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R. S. PATTON, Director

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EARTHQUAKE INVESTIGATIONS IN CALIFORNIA 1934–1935

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PREFACE

The seismological work of the Coast and Geodetic Survey began in 1925 with the transfer of seismological work of the Government from the Weather Bureau, though prior to that time for 25 years seismographs had been operated at the magnetic observatories in connection with magnetic work. The early observations related to the collection and publication of information regarding United States and other earthquakes and the operation of teleseismic instruments for the recording of more or less distant earthquakes, especially in places where similar work was done by no other agency. Development of cooperative work has always been a feature of the program. Before many years the view developed that the program should be steadily developed in the direction of finding facts which would aid in the saving of life and property during strong earthquakes, as has been found necessary in other countries, notably Japan and Italy.

Congress provided additional funds in 1932 which made it possible to undertake greatly needed and hitherto lacking observations of strong earthquake motion, especially in regions where damage occurs. California was selected since even though other parts of the country have had more severe earthquakes, the conditions both as to type of earthquake, frequency, and probable immediate economic return from the investigation made it the most suitable place.

Existing instruments were inadequate for such investigations as their records failed because they were not adapted to the recording of destructive ground movements. New instruments had to be developed. This was done with the aid of various cooperating institutions, notably the National Bureau of Standards, the Massachusetts Institute of Technology, and the University of Virginia.

The obtaining of useful records of the Long Beach earthquake on March 10, 1933, gave new impetus to the program. It was seen that many allied observations had to be made to make the program of damage prevention effective. Drs. Robert A. Millikan and Arthur L. Day presented a program to the Public Works Administration and the program was approved subject to the work being under the supervision of an existing Government agency. The Coast and Geodetic Survey was selected as the proper agency. The detailed plans were worked out at a series of conferences in California of representatives of all the different interests, attended by the writer. The year's work which followed is the subject of the present volume.

Strong emphasis should be laid on the cooperative nature of the work. This is discussed in detail elsewhere, but it should be stated that if any of the many agencies had failed to do their part the result would have been far less effective. Many persons gave their time without recompense, and besides, useful occupation was provided to a considerable number of college graduates who were without other employment. Too much credit cannot be given to those who helped to make the work effective, some of whom are authors of chapters of this volume.

Mention should also be made of those outside of California who took part in an advisory capacity. At the Massachusetts Institute of Technology the results of laboratory vibration work on elevated tank models, by Mr. Arthur C. Ruge, were made available insofar as they affected the observational program in California. Results on actual tanks in California were coordinated, thereby doubling the value of each investigation. Dr. J. J. Creskoff made useful suggestions in connection with the preparation of reports and the collecting of miscellaneous information which might have a bearing on the observational data.

It should be emphasized that the other branches of the Federal Government have profited from this investigation. The information obtained has been used in the design of structures in California in which much Federal money is invested and also in such structures as Boulder Dam and the Madden Dam at Panama. Sites for Federal structures have been investigated.

It should be pointed out that the results in this volume represent not much more than a start. There are many as yet unsolved problems connected with the program, especially in the practical application of the data. Engineering authorities are hesitant about giving advice until further study can be given the various phases of the problem, but an excellent start toward a solution has been made. Credit must be given to the public-spirited attitude of those who have permitted publication of data relating to their buildings. This is done only when full consent is obtained. It is recognized that such publication can do no harm and is likely to result in great benefit.

The various chapters of this report have been prepared by those who were actually engaged in prosecuting the work. In many cases the chapter contains only a fraction of the data obtained, being primarily intended to describe the methods used and the nature of the results, with enough data to enable the reader to grasp the scope of the work. More detailed results on some of the projects are available in mimeographed form.

Among the institutions cooperating in this program the National Bureau of Standards was active in the development of strong-motion instruments and tilt meters; the California Institute of Technology in building-vibration studies and the investigation of structural damage; Stanford University in vibration-machine development and related vibration studies; the University of California in the operation of tilt meters and the collection and appraisal of noninstrumental reports; the Seismological Laboratory of the California Institute of Technology and the Carnegie Institution of Washington at Pasadena in the investigation of ground-wave periods recorded on seismographs, and the development of galvanometric recording seismographs for special study of structural vibrations; the Massachusetts Institute of Technology in the development of automatic starters, and the discussion of elevated tank vibration problems; and the University of Virginia in the development of strong-motion instruments. The success of the program to date is due largely to the enthusiastic cooperation of the interested personnel of these institutions as well as to others who were especially qualified by training and experience to act in an advisory capacity.

