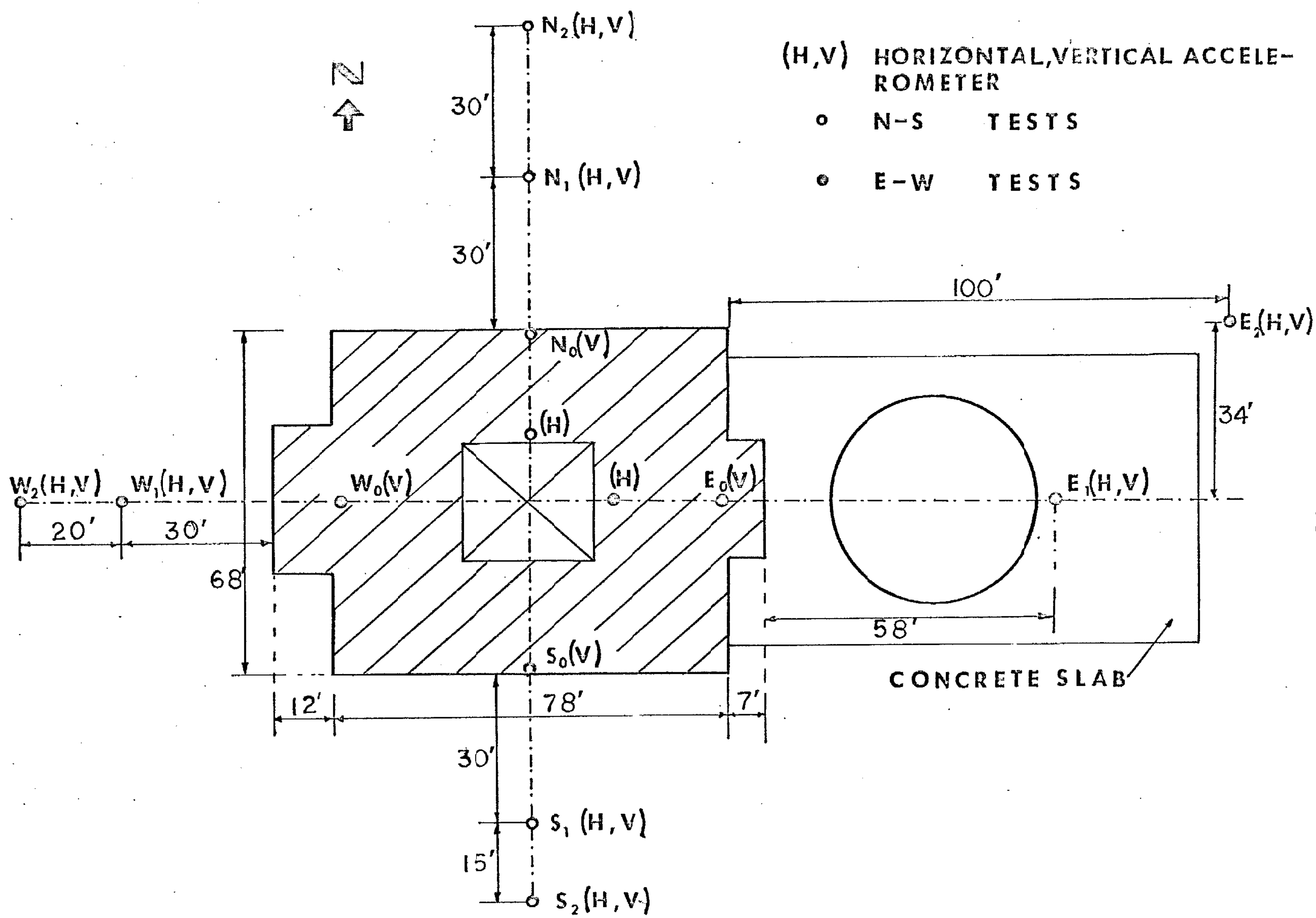


Fig. 6.1 LOCATION OF THE ACCELEROMETERS DURING THE BUILDING-GROUND COUPLING TESTS

each record was taken.

The locations of the accelerometers and the types used at each place may be seen in Fig. 6.1. It was not possible to place all the accelerometers completely symmetrically with respect to the building because of the desire to locate the instruments over undisturbed soils, and for other reasons that will be explained later. It was possible to place the accelerometer on undisturbed soil in all the cases in the N-S direction tests, but not in the E-W direction.

Test Results

The amplitudes of the ground vibration, normalized with respect to the motion of the roof, are indicated in Fig. 6.2 for the N-S direction tests and in Fig. 6.3 for the E-W direction tests.

The horizontal displacements of the ground, expressed as percentages of the displacements of the first and basement floors, are given in Table 6.1. A graphical comparison may be seen in Fig. 6.4.

The vertical displacements of the ground, normalized with respect to the vertical displacement of the North edge of the first floor for the N-S direction, and with respect to the vertical displacement of the West shear wall and the first floor level, for the E-W direction, are shown in Fig. 6.5.