The following is a list of persons mentioned in this publication who have taken active parts in seismological investigations of engineering interest:

Mr. Maxwell Allen, author of a number of papers on the earthquake history of the west coast.
Dr. Hugo Benioff, Research Associate, Pasadena Seismological Laboratory of the Carnegie Institution of Washington, and the California Institute of Technology.
Dr. J. A. Blume, Observer, U. S. Coast and Geodetic Survey.
Prof. Perry Byerly, Director of the Seismological Station of the University of California, Berkeley.
Dr. J. J. Creskoff, Consulting Engineer, and Consultant on Aseismic Design, U. S. Treasury Department.
Mr. W. G. Corlett, Chairman, Architects Advisory Committee to the State Division of Architecture, Oakland, Calif.
Dr. A. L. Day, Chairman of the Advisory Committee in Seismology of the Carnegie Institution of Washington.
Dr. B. Gutenberg, Research Associate, Pasadena Seismological Laboratory of the Carnegie Institution of Washington and the California Institute of Technology.
Prof. M. Ishimoto, Director of the Seismological Research Institute of the Imperial University, Tokyo.
Prof. L. S. Jacobsen, Associate Professor of Mechanical Engineering, Stanford University.
Comdr. T. J. Maher, Inspector in Charge of the San Francisco Field Station of the Coast and Geodetic Survey.
Prof. R. R. Martel, Professor of Structural Engineering, California Institute of Technology.
Mr. R. S. McLean, Assistant Magnetic and Seismological Observer, U. S. Coast and Geodetic Survey.
Mr. H. E. McComb, Chief of Section of Observatories and Equipment, Division of Terrestrial Magnetism and Seismology, U. S. Coast and Geodetic Survey.
Mr. G. E. Merritt, Section of Interferometry, Division of Optics, National Bureau of Standards.
Mr. F. Neumann, Chief of Section of Seismology, Division of Terrestrial Magnetism and Seismology, U. S. Coast and Geodetic Survey.
Dr. C. F. Richter, Assistant Research Associate, Pasadena Seismological Laboratory of the Carnegie Institution of Washington and the California Institute of Technology.
Mr. E. C. Robison, formerly Junior Observer, U. S. Coast and Geodetic Survey.
Mr. A. C. Ruge, Research Associate in Seismology, Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology.
Prof. K. Suyehiro, late Director of the Earthquake Research Institute, Tokyo, Imperial University of Japan.
Mr. F. P. Ulrich, Associate Magnetic and Seismological Observer, in Charge of the U. S. Coast and Geodetic Survey seismological program in California.
Mr. A. J. Weed, Seismologist, University of Virginia.
Dr. F. Wenner, Chief of Electrical Resistance Measurement Section, National Bureau of Standards.
Mr. H. O. Wood, Director of the Pasadena Sesimological Laboratory of the Carnegie Institution of Washington and the California Institute of Technology.

N. H. Heck,
Chief, Division of Terrestrial Magnetism and Seismology,
United States Coast and Geodetic Survey.

EARTHQUAKE INVESTIGATIONS IN CALIFORNIA, 1934–35

Chapter 1.—FIELD PARTIES AND PROBLEMS

F. P. ULRICH

Plans for work were developed at a series of conferences in the San Francisco Bay region and in southern California which were attended by engineers, architects, seismologists and others interested for business reasons. Availability of existing instruments and methods and need for developing new ones to give the information needed for the design of earthquake-resistant structures were discussed.

Tentative plans by Dr. R. A. Millikan for southern California, by Mr. Henry Dewell representing a group of engineers, and by Messrs. Will G. Corlett and Walter T. Steilberg representing a group of architects, were considered at these conferences, and with many contributions from other members of the conferences, formed the basis of the plan adopted.

For practical operation charge of the work was assigned to the writer, who divided his party into subparties, each charged with specific projects. A brief description is given here of the different subparties and the work upon which each is engaged, together with comments on some of the problems that have arisen.

San Francisco office.—The work carried on at the San Francisco office could very well be divided into several parts. The first part consists of the administrative work, including the handling of accounts, purchase of supplies, correspondence, and contact work necessary to carry on some of the field work. The second part consists of the questionnaire program. This work is carried on in very close cooperation with Professor Byerly, of the University of California, and a separate report covering this work has been prepared by him and Dr. Henrietta S. Dyk. The third part consists of the study of the field records and the preparation and distribution of preliminary reports. Up to April 1, 95 reports had been prepared and distributed to interested engineers. These reports are in mimeograph form, giving the various periods of vibration and general information about the building and the ground upon which the building is located. More complete reports with pictures are placed for the use of engineers at the following places: University of California (Professor Byerly); Stanford University (Professor Jacobsen); California Institute of Technology (Professor Martel); and Massachusetts Institute of Technology (Mr. Ruge); secretary, Structural Engineers' Association of Northern California; secretary, Structural Engineers' Association of Southern California; secretary, San Francisco section, American Society of Civil Engineers; secretary, Los Angeles section, American Society of Civil Engineers; United States Coast and Geodetic Survey, Washington, D. C.; United States Coast and Geodetic Survey, 510

Customhouse, San Francisco; and United States Coast and Geodetic Survey, 286 Chamber of Commerce Building, Los Angeles, Calif.

Strong-motion work.—Two parties, one in northern California and one in southern California, have been engaged in handling the strong-motion work. This consists of the operation and servicing of 51 strong-motion instruments scattered throughout the State of California. A separate report has been prepared covering this work.

Vibration work.—Two parties have been engaged in vibration work. This work covers the observation of periods of vibration of various structures. Up to April 1, period observations had been made in 292 structures. A report on these observations is covered in a separate paper.

A brief history of development of instruments and methods will be of interest. The only previous similar work in the region was by Prof. Perry Byerly who had measured the periods of 16 buildings in San Francisco using the Hall vibration meter. This instrument, which resembles the Bosch-Omori seismograph except that is has a vertical component, proved satisfactory only where a large amplitude of motion could be obtained. Besides it is large and heavy, and requires considerable time for setting up and dismantling, and accordingly it is not suited to quantity observations.

A very light and portable instrument developed by Mr. Frank Neumann, of the Coast and Geodetic Survey, was tried out. It has a small mass attached at the end of a horizontal arm which is free at one end. The particular feature is the practically frictionless transmission of the vibrations to the recording mirror. This apparatus showed good possibilities although in its only available form it was suited to only a few problems. Later Mr. Ralph S. McLean used the same principle in another instrument with which he measured some building vibrations. He also built a recorder which uses motion-picture film. This recorder is small enough to be used advantageously in places such as tank towers, where only a limited space is available.

The Coast and Geodetic Survey has developed a small portable type of vibration meter which is easy to set up, gives a range in period up to 6 seconds, and has a magnification around 200. The first experimental instrument was used continuously in northern California until the working instruments were constructed. In southern California, several regular Wood-Anderson seismometers were used extensively for building vibration work. These instruments are still used occasionally when a high magnification is desired, as for instance, on dams or similar rigid structures. Each of the two parties now has several of the new vibration meters and recorders which are equipped with a common simultaneous time-marking system. This has been found very useful in making detailed studies of buildings. Descriptions and pictures of these instruments appear in this publication in the chapter on vibration work.

It appeared evident in the course of the conferences that additional instruments would be needed for recording earthquakes and other vibrations, and it was believed that suitable instruments using the same general principle as the Benioff seismometer[1] would prove especially useful. Dr. Benioff agreed to supervise such development

[1] H. Benioff. A New Vertical Seismograph, Bulletin of the Seismological Society of America. Vol. 22, No. 2, June, 1932, p. 155.

work at the Seismological Research Laboratory at Pasadena. The results are described on p. 226.

Ground and building vibrator.—Under Prof. L. S. Jacobsen and Mr. John A. Blume, a ground and building vibrating machine has been designed, constructed and tested in the field. For building work, this machine has proved very satisfactory, but the usefulness of this particular machine for determining ground periods is still uncertain due primarily to the relatively small amount of power that can be generated. The report on this work will be discussed in another chapter.

Other field work.—A party under the supervision of Prof. R. R. Martel has been engaged in a study of damage to type III masonry buildings during the Long Beach earthquake. An exhaustive search has been made from available records and reports to determine building damage, which was later verified by actual inspection of the building.

In another project under Dr. Gutenberg, the work consisted of measuring thousands of earthquake periods on seismograms and arranging these periods in tabular form. As both projects are covered in separate chapters no detailed discussion will be given here.