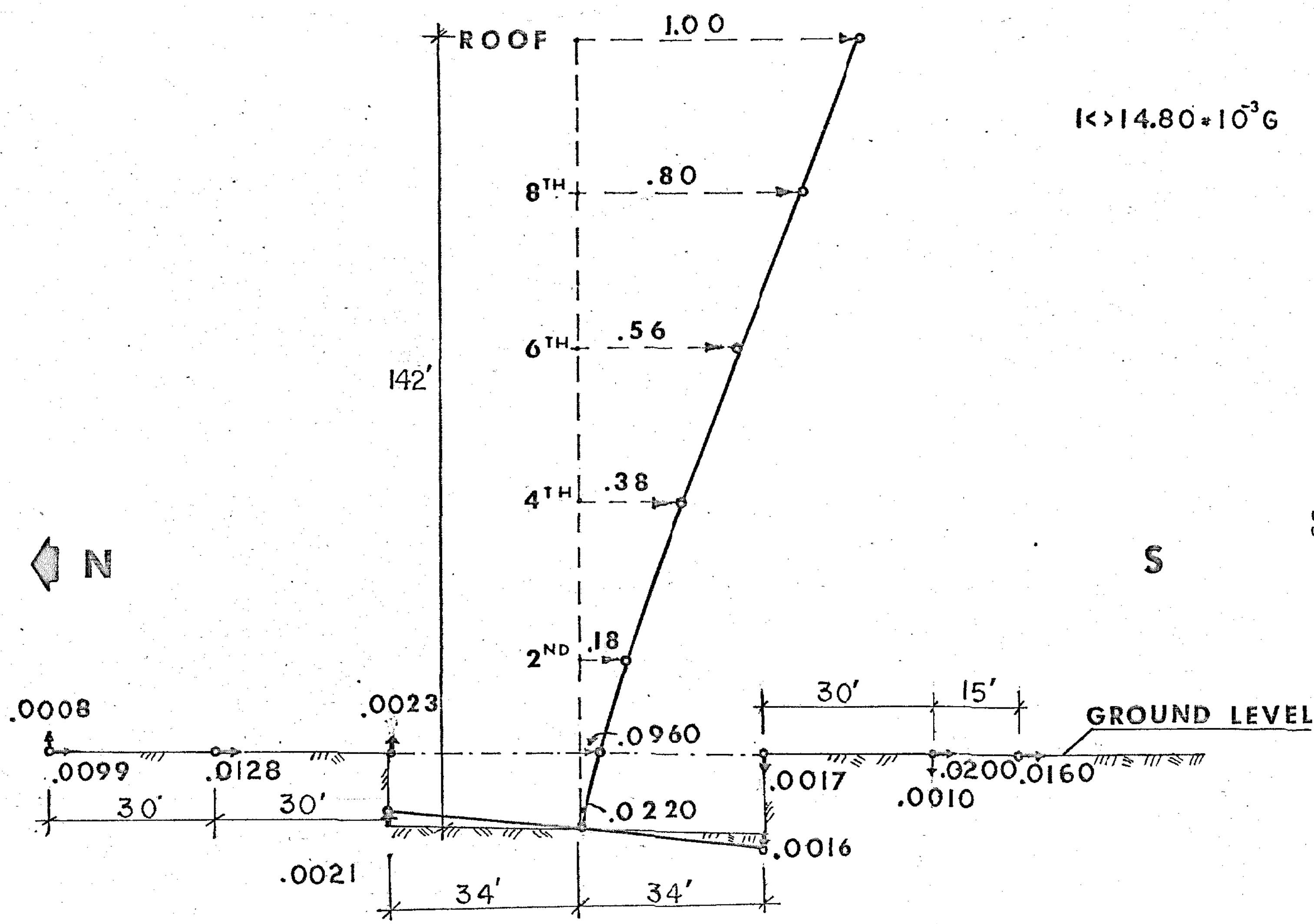


Fig. 6.2 RELATIVE MOTION OF THE BUILDING AND THE NEARBY GROUND, N-S TESTS

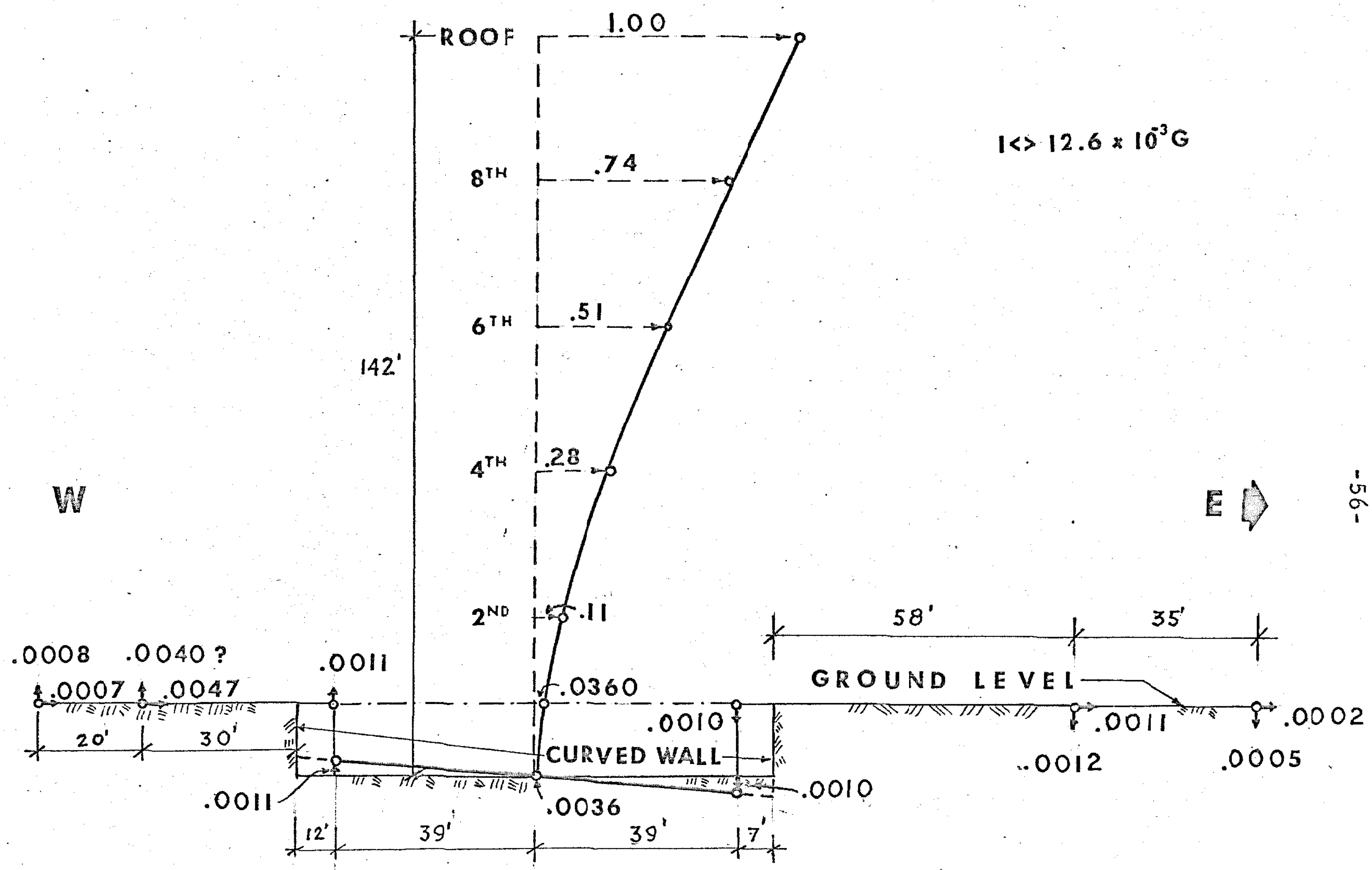


Fig. 6.3 RELATIVE MOTION OF THE BUILDING AND THE NEARBY GROUND, E-W TESTS

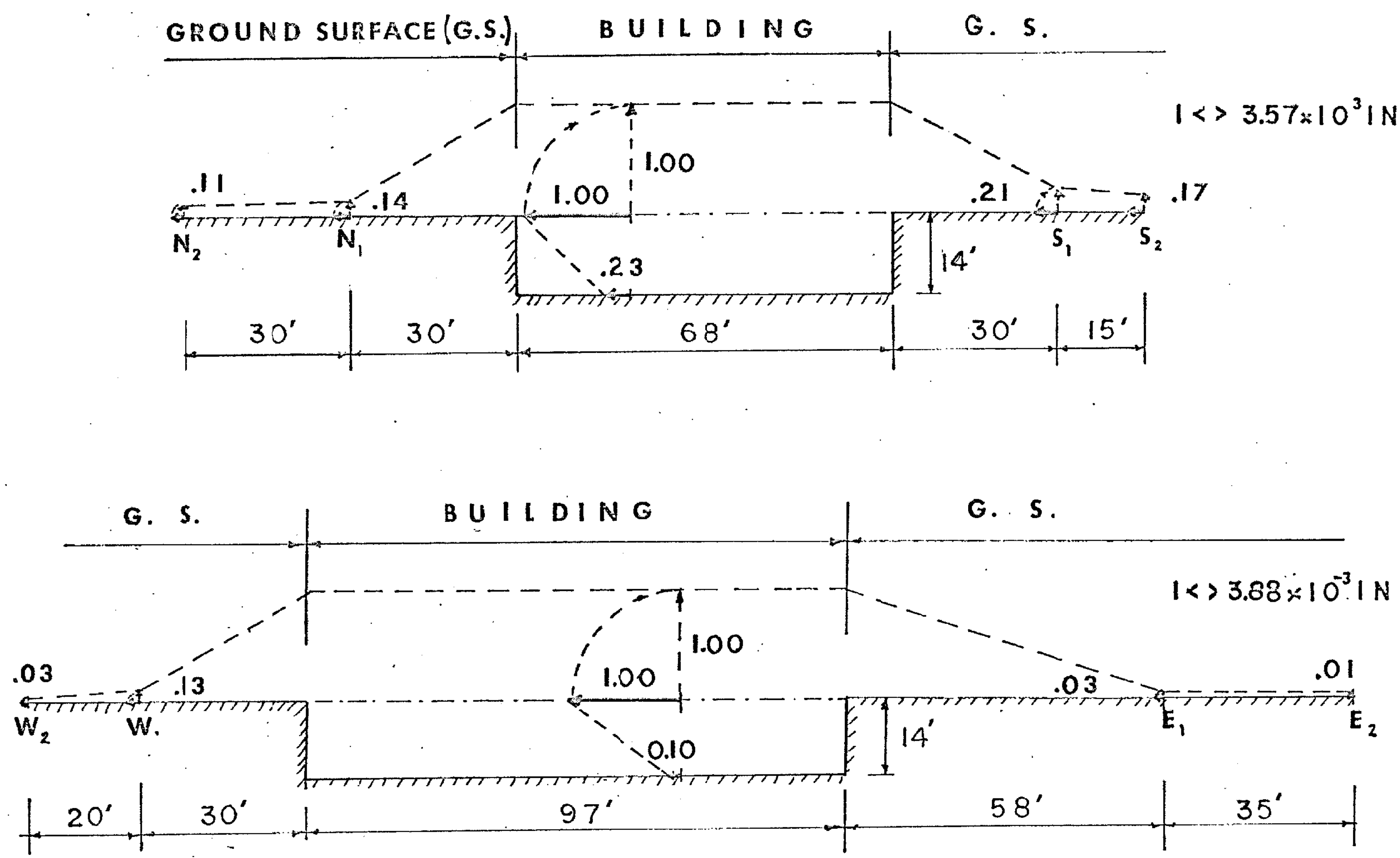


Fig. 6.4 RELATIVE HORIZONTAL MOVEMENT OF THE BASEMENT AND THE NEARBY GROUND. ABOVE; N-S DIRECTION, BELOW: E-W DIRECTION

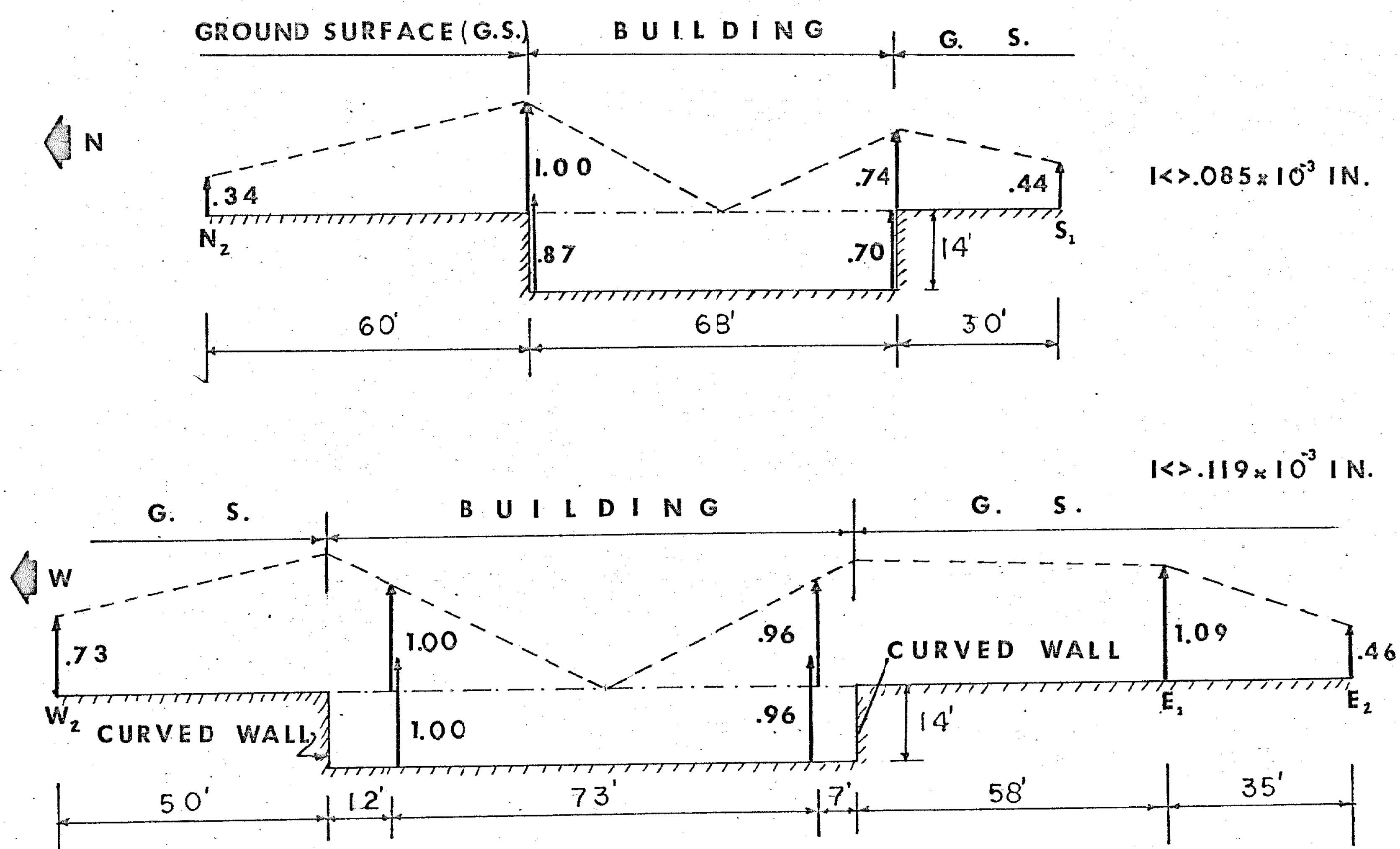


Fig. 6.5 RELATIVE VERTICAL MOVEMENT OF THE BASEMENT AND THE NEARBY GROUND. ABOVE: N-S DIRECTION, BELOW: E-W DIRECTION

Table 6.1

Relative Horizontal Displacement of the Ground

Location (See Fig. 6.1)	Distance From Building (feet)	Test Direction	Horizontal Displacement (HD) (in per cent) of the HD of the 1st floor	of the HD of the basement floor
N_1	30' N	N-S	14	61
N_2	60' N	"	11	48
S_1	30' S	"	21	91
S_2	45' S	"	17	73
W_1	30' W	E-W	13	130*
W_2	50' W	"	3	30
E_1	58' E	"	3	30
E_2	100' E	"	1	10

*See Text.

Finally, the absolute values of the horizontal and vertical displacements of the basement floor, and of the ground at the different distances from the building, are given in Table 6.2, together with the ratio of the horizontal to the vertical displacement at each location. The two displacement values for each location were recorded simultaneously.

Table 6.2

Absolute Values of the Horizontal (HD) and Vertical Displacement (VD)

Location (See Fig. 6.1)	Distance From Building	Test Direction	HD ($\times 10^{-3}$ in)	VD ($\times 10^{-3}$ in)	HD/VD Ratio
Basement Floor		N-S	0.820	0.074	11
N_2	60' N	"	0.390	0.029	13
S_1	30' S	"	0.750	0.037	20
Basement Floor		E-W	0.388	0.119	3.5
W_2	50' W	"	0.126	0.087	1.45
E_1*	58' E	"	0.126	0.130	0.97
E_2	100' E	"	0.038	0.055	0.71

*Over concrete slab.

Interpretation of the Results

1) The results of the N-S direction tests are more consistent than those of the E-W direction. The explanation of this may be that the accelerometers were placed over undisturbed soil in the former tests, but not in the later. The actual placing of the accelerometer was as follows: On the East side, the accelerometers were placed on a concrete slab at E_1 (see Fig. 6.1), and at the edge of the memorial pool excavation, 34' N of the E-W center line, at E_2. On the West side, some of the ground had been compacted by the passage of heavy trucks, and for this reason it was not possible to get any signal at a

point 60' W from the building. At 50' W (location W_2 in Fig. 6.1), the horizontal component of the recorded acceleration was very small with respect to the horizontal acceleration recorded 20' closer to the building at W_1, where the accelerometers were placed over 2 feet of the original fill soil, which had not been disturbed.

The fact that there was no signal recorded on the soil that had been compacted indicated that there was very little transmission of vibration from undisturbed soil to more rigid compacted soil.

2) The fact that the horizontal displacement of the ground recorded at W_1 was larger than that of the basement floor (see the value marked with the asterisk in Table 6.1), seems to indicate that at least part of the energy was being transferred from the building into the ground through the basement wall, but it is reasonable to assume that most of the energy was transferred through the foundation into the ground beneath it, which had been consolidated by the weight of the building.

3) From the record it was seen that the ground on which the accelerometers were placed (see Fig. 6.1), was vibrating in phase with the building, so that the half wave length was more than 100 feet. This is consistent with the value of the frequencies of excitation (2 c.p.s. in the N-S direction and 1.5 c.p.s. in the E-W direction).

4) A comprehensive treatment of the case of wave radiation from a vibrating source is given by Barkan (see Ref. 28). This includes a thorough theoretical analysis, with experimental verification, for the case of the vertical oscillation of a uniformly loaded circular rigid foundation block, 1 square meter in area, resting on

a semi-infinite isotropic elastic body. The exciting force was varied between 16.6 and 20 c.p.s. The experimental data for the vertical oscillations agree with the theoretical prediction, after the energy absorption of the soil is taken into consideration, but the theory of surface waves proposed does not adequately predict the amplitude of the horizontal displacement at a given distance from the vertically vibrating source.

The difficulties in the analysis of the vibration of the foundation of a building may be seen by referring to the Millikan Building tests: The foundation area was bigger and its shape more complicated (see Fig. 1.2) than the cases tested by Barkan; horizontal and rocking vibration occurred simultaneously, so the load was not uniformly distributed; the soil, as shown in Figs. AVI-1 and AIV-2, is not an isotropic elastic body; and the frequency of excitation was low (2 to 1.5 c.p.s.). Thus in the building-ground vibration problem further experimental and theorctical work is needed.

VII. ADDITIONAL TESTS

A. Measurements With the Lunar Seismometer

When a building is under construction, the time available to perform a steady state vibration test is limited by practical considerations. The vibration generators remained installed at the Millikan Library for about 6 weeks.