Chapter 2.—STRONG-MOTION PROGRAM AND TILTMETERS

N. H. Heck, H. E. McComb, and F. P. Ulrich

STRONG-MOTION PROGRAM

The original appropriation for the California seismological program provided funds for the purchase of a number of strong-motion instruments to be installed in California where there was the greatest opportunity for measuring the maximum forces set up by earthquakes. In selecting the various locations for the instruments, preference was given, in most cases, to areas having the greatest population or where there was a concentration of large buildings. The main purpose of the

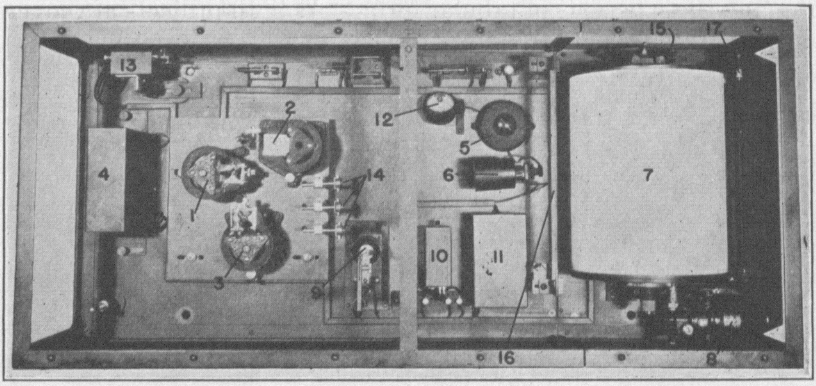


Figure 1.—Accelerograph, plan view. (1) Longitudinal accelerometer, (2) vertical accelerometer, (3) transverse accelerometer, (4) time-marking clock, (5) lamp rheostat, (6) lamp, (7) recording drum, (8) driving motor, (9) pendulum starter, (10) control relay, (11) starting relay, (12) milliammeter, (13) simultaneous time-marker, (14) base-line mirrors, (15) revolution counter, (16) cylindrical lens, (17) tape attachment.

strong-motion instruments is to gather seismological data necessary in designing structures which are reasonably earthquake-resistant. The engineer must know, among other things, something about the forces which such structures will have to withstand. In the past most buildings have been designed primarily from a static viewpoint, considering only vertical loads. The engineer has had to consider primarily the dead vertical loads, and such maximum live loads that were considered were just that much more weight added to dead load. These live loads were relatively slow moving and there was usually an appreciable interval between the maximum and minimum. The live horizontal loads have been treated very similarly to the vertical loads. When such horizontal loads have been considered, it has usually been a wind force, in which case the change from maximum to minimum has taken an appreciable interval of time. In earthquakes this interval is sometimes only a few tenths of a second and in such cases static considerations do not hold. The problem becomes one of dynamics. In a dynamic analysis the necessary data consist of continuous records of acceleration and displacement during the active period of the earthquake. This is the kind of information obtained by the strong-motion instruments.

The instruments developed for the respective purposes are called accelerographs (fig. 1) and displacement meters (fig. 2). Each consists of the instruments which respond lirectly to the earthquake, a starting device, the automatic recorder. the time-marking clock, the light and optical system, electric circuits, and batteries. Local current supply cannot be used since it is likely to be put out of commission by the earthquake. The starting device and some of the special features of the automatic recorder are necessary in order that the instruments may remain in a state of inactivity until an earthquake occurs, and then instantly start making a satisfactory record. In view of the possibility of long delay, continuous recording such as is customary with ordinary seismographs is impracticable on account of the cost of photographic paper and attendance. The time-marking device is simply a shutter attached to the balance staff of a spring clock,

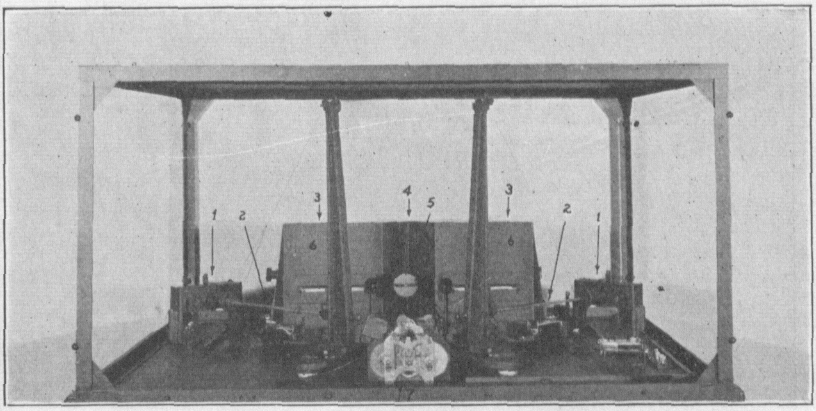


FIGURE 2.—Displacement meter, end view. (1) Damping wells, (2) booms, (3) recording drums behind shield, (4) driving motor behind shield, (5) lamp, (6) shield. (7) time-marking clock.

which every half second eclipses the light from a fixed source and puts a mark 0.2 second long on the side of the record.