Before the vibration generators were installed, and after they were taken away, the natural period of vibration and the damping of the building for the lowest translational mode in the N-S and E-W directions were measured using the Lunar Seismometer (see Appendix II), exciting the structure with a rhythmical movement of the operator's body. This type of vibration is called "man excited vibration."[35] These measurements were used to allow completion of the study of the influence of the precast concrete wall panels (which are typical of those being installed in many tall buildings in Southern California) on the dynamical response of the building, and also to measure the natural period of vibration and the damping of the building at very low force levels. Comparison of these results with those from the steady state tests made at higher force levels gave some additional information about the nonlinear characteristics of the damping, and the stiffness of the building. Comparing the periods measured at the two different force levels also should help to relate the many building periods which have been measured at very low force levels[36,37,38,39,40] (using such exciting sources as wind, vehicular traffic, movement of the elevators, etc.), to periods appropriate to higher levels of exciting force.

Test Results

1) The first period measurements with the Lunar Seismometer were made shortly before the precast concrete wall panels were placed. The natural period measured for the lowest mode in the N-S direction was 0.46 sec; in the E-W direction it was 0.71 sec.

2) For a more complete check on the influence of the precast concrete facade on the response of the building three accelerometers were placed on the 9th floor as shown in Fig. 7a-1, allowing measurements to be made simultaneously at locations 1, 2 and 3. After the North side facade was placed, steady state tests were performed using only one vibration generator at a time, at the lowest possible force level. In the N-S direction the shaker located at A (Fig. 1.1) was used at force level O_1, and the measured period was 0.49 sec. In the E-W direction the exciter located at B was used, again with force level O_1, and this gave a period of 0.67 sec. The response curves for the 3 locations on the 9th floor, for the steady state testing in the E-W direction, are given in Fig. 7a-1. Note in that figure that at the North side the acceleration has been greatly reduced, because of the increase in stiffness resulting from the installation of the precast concrete panels on that side, and hence the center of torsion has moved northward.

After the South facade was placed, the measured period changed to 0.505 sec in the N-S direction, but remained 0.67 sec in the E-W direction. The center of torsion was once more over the E-W center line.

3) Three months after the vibration generators were re-

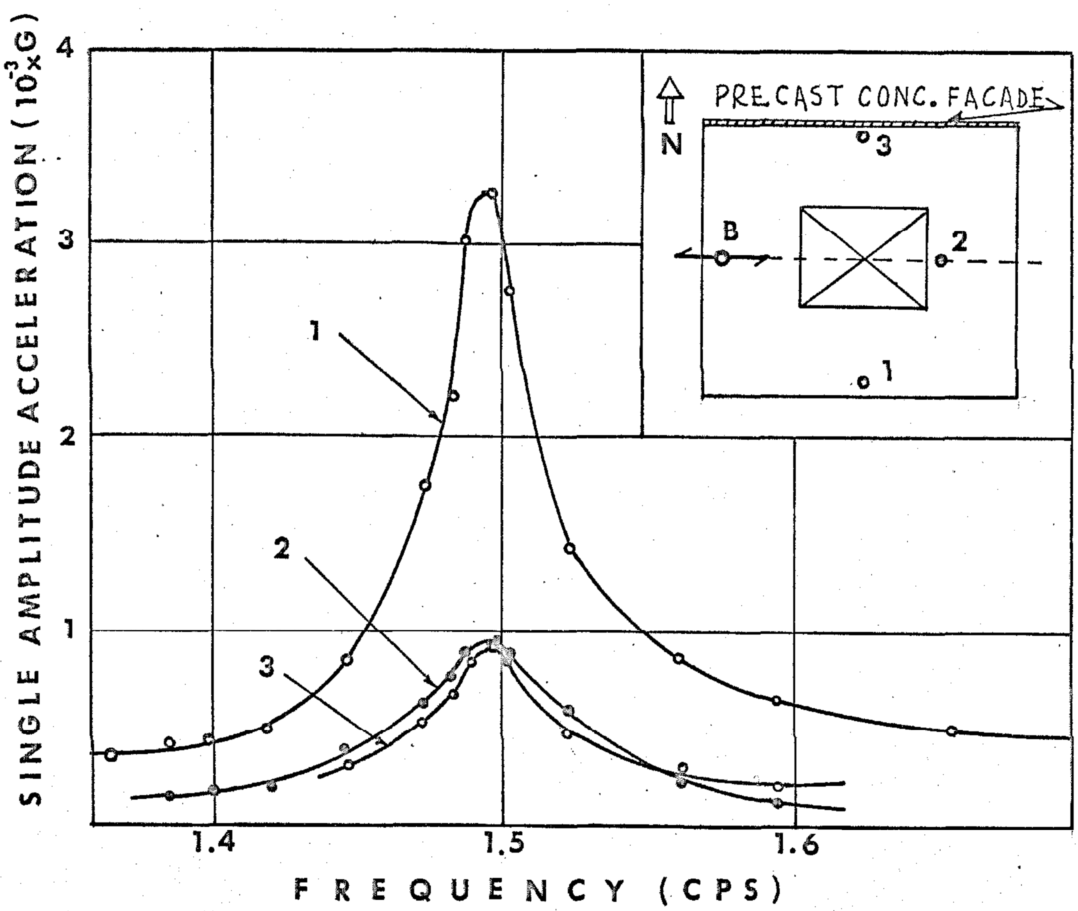


Fig. 7a-1 INFLUENCE OF THE NORTH SIDE WINDOW PANELS ON THE RESPONSE OF THE 9th FLOOR IN THE LOWEST TRANSLATIONAL MODE, E-W DIRECTION moved, the Lunar Seismometer was used a second time to measure the natural periods of the building. The result was 0.52 sec in the N-S direction, and 0.64 sec in the E-W direction. During those three months only finishing work had been done on the building.

4) The final use of the Lunar Seismometer was made about 6 weeks later, when the N-S and E-W first natural periods were measured, with different levels of man excitation. The magnification factor of the Lunar Seismometer was not altered during these tests, which began with small excitation forces, and increased step by step until the last test was made at the maximum capability of the operator. There was no way to measure the absolute value of exciting forces, but relative values were found by comparing the amplitudes of the recorded motion.

In the N-S direction, for increasing exciting forces, the periods measured were: 0.507, 0.515, 0.519 and 0.522 sec. In the E-W direction the period measured varied only between 0.642 and 0.644 sec.

5) It was noted that after having some practice in using the Lunar Seismometer it was possible to maintain the man excitation at a fairly constant force level, so it was possible to perform a free vibration decay test and then compute a damping value from the records.

To determine the damping, the test was made as follows: The building was exciting at resonance at the maximum capability of the operator. After the motion became steady state it was kept at this level for about 12 seconds, then the body movement was suddenly

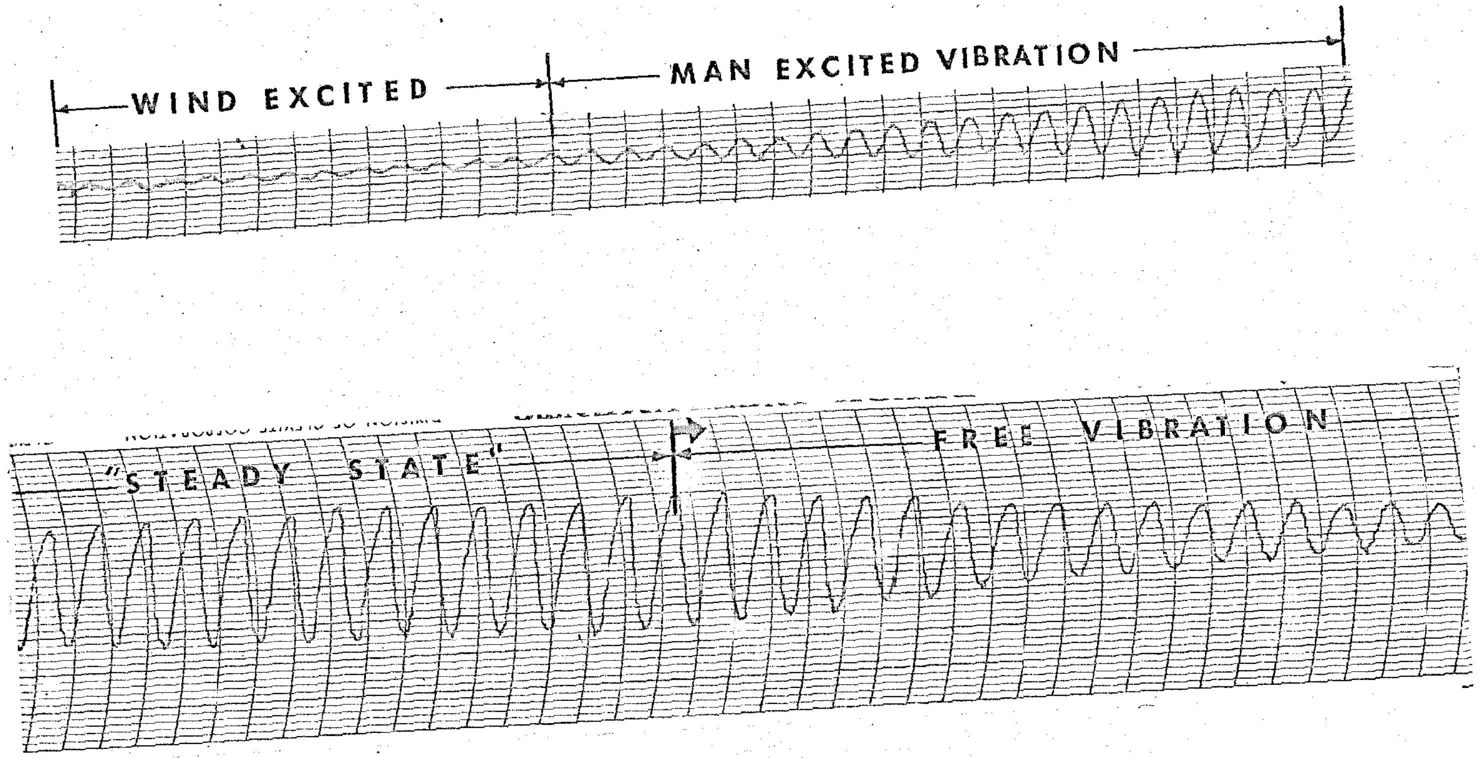


Fig. 7a-2. RECORDS TAKEN WITH THE LUNAR SEISMOMETER.

stopped, letting the building vibrate freely. A record from one of these tests is shown in Fig. 7a.2.

The percentage of critical viscous damping, computed from curves similar to Fig. 7a.2, varied from 0.6 per cent to 0.9 per cent in the N-S direction, and 0.6 per cent to 0.8 per cent in the E-W direction.

Interpretation and Comments on the Test Results

1) The effect of the precast concrete wall panels placed on the North and South sides was to increase the natural period of vibration in the N-S direction, and decrease the period in the E-W direction, i.e. in the N-S direction it was the increase in mass that affected the period, but in the E-W direction the increased stiffness in that direction predominated.

2) Placing the North side facade caused the natural period in the E-W direction to decrease from 0.71 sec to 0.67, but it was unchanged after the South side facade was placed. It appears that the North side facade was very effective in increasing the stiffness in the E-W direction, but the relatively small additional increase in stiffness due to the installation of the South facade was compensated for by the increase in the mass, so the period of vibration did not change.

3) The change in the N-S period from 0.505 to 0.52 sec, and in the E-W period from 0.67 to 0.64 sec, during the 3 months following the completion of the main tests, was due to finishing work performed in the building. This included the placing of granite plates on the shear walls running N-S; the covering of the joints between

the precast concrete panels and the structural elements of the building by beam-like elements (see Fig. AV-11) which run in the E-W direction; and the fitting of the glass in the windows. The increase of the period in the N-S direction indicates that the granite plates did not increase the N-S stiffness of the shear walls, but the increase in the E-W period shows that the finishing work done on the elements running E-W significantly increased their stiffness in that direction.

4) The percentage of critical viscous damping computed from free vibration decay tests using the Lunar Seismometer (0.6 per cent to 0.9 per cent) is consistent with the results computed from the steady state tests, and also with the results computed from a free vibration decay test carried out by exciting the air handling unit of the air conditioning system by hand, and recording the decay of the free vibrations of the building with two accelerometers, one on the roof, and one on the first floor. This test gave a damping value of 0.78 per cent of critical viscous damping.

B. Vibration of the Equipment Located on the Roof

The usual practice with buildings of intermediate height is to locate the elevator and air conditioning equipment on the roof. If a piece of this equipment has a natural frequency close to one of the lower natural frequencies of the structure, it may act as a vibration amplifier and be violently shaken during either an earthquake or a vibration test of the structure.

The dynamic response of equipment located on the roof to a given disturbance should be studied for two reasons. First, to find the response of the equipment itself, so it can be properly protected, and second, to calculate the inertial forces developed by the equipment, so these can be added to the inertial forces developed by the main structure. J. Penzien and A. Chopra[41] have studied the earthquake response of an appendage on a multistory building. They simplified the problem by assuming that for each normal mode of vibration of the structure a two degree of freedom system could be used as a simplified model of the structure plus appendage. These models predicted quite well the maximum response of an appendage attached to the roof of a multistory building, for the whole range of ratios of building period to appendage period that is of practical interest.

During the vibration test of the Millikan Library in the N-S direction, it was noticed that the vibration of the air handling unit was much greater than that of the other pieces of air conditioning equipment located on the roof. The vibration amplification was measured by placing one horizontal accelerometer on the roof, and another on the top of the air handling unit, as shown in Fig. 7b-1. Pictures of the air handling unit, and details of its connection to the supporting concrete blocks may be seen in Appendix V.

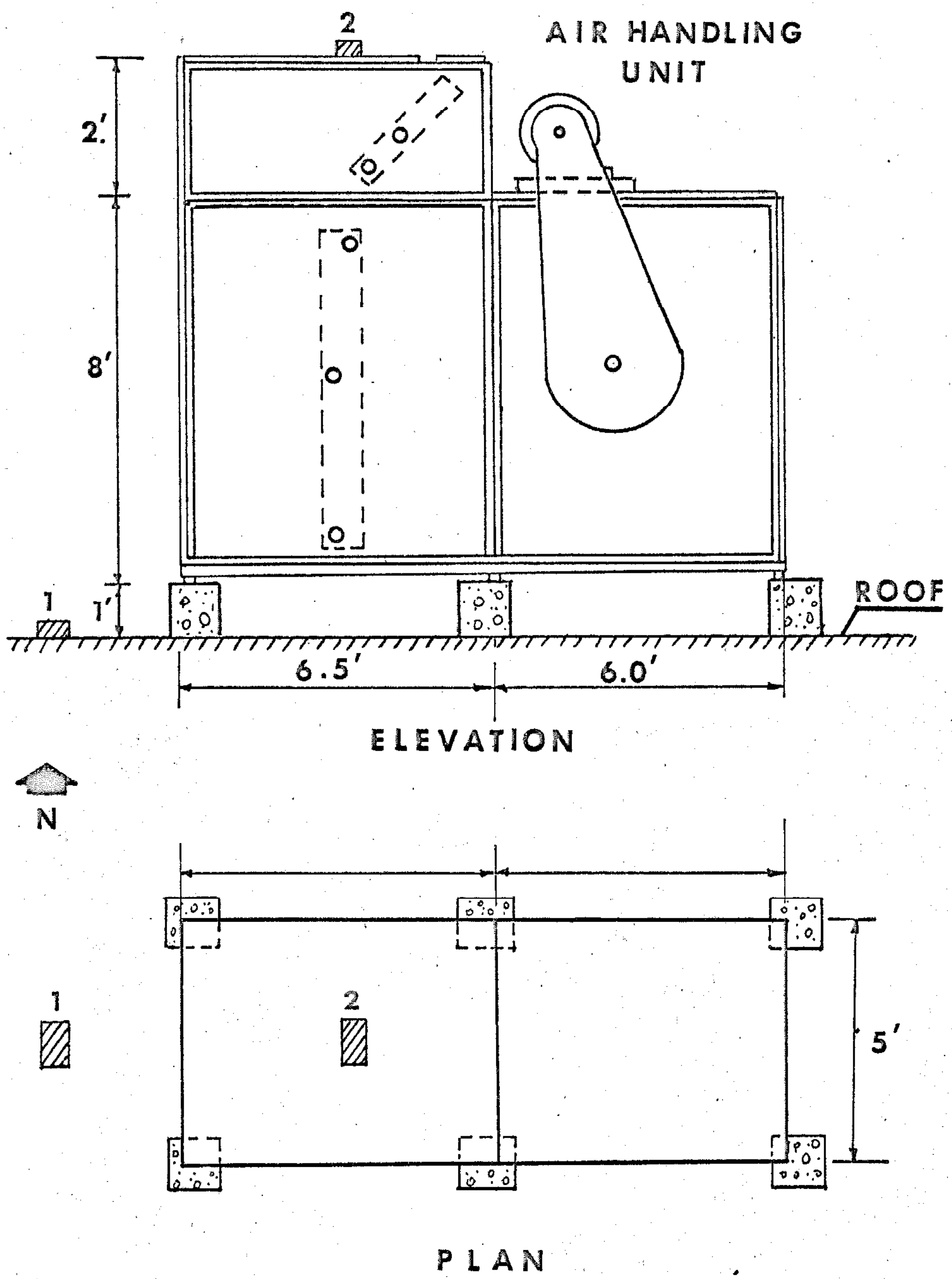


Fig. 7b-1 PLAN AND ELEVATION VIEWS OF THE AIR HANDLING UNIT, SHOWING LOCATIONS OF THE ACCELEROMETERS

Test Results

Because of the intensity with which the air handling unit was being shaken, it was not surprising to find that the acceleration measured on the top of it was about 8.5 times the acceleration of the roof. The values of the accelerations recorded were:

On the roof (location 1 Fig. 7b-1)	1.58 per cent g
On the top of the air handling unit (location 2 in Fig. 7b-1)	12.30 per cent g

VIII. GENERAL CONCLUSIONS

The following conclusions can be drawn from the results of the dynamical test:

1) The building has the nonlinear stiffness properties of a "softening system," since the natural periods of vibration increased as the force level was raised. However the observed nonlinearity was small for the range of the exciting forces applied to the building: in the N-S direction, increasing the exciting force by 3.8 times increased the period 2.9 per cent, in the E-W direction increasing the force 7.5 times increased the period 2.72 per cent; and in torsion increasing the force 2.1 times increased the period by 1.2 per cent.

2) The periods measured for this 9 story reinforced concrete building were shorter than those measured for other buildings in California of the same height, indicating that it has a relatively high rigidity. The ratio of periods for the two lowest modes indicates that deflection of the building due to bending was important during the tests, and thus the theory developed for shear buildings would not apply to the analysis of the Millikan Library.

3) The shapes of the first modes in the N-S and E-W directions, and in torsion, remained unchanged with variation in the frequency of excitation, and with increases in the level of the exciting force, with two exceptions:

a) The mode shape was not well defined for "small" exciting forces; and,

b) The 1^{st} floor and basement components of the normalized mode shape in the N-S tests, and those of the basement in the torsional tests, consistently increased with increases in the exciting force. The fact that this phenomenon took place in the N-S direction and in torsion, but not in the E-W direction, suggest that it occurred because of the relatively high rigidity of the resisting elements in the N-S direction.

4) The damping value consistently increased with increases in the level of the exciting force. This is clear from the following results: In the N-S direction, when the force at resonance changed from 1448 to 5430 lbs, the damping value computed by the standard method changed from 1.16 per cent to 1.64 per cent of the critical viscous damping. In the E-W direction, forces at resonances of 434 and 3290 lbs gave damping values of 0.69 and 1.48 per cent respectively, and in torsion, forces of 1535 and 3525 lbs gave damping values of 0.93 and 1.05 per cent respectively.

5) The damping values may be computed from data recorded at any floor of the building without changing the results.

6) The assumption that each floor system was moving as a rigid body during the vibration tests was correct, in spite of the relatively high rigidity of the shear walls oriented in the N-S direction. During the tests it also became clear that the synchronization of the vibration generators was very precise, and that the recording system was working correctly.

7) It was found that it is more accurate to assume that a building of this general type is fixed at the foundation, rather than at ground level as is usually assumed in the seismic analysis

of buildings. During the tests the part of the building below ground level behaved much as if no back fill were present at all, in spite of the fact compaction up to 90 to 100 per cent of the maximum soil density was carried out.

8) Period measurements made at very low force levels gave results relatively close to those measured from steady state tests at higher force levels, and the difference between them may be neglected for practical purposes. The difference may be further reduced if the man excited vibration tests are made at the maximum capability of the operator.

9) It was possible to perform free vibration decay tests, after man excitation of the building, but the damping value computed at that force level was only about 1/2 to 1/3 of the value computed from steady state tests at the highest force level. This confirms the non-linear characteristics of the damping.

Recommendations

It has been shown in the conclusions that both the natural period of vibration and the damping of the building (the 2 most significant parameters affecting the dynamical characteristics of the structure) increased with increases in the exciting forces. Up to 1967, no systematic test has been performed on a reinforced concrete building to find how these two parameters vary with changes in the level of the exciting force up to stress intensities comparable to those that a strong earthquake would cause in a building, because structures tested have been relatively large, and the inertia forces generated by each vibration generator have been limited to 5000 lbs for practical reasons. However the vibration generators have a very precise control, and excellent stability, up to about 8 c.p.s., which permits a precise measurement of the motion. Also the force level can be increased in small steps, by appropriate weight combinations. Thus tests performed with these shakers, in buildings of appropriate size and shape, could provide valuable additional data.

Specifically, it is suggested that tests be performed in specially designed building models (in which structural damage would be permitted, which is not tolerable in full size buildings for obvious reasons) with the following characteristics:

a) The structure should be very simple to analyze, and should have well separated natural frequencies.

b) It should initially be fixed to a large concrete foundation, so that the soil influence would be virtually eliminated. After the response of the superstructure is well understood, a study of the effects of the soil properties on the structural vibrations should be undertaken.

c) The model structure should be small enough so that the vibration generator could produce in it stresses of similar intensity to those that a strong earthquake would produce in a real building, and big enough so that the reinforced concrete structural members and joint conditions of the real building could be reproduced.

d) Because it is reasonable to assume that a shear type building would have more damping than a bending type building of similar size and shape, it would be interesting to test both a shear type building and a bending type building model.

APPENDIX I

Program of Millikan Library Vibration Tests

Table AI-1

Program of the Vibration Tests

Test No.	Weight Combination	Test Description	Date Performed	Comments**
0	*	Preliminary test	Nov. 8-9, 1966	Lunar seismometer
1	O_1	Identification of freq., N-S direction	Nov. 11, 1966	West side exciter only
2	O_1	Identification of freq., E-W direction	Nov. 12, 1966	East side exciter only
3	F_1	Check of rigidity of floor systems	Nov. 19, 1966	"
4	F_1	Determination of center of torsion	Nov. 19, 1966	Both exciters 180° out of phase
5a	F_1	Translational mode, N-S direction	Nov. 20, 1966	
5b	F_2	"	"	
5c	F_3	"	"	
6	F_4	Basement motion, N-S direction	Nov. 26, 1966	
7a	"	Ground vibration, North side	"	
7b	"	Ground vibration, South side	"	
8a	"	Translational mode, N-S direction	"	
8b	F_5	"	Nov. 27, 1966	
8c	F_1	"	"	

Table AI-1 (continued)

Test No.	Weight Combination	Test Description	Date Performed	Comments**
8d	O_1	Translational mode, N-S direction	Nov. 27, 1966	Second attempt to determine the 2nd mode
9a	F_1	Torsional mode	"	East side exciter only
9b	F_2	"	"	"
9c	F_3	"	"	"
10a	O_1	Translational mode, E-W direction	Dec. 3, 1966	2nd mode
10b	O_2	"	Dec. 4, 1966	1st and 2nd mode
10c	F_1	"	"	
10d	F_2	"	"	
10e	F_3	"	Dec. 3, 1966	
10f	F_4	"	"	
10g	F_5	"	"	
11a	F_4	Basement Motion, E-W direction	"	
11b	"	Ground Vibration, West side	Dec. 10, 1966	
11c	"	Ground Vibration, East side	"	

Table AI-1 (Concluded)

Test No.	Weight Combination	Test Description	Date Performed	Comments**
11d	O_1	Ground Vibration, East side	Dec. 10, 1966	Second mode
12	O_1	Check of coupling translational E-W & torsional mode	"	"
13	*	Period measurements	March 17, 1967	Lunar Seismometer
14	*	Period and damping measurements	April 27, 1967	"

*Man and wind excitated vibration.

**When there are no comments indicated, this means that both vibration generators have been used, exciting the building in the first mode in the direction indicated by the test description.

Table AI-2

Lead Weight Combinations* Selected for the Tests

Weight Combination	East Side Unit (Counterbalanced)	West Side Unit (Not Counterbalanced)	Unbalanced Moments Both Units	Maximum Frequency (cps)
O_1	S1 (455)**	Empty (427)	(982)	9.1
O_2	S2 (909)	S1 (947)	(1856)	7.2
F_1	S4 (1818)	S3 (1801)	(3619)	5.2
F_2	L2 + S1 (3284)	L2 (3350)	(6634)	3.8
F_3	L3 (4225)	S2 + L2 (4204)	(8429)	3.5
F_4	L4 (5660)	L3 + S2 (5619)	(11,279)	3.0
F_5	S4 + L4 (7478)	S3 + L4 (7461)	(14,939)	2.6

*Detailed explanation may be found in reference 1.