The automatic recorders, developed by the Coast and Geodetic Survey, especially through the work of Messrs. D. L. Parkhurst, H. E. McComb, and Edward C. Robison, are motor-driven (fig. 1). When the starting device operates, the light goes on, the recorder motor starts, the drum starts turning at a peripheral speed of about 10 millimeters per second, the clock is released, and a buzzer operates, signaling anyone in the vicinity that the recorder is in operation. An addition found necessary for satisfactory operation is a counting device in a suitable place outside the recording room so that an attendant can determine if the recorder has been in operation, also whether it has turned a sufficient number of times to require a replacement of the paper. After completing 1⅓ turns, the drum stops automatically and all other activity ceases unless the earthquake is still going on, in which case the above cycle is repeated. In some cases a second shock may be delayed, in which case the recorder remains at rest with the light off until it occurs, after which the above cycle is repeated. In the earlier models the drum was papered with a single sheet of photographic paper, and moved longitudinally by a helical screw to reduce overlapping of the traces. In has been found more satisfactory to use a longer strip of paper winding on spools and passing around the drum. Tape attachments embodying this principle have been installed on all the accelerographs. In the later models the width of the recording drum has been increased from 6 to 12 inches, and provision has been made for flashing time marks on both edges of the paper.

Another important improvement in the newer models is a wiring arrangement by which 2 or 3 accelerographs located in different parts of a building can be connected together so that they will all start record-

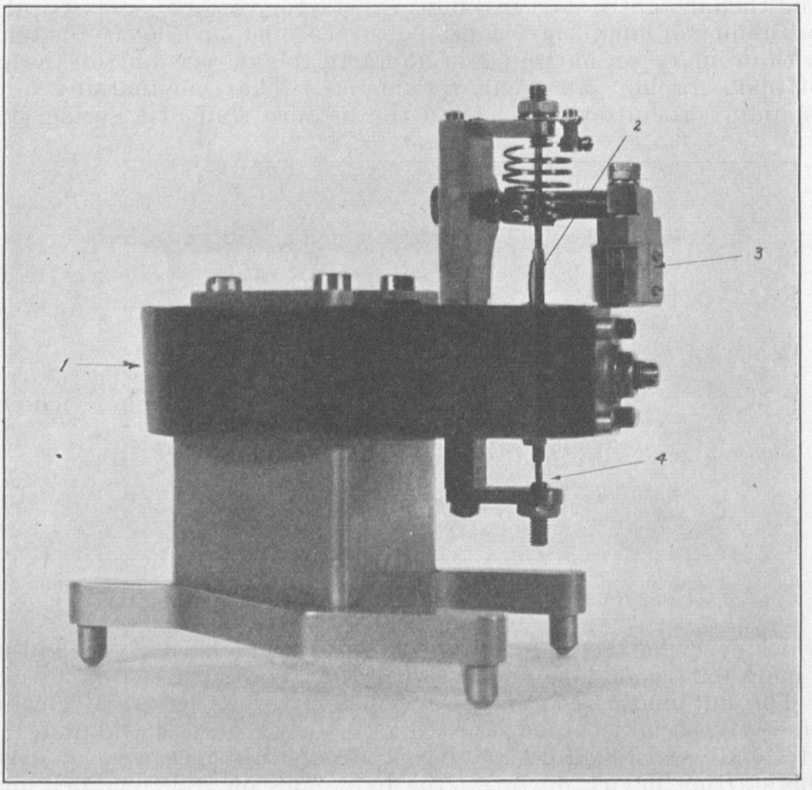


FIGURE 3.—Accelerometer. Transverse component, showing mirror just above damping magnet, and prism for deflecting light so that the deflected light will be brought parallel to the light beams from the other components. (1) Damping magnet, (2) mirror, (3) prism, (4) pivot suspension.

ing at the same time, and the time marks on the records will be made simultaneously, providing a basis for comparison of the motion of the different parts of the structure.

An accelerograph has three accelerometers on a common base. As in the case of seismographs, there are three components set at right angles to each other. There is also a starting pendulum and the parts that have been described. The whole apparatus is placed in a light-tight box, 45 inches long by 20 inches wide and 12½ inches high provided with a removable cover, one end of which is hinged so that there can be access to the drum for the removal of paper and repapering without taking off the cover. The box is rigidly bolted to a concrete block. The whole is enclosed in a dark room which should be kept locked. This may be an enclosure in a larger room and of inexpensive construction.

The parts of an accelerograph requiring description are the accelerometers and the starting devices. The accelerometers (fig. 3) were designed by Dr. Frank Wenner, of the Bureau of Standards. The late Prof. K. Suyehiro in one of his lectures stated that the Wood-Anderson torsion seismometer would make an excellent accelerometer. This statement needs modification since that instrument has a very high sensitivity and can be used only for very moderate earthquakes. However, what he meant was that it embodied a correct principle.

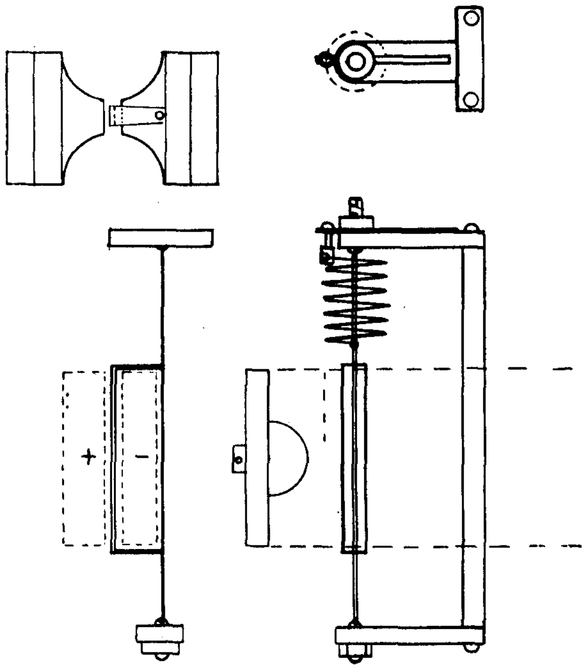


FIGURE 4.—Diagram of accelerometer with pivot suspension.

Dr. Wenner has recognized this in utilizing the principle to some extent while making a very different type of instrument.

The accelerometer has three essential parts in addition to the supporting frame: A suspension, a loop or vane, and a cobalt-steel magnet. Two types of suspension have been used. In the quadrifilar suspension, fine wires are threaded through four holes at the top and bottom. Midway between the top and bottom the suspension passes through and is rigidly clamped to a small tube which is part of a rectangular loop or vane forming the steady mass of the accelerometer. The suspension is pyramidal in form above and below the tube in order to prevent the setting up of so-called "violin-string vibrations" in the suspension itself. In most seismometers the essential part is a mass so supported that it is free to act as a pendulum or to swing around an axis, and in this case the loop or vane has that function. Torsion in the suspension furnishes the restoring force when the mass is moved from its position of rest with respect to the supporting