**Quantities inside parenthesis are unbalanced moments in lb-in.

APPENDIX II

Instrumentation

Introduction

The choice of instrumentation was greatly facilitated by the experience gained from experimental tests carried out by the Caltech Earthquake Engineering Research Laboratory from 1961 to 1966. Various reports describe in detail most of the instruments used in the present tests. The characteristics of the vibration generators, as well as their installation and operation were reported by D. E. Hudson[1]. The characteristics of the recording system, used in a vibration test of a reservoir outlet structure, and forced vibration tests of an earth dam, were described in detail by W. O. Keightley[3]. This report included also the manufacturers and prices of the instruments at the time the tests reported by him were performed (1962).

Description of the Instruments Used in the Tests of the Millikan Library Building

A brief description of the instruments used in the tests is included below. They are grouped as follows: a) Exciting system, (b) Recording system and c) Accessory instruments. The control and recording system set on the 9th floor of the Millikan Library is shown in Fig. AV-1.

a) Exciting System. The exciting system consists of the vibration generators and their portable speed control consoles.

Essentially, the vibration generators consist of two eccentric weights which are rotated about a common vertical shaft in opposite directions by a chain drive system (see Fig. AV-2), which is connected to a 1-1/2 horsepower D.C. motor through a timing belt.

The eccentric loads are so arranged that their inertia forces cancel each other in one direction and generate a rectilineal sinusoidally varying horizontal inertia force perpendicular to the direction of zero force. The inertia force may be increased to a maximum of 5000 pounds by adding lead weights to the baskets.

The inertia force generated by the vibration generators can be computed using the following formula:

$$\text{Inertia force (pounds)} = 0.102(\text{WR lb-in})(\text{n rev/sec})^2$$

Each vibration generator can be driven alone or synchronized with other units. A total of 4 units have been built and tested at the California Institute of Technology, under the direction of a Special Committee of the Earthquake Engineering Research Institute for the State of California, Department of Public Works, Division of Architecture. Two of those units were used in the present tests.

The vibration generators are driven and controlled from a portable speed control console. The electrical circuits were designed by Prof. T. K. Caughey and R. V. Powell and are shown in reference 1. The main feature of the speed control system is that it is very accurate and stable, more so than any other system developed up to 1967 for testing civil engineering structures.

b) Recording System. The main components of this system are: accelerometers, amplifiers, and the recording elements. At the beginning of the tests, 11 accelerometers were available; their characteristics are given in table AII-1.

Table AII-1

Accelerometers Used in the Tests

Accelerometer type	Quantity	Range	Maximum Voltage	Natural Frequency (cps)
Statham A5-2-350	4	± 2g	9	100
Statham A4-0.50-350	2	±0.50g	9	21
Statham A4-0.25-350	2	±0.25g	9	15
Statham A4-0.2-350	2	±0.20g	9	12

The ± 2g accelerometers were used on the 6 upper floors, and for determining the vertical accelerations. The ± 0.5g accelerometers were used on the lower floors, and the more sensitive ± 0.25g and ± 0.2g were used on the first floor, basement and for measuring the horizontal acceleration of the ground outside the building.

The signals from the accelerometers were amplified by 8 William Miller C-3 type amplifiers (see Fig. AV-3). The serial numbers corresponding to each channel were: 74(1), 195(2), 73(3), 71(4), 76(5), 196(6), 72(7) and 75(8). At any time, no more than six channels were used simultaneously, leaving the other two in reserve so that if any of the channels suffered a malfunction, it could be promptly replaced and the experiment maintained on schedule.

The recording was made with a Consolidated Electrodynamics Corporation Recording Oscillograph, on a 7" wide, light sensitive paper. The paper speed could be varied from 0.25 to 64 in/sec and the time marks could be recorded each second or 1/10 second.

c) Accessory Instruments. The instruments that completed the set were:

Tachometer Digital Counter. A tachometer connected to the drive motor shaft and a digital counter were used to measure the speed of the vibration generators at any instant. The tachometer and counter, at their usual setting generated a count of 300 per cycle per second.

Monitor. The output of an accelerometer placed on the 9^{th} floor was continuously monitored with a Sanborn Dual Channel Amplifier-Recorder, Model 321, by recording the acceleration with a hot-wire writing arm on heat sensitive Sanborn Permapaper recorder chart. The speed of the paper could be changed from 1 to 100 mm per second.

Mounting Blocks. 6 lb blocks on which the accelerometers were mounted were used for easy and fast installation of these instruments, see Fig. AV-4.

Calibration Table. This calibration device for the accelerometers consisted of a tilting surface mounted over an adjustable tripod with an attached level bubble (see Fig. AV-5). Starting the tilting surface from the horizontal position ("0" acceleration), by changing the inclination of the table, the component of the acceleration in the direction of the axis of the table may be set at 0.05, 0.10, 0.15, 0.20, ..., 0.60 or 0.65g.

All the tests were calibrated at 0.05g. For this value the ± 2g accelerometers gave a reading of about 50 chart lines with an amplifier attenuation of 36 decibels.

Voltmeter. A standard DC voltmeter was used for a precise synchronization of the vibration generators.

Lunar Seismometer. This instrument is a portable velocity sensitive seismometer, developed in connection with the space program, with a maximum magnification of approximately 10,000 at 4 cps and 1000 at 1 cps.

APPENDIX III

Material Properties

The dynamic characteristics of a structure depend on its geometry and its material properties. The Millikan Library is a reinforced concrete building, so this appendix is concerned with the properties of the concrete and its ingredients, and with the characteristics of the reinforcing steel.

If the structure responds linearly, the dynamic characteristics are directly related to the material properties, by the modulus of elasticity, but for nonlinear response, there is a more complicated dependence on the material properties. For a better judgement of the test results, and for further analysis of the building, the more significant properties of the materials are included herein.

Modulus of Elasticity of Concrete

The modulus of elasticity of concrete depends mainly upon its compressive strength, and to a lesser degree on the age, moisture content and type of aggregate. Tests made at the Research Laboratory of the Portland Cement Association in Chicago show that the values of the initial tangent modulus is nearly the same as the secant modulus at 1/3 of the ultimate strength.

Table AIII-1 taken from reference 42 shows how the initial modulus of elasticity is a function of the other parameters. More information about material properties can be found in references 43, 44 and 45.

Specifications for the Concrete and Steel Used in the Millikan Library Building

The minimum 28 day compressive strengths of the concrete for the structured elements are as follows:

A)	Beams, girders, slabs	4000 psi
B)	Walls	3000, 4000 & 5000 psi
C)	Columns	4000 & 5000 psi
D)	Footings	3000 psi

The quality control was made by taking 3 standard compression test cylinders for each 100 cubic yards of concrete, or for a fraction placed during a day.

The reinforcing steel bar (except for columns): Intermediate grade deformed bars conforming with ASTM A-15, with deformation conforming to ASTM A-305. Steel for columns: hard grade conforming with ASTM A-15.

Reports on Material Properties Tests

There are available complete sets of results of compression tests of concrete, tensile tests of steel, sieve analysis of the aggregate and cement tests, including the analysis of chemical components, which were performed throughout construction of the building. A summary of the results of the most significant tests are given in tables AIII-2 and AIII-3.

Table AIII-1

Modulus of Elasticity of Concrete as a Function of Compressive Strength, Age, Water Content, and Mix Type (Reference 42)

Mix By Volume	Cement cu yd	Net Water Cement Ratio	Strength (psi)						Initial Tangent of Elasticity (1000 psi)					
			3 ds	7 ds	28 ds	3 ms	1 year	5 yrs	3 ds	7 ds	28 ds	3 ms	1 year	5 yrs
1:2	10.5	0.57	3220	4650	5960	7810	9170	10160	2870	3340	3770	4460	5530	5750
1:3	7.8	0.64	2160	3250	5140	6500	7570	8820	2490	2930	3480	4200	5400	5650
1:4	6.3	0.76	1560	2620	4460	5710	6720	7750	2310	2660	3340	3740	5340	5580
1:5	5.2	0.85	1340	2120	3510	5130	6010	6860	2230	2590	3130	3610	5340	5570
1:8	3.4	1.13	480	945	1740	2550	3140	3930	1710	1990	2770	3420	4970	5530

Table AIII-2

Concrete Compression Test Results

Location in Structure	Specified 28 day Strength (psi)	7 day Strength (psi)	28 day Strength (psi)	Testing Dates
Footing	3000	2725	2765	1/19 & 2/9/66
Footing in core wall	5000	3450	5445	1/25 & 2/15/66
Basement wall	5000 5000	3500 3715	- 5855	2/24/66 2/23 & 3/16/66
1st floor core wall	5000	5040	-	3/25/66
1st floor E & W wall	5000	4225	-	3/29/66
1st floor columns	5000	4510	-	3/13/66
Beams & slabs	4000	3200	4550	Average
Stairs	4000	3130	4510	Typical
2nd & 3rd floor core wall	5000	3360	-	4/22/66
2nd floor columns	5000	3436	4235	5/3 & 5/26/66
3rd floor E, W & core wall	5000	3925	5325	6/22 7/13/66

Table AIII-2 (Continued)

Location in Structure	Specified 28 day Strength (psi)	7 day Strength (psi)	28 day Strength (psi)	Testing Dates
3rd floor columns	5000	3850	-	6/17/66
4th floor columns	5000	4190	5535	7/4 & 7/25/66
4th floor E, W & core wall	5000 5000	3340 3680	5535 5375	6/21 & 7/12/66 7/21 & 7/12/66
5th floor interior core	4000	3255	4350	7/7 & 7/28/66
5th floor E & W wall	4000	3220	4720	7/7 & 7/28/66
5th floor columns	5000	3980	-	7/6/66
6th floor core wall	4000	3340	5270	7/22 & 8/12/66
7th to 8th floor W wall	4000	3165	4490	8/5 & 8/26/66
7th to 8th floor E wall	4000	3150	4740	8/5 & 8/26/66
8th floor, E, W & core wall	4000	3200	4510	8/19 & 9/9/66

Table AIII-2 (Concluded)

Location in Structure	Specified 28 day Strength (psi)	7 day Strength (psi)	28 day Strength (psi)	Testing Dates
9th floor columns	4000	3220	4740	8/18 & 9/8/66
9th floor walls	4000	3220	4880	9/2 & 9/23/66
9th floor to roof columns	4000	2950	4370	9/20 & 10/11/66
Roof beam & slab	4000	3225	-	9/15/66

Table AIII-3

Reinforcing Steel Tests

ASTM A15 Grade	Bond ASTM A-305	Size Number	Actual Area (sq ft)	Yield Pt. (lbs)	Max. Load (lbs)	Yield Pt. (psi)	Tensile Strength (psi)	Elongation (per cent)
Interm.	OK	9	.970	45,000	79,200	46,400	81,650	22.0
Interm.	OK	8	.774	36,000	63,800	46,500	82,450	21.0
Interm.	OK	6	.440	21,200	36,500	48,200	82,950	21.0
Interm.	OK	5	.304	14,600	23,700	48,050	77,950	22.0
Interm.	OK	4	.200	10,300	16,300	51,500	81,500	16.0
Interm.	OK	3	.105	5,700	8,000	54,300	76,200	18.0
Hard	OK	9	.976	80,300	113,200	82,250	116,000	17.0
Hard or Interm.	OK	6	.453	25,100	38,900	55,400	85,850	16.0
Hard or Interm.	OK	4	.200	10,300	16,300	51,500	81,500	16.0

APPENDIX IV

Soil Properties at the Construction Site of the Millikan Library Building

The soil properties at the construction site were investigated by Converse Foundation Engineers of Pasadena, some years before the building construction was initiated. An abstract of the report of those studies is included in this appendix. The soils were examined by drilling 5 borings, as shown in Fig. AIV-1.

One to two feet of fill soil was found in the two West side borings, but no fill was noted in the others. Otherwise, all the drillings gave practically the same results. The soils encountered during the drilling were continuously recorded and the results of boring No. 2 are shown in Fig. AIV-2, where the soil properties at different depths are indicated. There is no mention of a ground water table.

The porous upper soils had moderate shear strength but were quite compressible when saturated. The firm underlying sand and sandy soils showed moderately high shear strengths with relatively low compressibility and they were not much affected by changes in moisture content.

The load-settlement characteristics of the soils were studied by performing consolidation tests on representative samples. Typical consolidation curves may be seen in Fig. AIV-3.