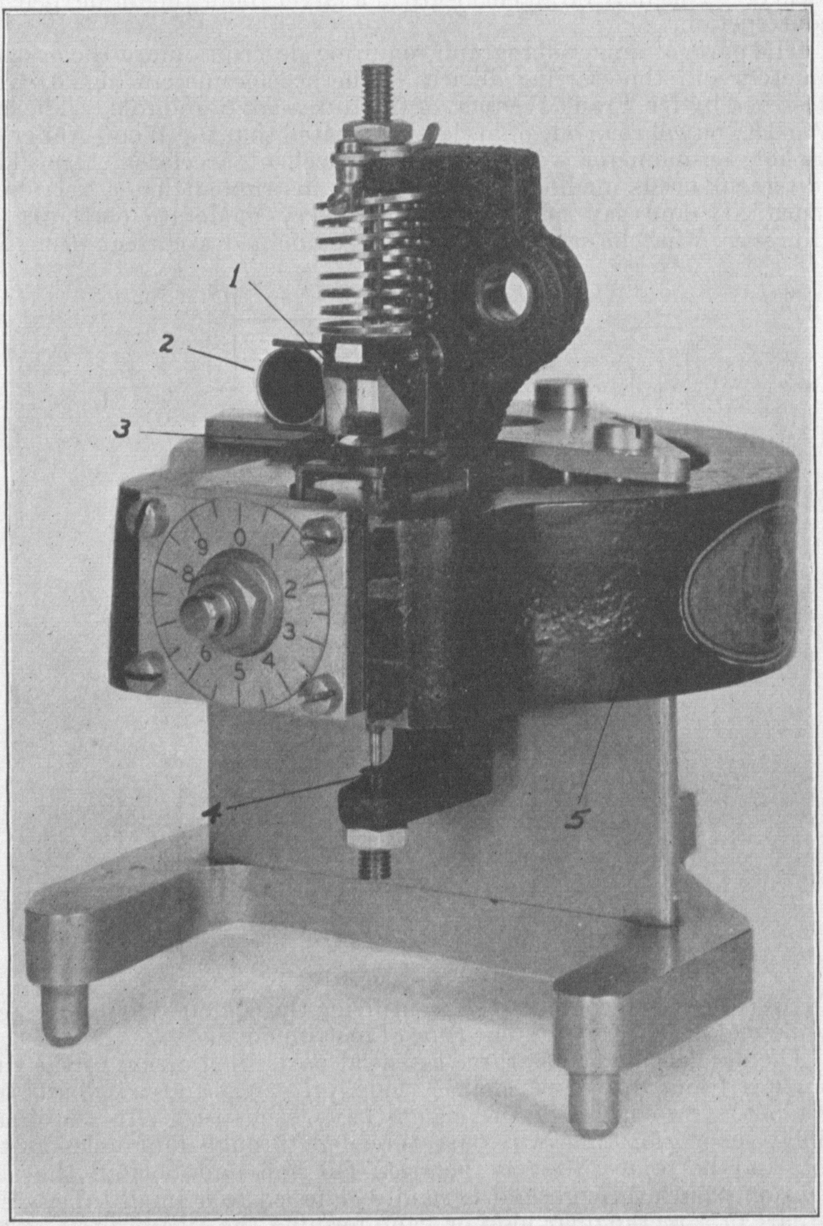


FIGURE 5.—Accelerometer with insensitive attachment. (1) Prism, (2) regular mirror, (3) insensitive mirror, (4) pivot suspension, (5) damping magnet.

frame. When the suspension is placed horizontal to record vertical motion, additional torsion is necessary to hold the vane horizontal.

The objection to this arrangement, which is otherwise satisfactory, is the difficulty of adjustment. A modified pivot suspension (figs. 3

and 4) developed by Mr. H. E. McComb, which is easy to adjust, has been substituted for the quadrifilar suspension. In this, a slender shaft which rests in pivots at each end has attached to it the vane and mirror. The restoring force is furnished by a helical spring similar to that used in chronometers. The period of the instrument is adjusted through change in the length of the spring. The accurate adjustment of the period is accomplished through tuning with an unbalanced synchronous motor. Suitable optical arrangements bring the beams from all three components parallel and in the same horizontal plane to the recorder. The outer part of the loop is free to move in the strong magnetic field of the cobalt-steel magnet, thus providing the necessary damping. The magnet may be removed to make necessary tests of the suspension.

A mirror attached to the vane makes possible photographic recording of its movement with reference to the supporting frame and, therefore, with reference to the earth. A recent improvement (fig. 5) consists in the addition of an auxiliary mirror, attached at an angle in such a way as to provide on the record, in addition to the regular trace, another trace of reduced amplitude so that if the motion is sufficiently violent to carry the regular light spot off the paper, the auxiliary spot will still be recorded. It is thought that with this arrangement the instrument will now be able to go through the strongest earthquake without loss of record.

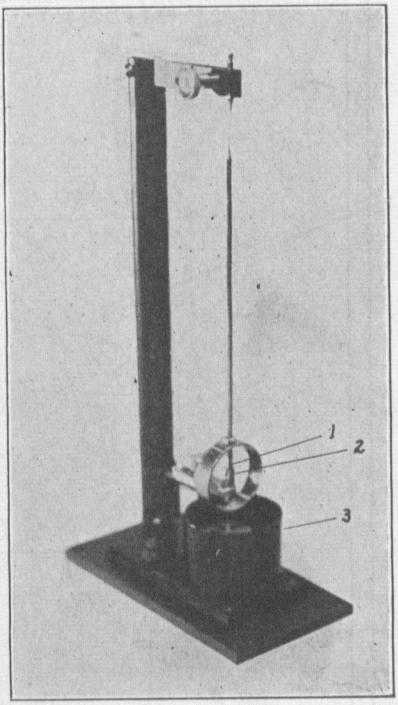


FIGURE 6.—Pendulum starter. (1) Platinum conical point, (2) platinum cup, (3) damping well.