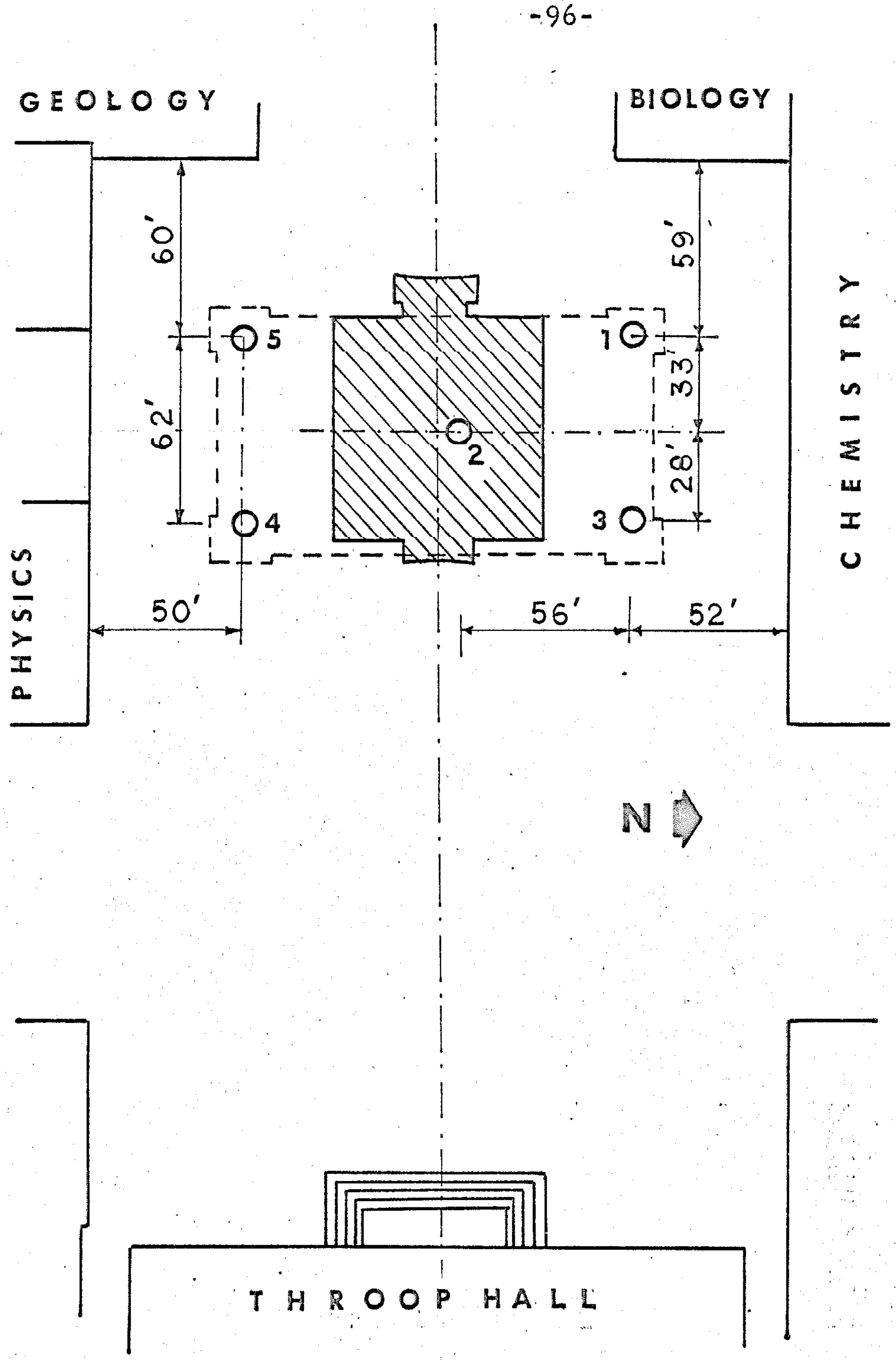


Fig. AIV-1. LOCATION OF THE BORINGS FOR THE STUDY OF THE SOIL PROPERTIES

Basement Excavation and Controlled Compaction Backfill

The basement excavation was made to such a depth that the foundation rested on firm undisturbed natural soil, about 18 feet below the ground level.

Tests and observations were made during the controlled compaction back fill, outside the basement walls and under the basement floor slab. Four types of soil were identified in this operation and their characteristics are given in table AIV-1.

Table AIV-1

Soil Type	Soil Description	Maximum Dry Density (lb/ft^3)	Optimum Moisture Content (per cent)
1	Brown silty sand and gravel	129.4	12.0
2	Brown silty sand	116.6	11.0
3	Brown fine to coarse sand and gravel	124.2	11.5
4	Gray brown fine to coarse sand	130.0	7.8

Tests made with samples taken during the compaction gave the following results: below the basement floor slab (soil type 1), the dry density averaged 120 lb/cu ft with soil compaction between 90-100 per cent of the maximum density, determined by the modified ASTM method, the field moisture content was 8.5 per cent of the dry weight. Outside the East exterior wall (soil type 4) the dry

Depth in Feet	Profile	Characteristics.				Code Classif.	Drive Energy ft.kips/ft	Field Moist. % Dry wt.	Dry Density lb/cu ft	Shear Resist Kips/sq-ft
5		Dry	Medium Soft	Dark Brown	Silty Sand + 10% Gravel to 2"	SM	1.7	4.7	98.0	0.31
				Brown	(Fine + 20% Gravel to 2" to + 30% Gravel to 3" Coarse) & occ. rock to 6"		0.7	4.0	100.1	0.35
							7.1	3.3	99.8	0.91
10			Medium		Sand fine + 40% Gr. to coarse + 20% Gr.	SW	1.8	2.6	105.6	0.88
			Loose M. Firm		Silty Sand	SM				
15			Dense		Sand fine to coarse + 30% Gr. to 3"	SW	8.2	3.9	113.2	0.69
					Sandy silt	ML				
			Firm	Light	Silty Sand 30% Gr. to 3"	SM	11.3	4.4	121.5	1.00
20, 25, 30			Dense	Brown	Sand fine to coarse + 40% Gravel to 3" & occ. rock to 6"	SW	12.0	2.3	112.3	1.13
							13.2	3.0	119.4	1.80
		Slig. Moist			9" Sandy Silt Streak		10.5	7.1	105.2	2.39
35		Moist	Firm	Brown	Alt. Merging 1"-3" streaks Sandy silt to Silty Sand.	ML & SM	9.7	7.7	101.0	2.64
			Dense	Light Brown	Sand + 30% Gr & Rock to 6" (Fine to Coarse) & occ. silty sand streaks.	SW & SM				
			Firm	Brown	Silty Sand	SM				
40					Sandy Silt	ML	5.9	2.2	105.6	3.04
45					Silty Sand Fine to Medium occ. Gravel + Alt. Streaks of Sand silt	SM & ML	15.0	9.4	113.8	3.66
50					+ Streaks of sand + 20% Gr.	& SW	17.3	11.2	99.5	3.18
			Dense	Light Brown	Sand Fine to Coarse + 30% Gravel	SW				
55				Brown	7" Silty sand streak		12.0	3.7	98.8	3.55

Fig. AIV-2. RESULTS OF SAMPLING OF BORING No. 2.

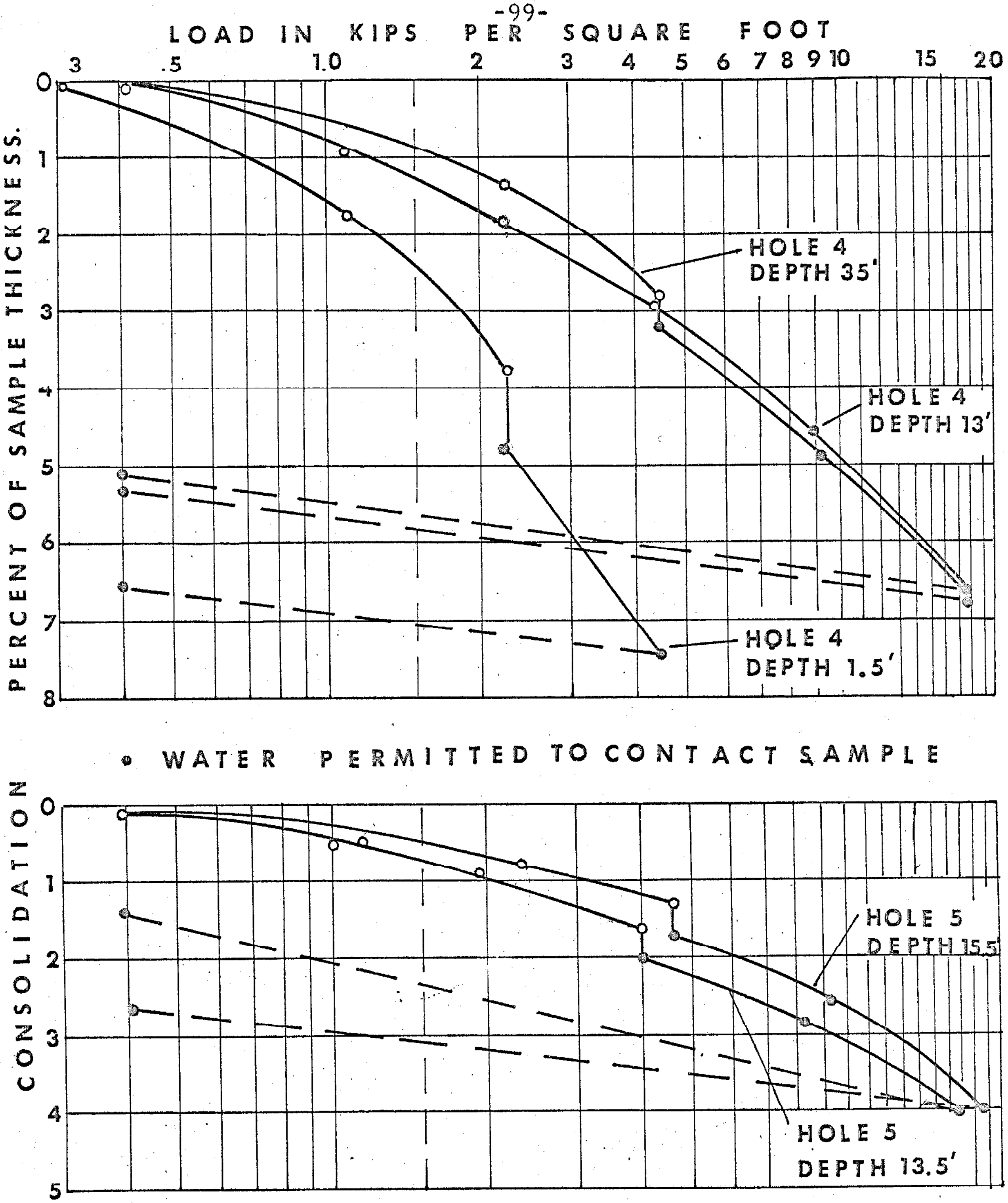


Fig. AIV-3. TYPICAL CONSOLIDATION CURVES.

density averaged 122 lb/cu ft with a soil compaction between 90-100 per cent. Tests made with samples taken at the North, West and South walls show results similar to those obtained at the East wall.

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APPENDIX V

PHOTOGRAPHIC MATERIAL

INSTRUMENTATION

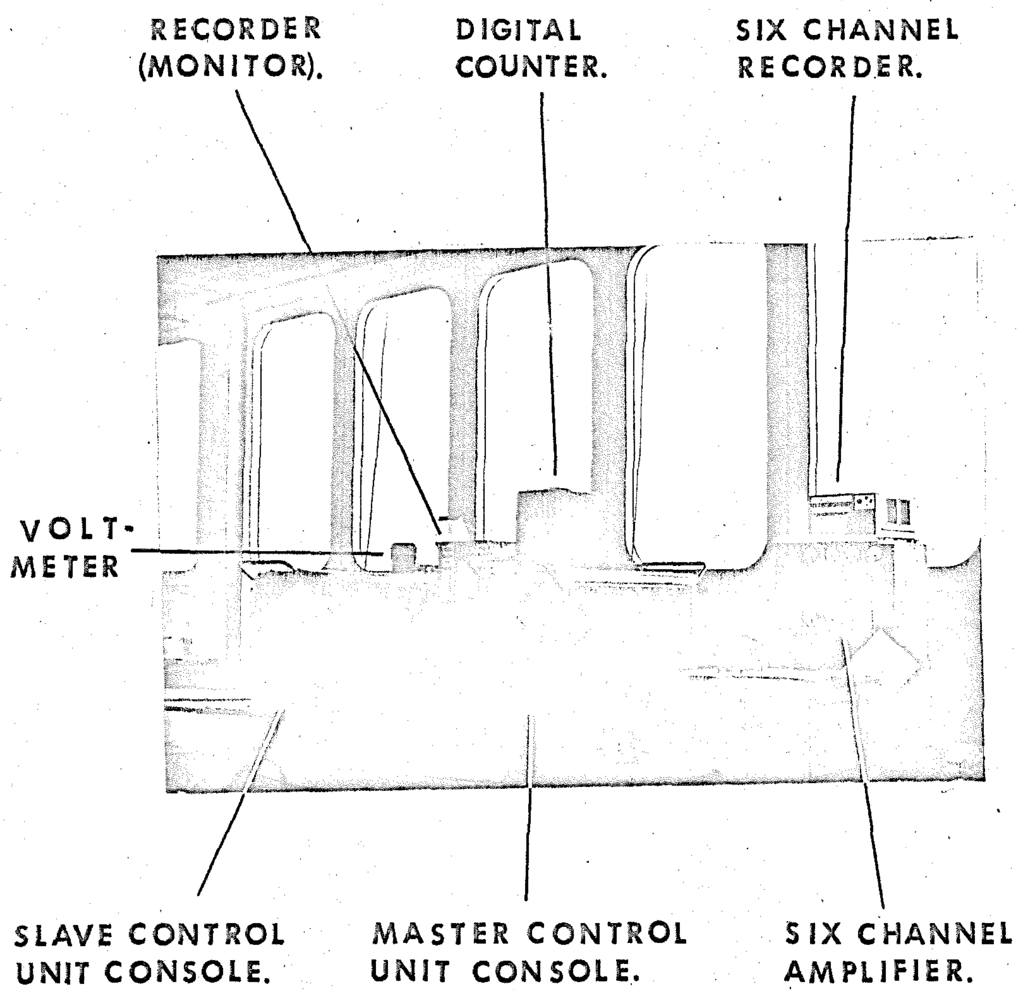


Fig. AV-1. SETUP OF THE CONTROL AND RECORDING SYSTEMS ON THE 9TH FLOOR OF THE MILLIKAN LIBRARY.

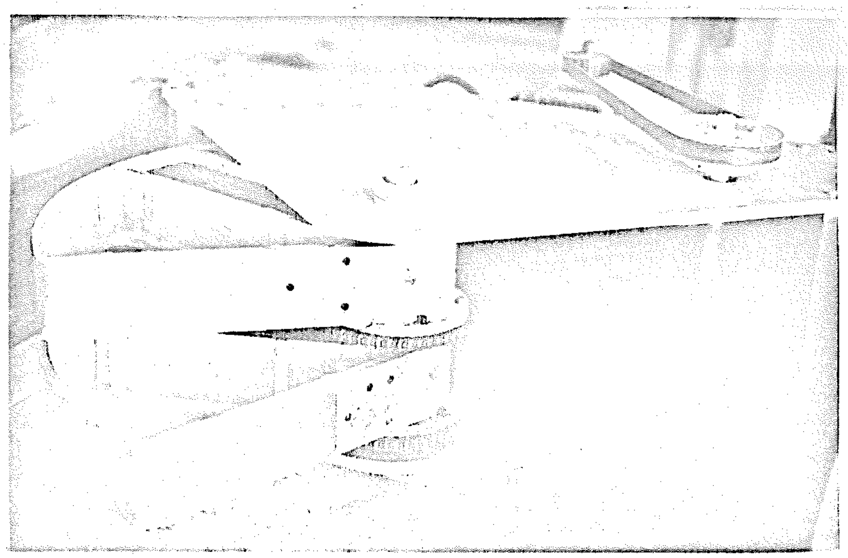


Fig. AV-2. VIBRATOR GENERATOR.

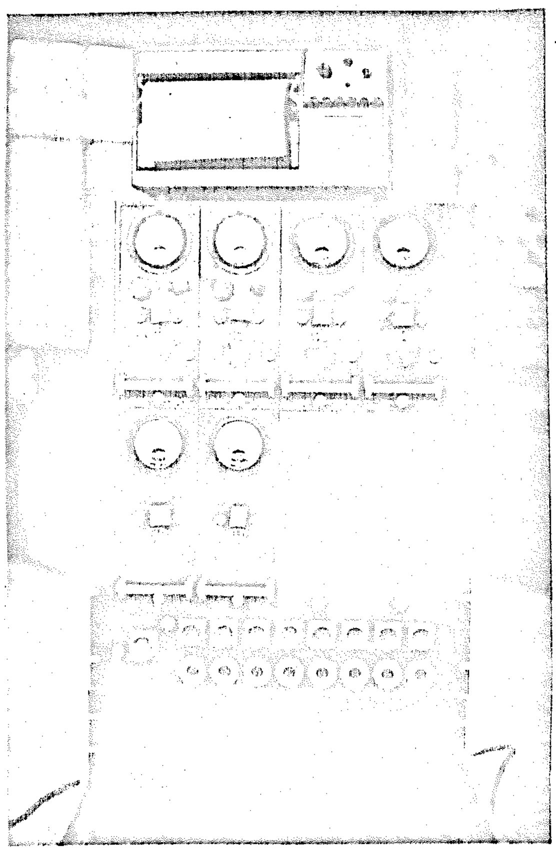


Fig. AV-3. CLOSE VIEW OF THE RECORDER AND AMPLIFIER.

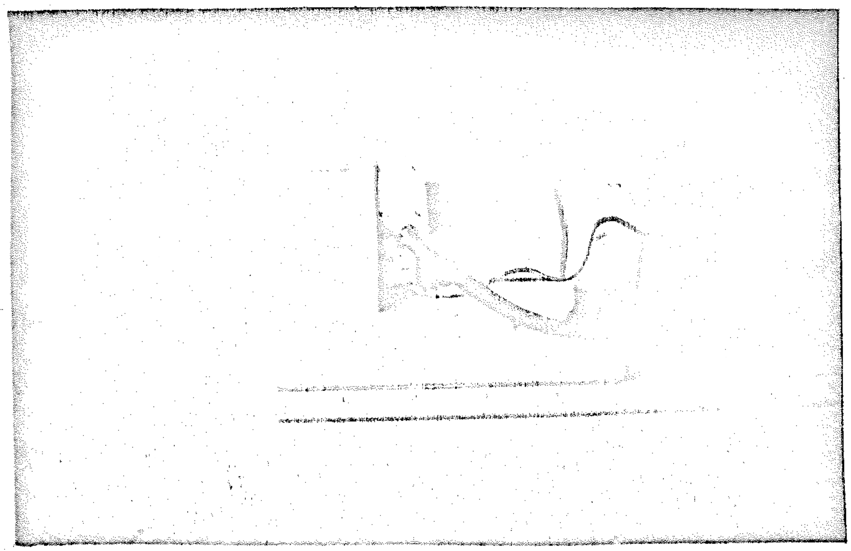


Fig. AV-4. ACCELEROMETER AND MOUNTING BLOCK.

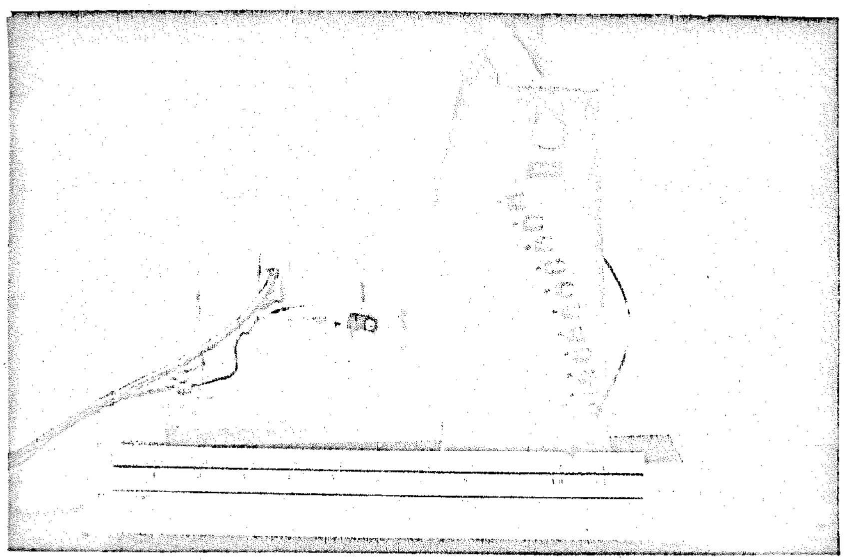


Fig. AV-5. ACCELEROMETER BLOCK ON THE
TILTING CALIBRATION TABLE.

THE BUILDING

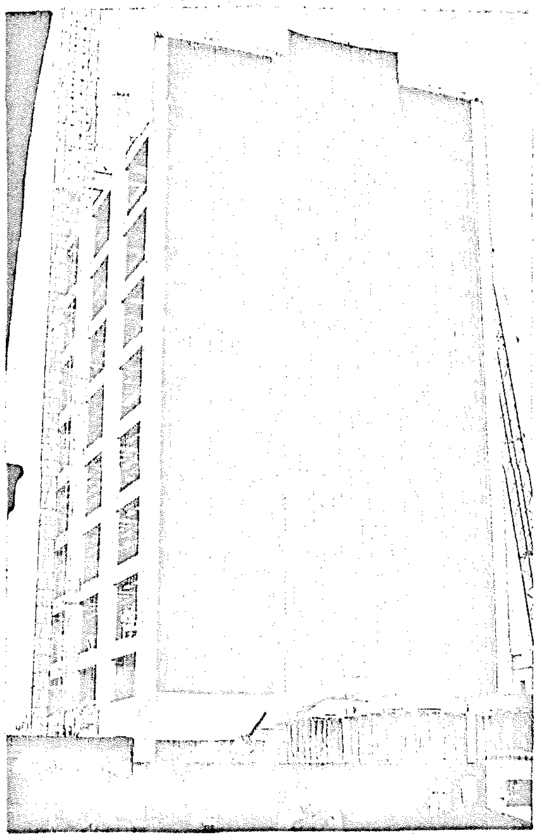


Fig. AV-6. GENERAL VIEW OF THE BUILDING LOOKING NORTH-WEST, BEFORE THE FACADE AND TOP BEAM WERE PLACED.

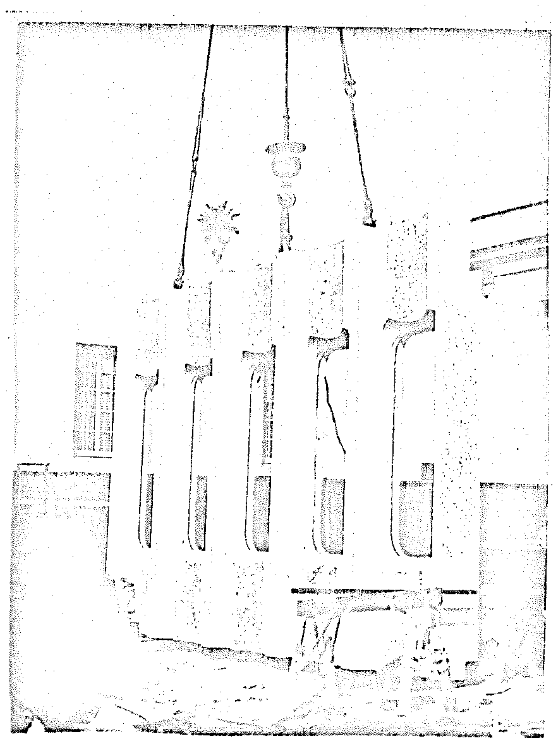


Fig. AV-7. ONE OF THE PRECAST CONCRETE WINDOW WALL PANELS.

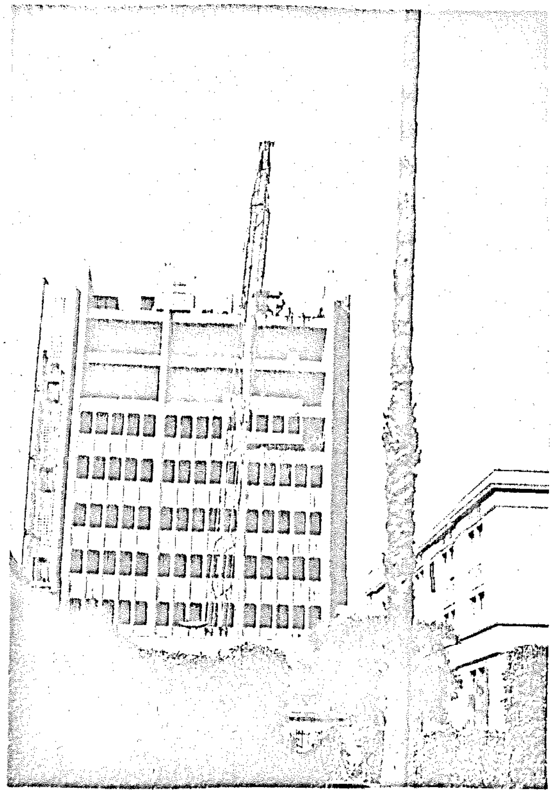
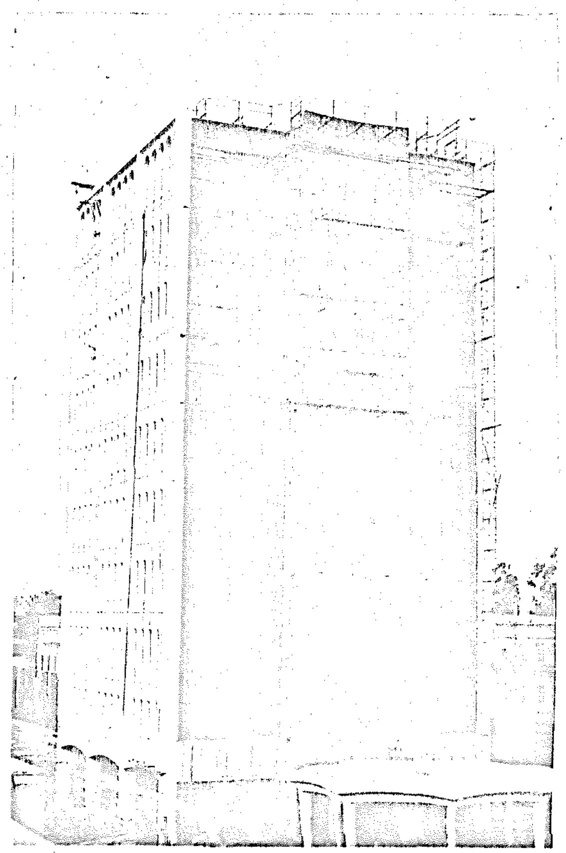


Fig. AV-8. CONCRETE PANEL BEING PLACED ON THE SOUTH SIDE.

Fig. AV-9. GENERAL VIEW OF THE BUILDING WITH THE PANELS INSTALLED.



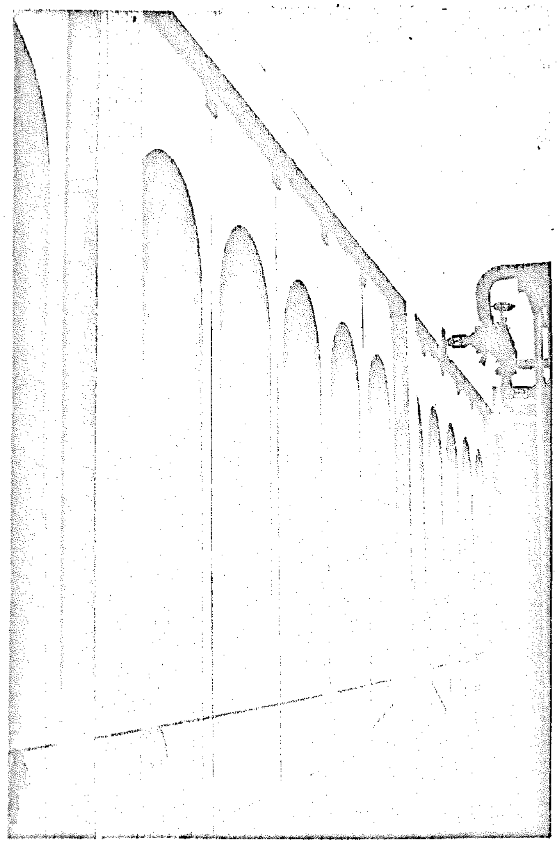


Fig. AV-10. NORTH SIDE ROOF PANEL, LOOKING FROM THE INSIDE OF THE BUILDING. NOTE THE STEEL CONNECTION BETWEEN THE TOP BEAM AND THE PRECAST WINDOW WALL PANELS.

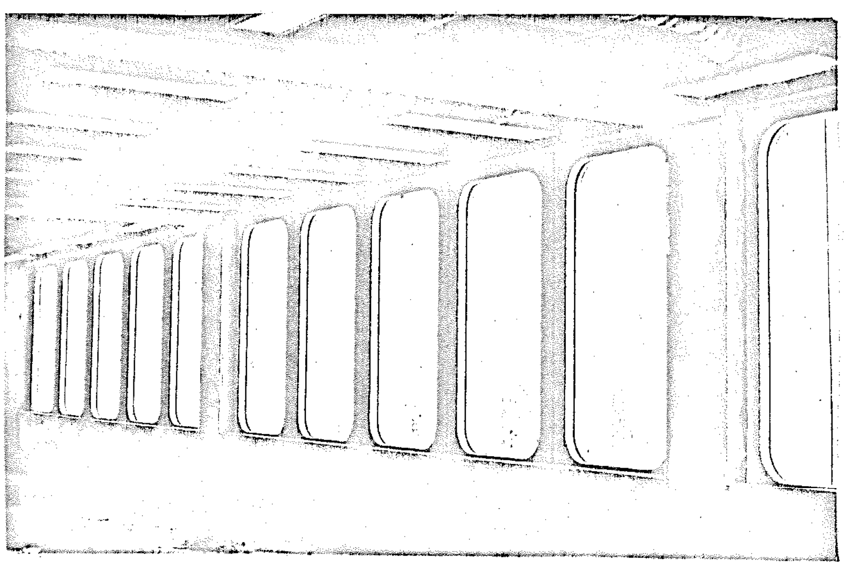


Fig. AV-11. INTERIOR VIEW OF A TYPICAL FLOOR. THE BEAM-LIKE ELEMENTS BELOW THE WINDOWS ARE NON STRUCTURAL.

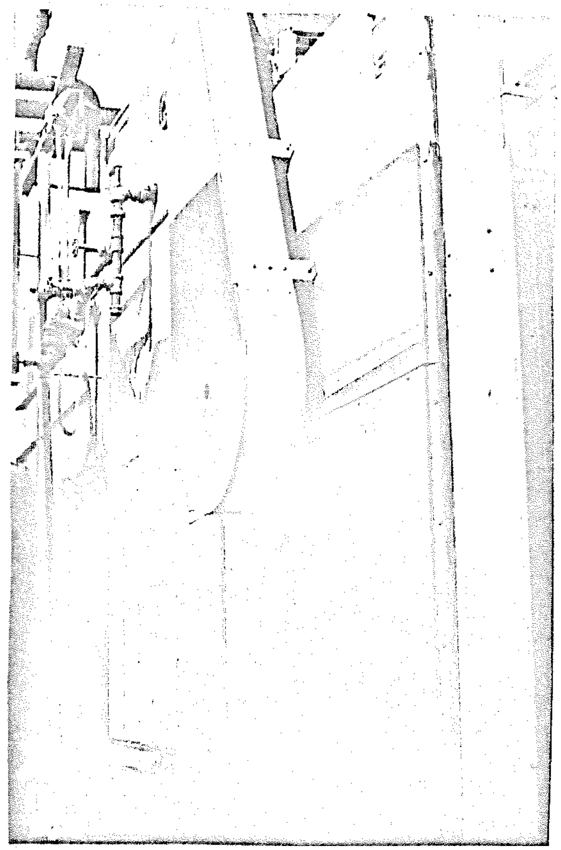


Fig. AV-12. SOUTH VIEW OF THE AIR HANDLING UNIT OF THE AIR CONDITIONING SYSTEM.

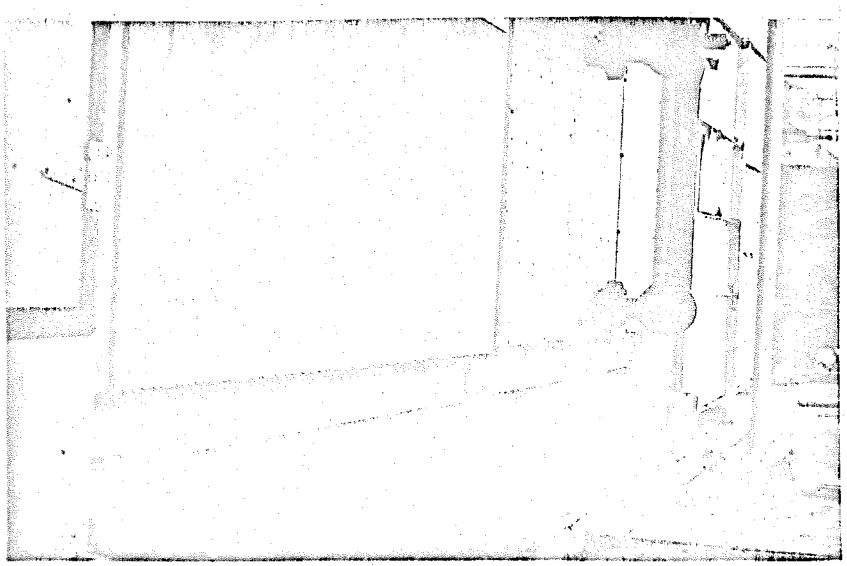


Fig. AV-13. DETAIL OF THE MOUNTING SYSTEM FOR THE AIR HANDLING UNIT.

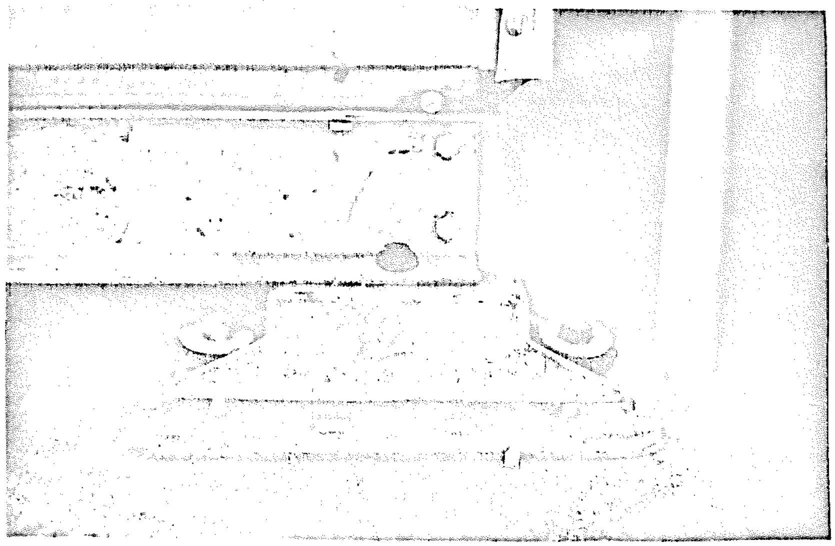


Fig. AV-14. CLOSE VIEW OF ONE OF THE ISOLATION MOUNTINGS.

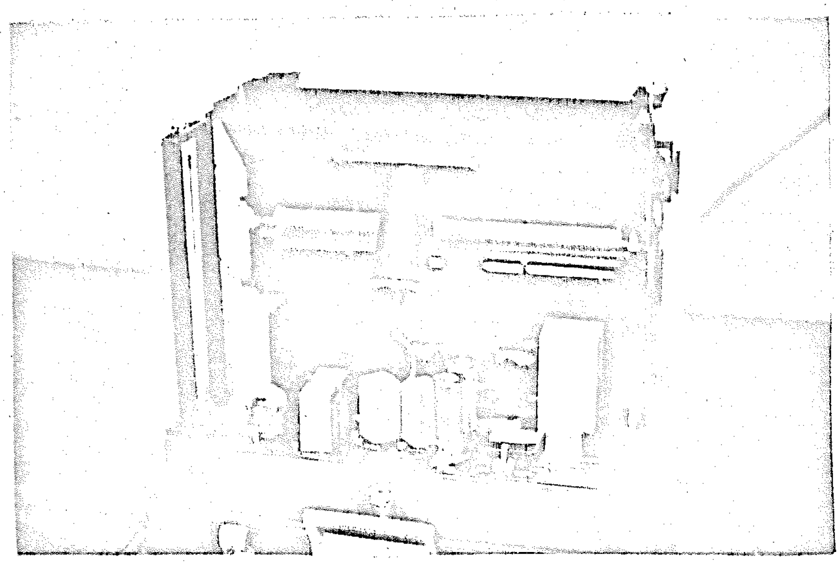


Fig. AV-15. STRONG MOTION ACCELEROGRAPH UED MODEL AR 240, TO BE INSTALLED ON THE ROOF AND IN THE BASEMENT OF THE MILLIKAN LIBRARY.