The first starting accelerometer used was the Braunlich contact accelerometer (shown at 4 in fig. 8) developed by Mr. M. W. Braunlich, of the Massachusetts Institute of Technology. It consists of a mass mounted on a spring support in such a way that, when a certain acceleration is reached, an electric contact is broken. It was found in practice that, in many cases, vibrations from various nonseismic causes operated the starter. It was changed to a make-circuit device and in this form its use is being continued, especially for the Weed instrument described later. However, there is another serious objection. The instruments respond fully only to an impulse in one direction. Theoretically, there should be at least six contact accelerometers for each accelerograph and this is impracticable. Actually, two are used and, therefore, the response to impulses varies greatly with the direction from which the impulse comes.

The type of starter now generally in use (fig. 6), which was developed by Mr. H. E. McComb, is a pendulum of 1-second period which carries a platinum cone separated by an air space from a platinum ring. With small movements of the pendulum such as might be caused by artificial vibrations, no contact is made, but when the motion is sufficient, as in a strong earthquake, contact is made and an electrical circuit closed. It requires about 0.2 second for an earthquake to start a strong motion seismograph.

Although the accelerometers are designed primarily to measure ground accelerations, it is possible, in some cases, by the methods

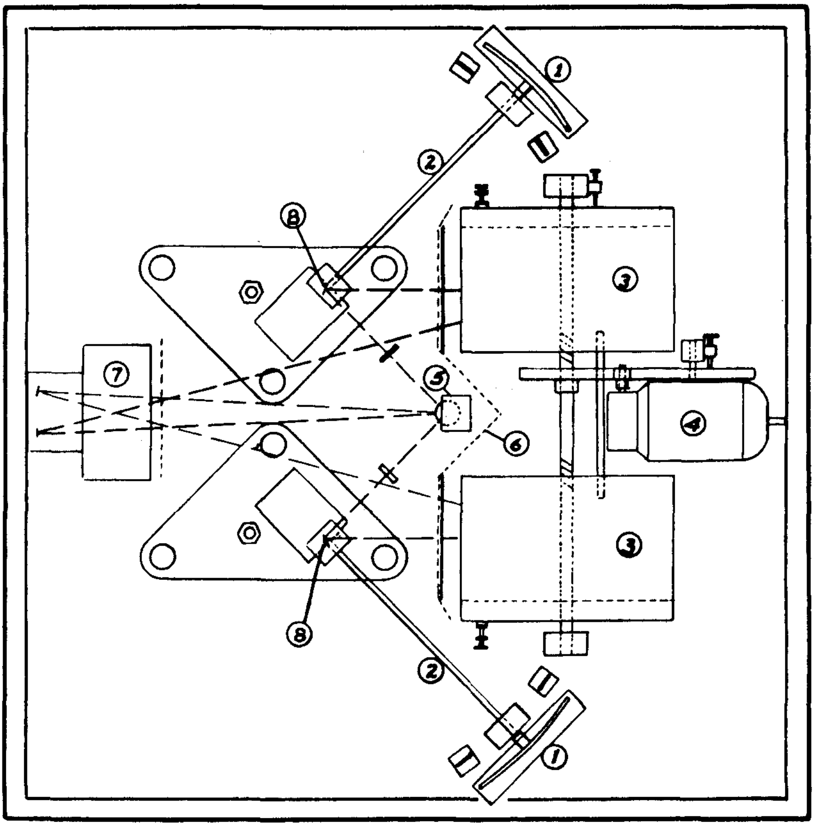


FIGURE 7.—Plan of displacement meter. (1) Damping wells, (2) booms, (3) recording drums, (4) driving motor, (5) lamp, (6) shield, (7) time-marking clock, (8) mirrors.

developed by Mr. Frank Neumann, of the United States Coast and Geodetic Survey, to integrate the records obtained with them and obtain ground displacements. While the full usefulness of such methods has not been definitely established, every effort is being made to facilitate its application, especially in obtaining the type of records most suitable for such work.

The instruments intended primarily for measurement of ground displacement, and hence called displacement meters, although of different detail from the accelerographs, are fundamentally the same except for the type of seismometer pendulum used and the starting device. The former is an ordinary seismograph (fig. 2) of the horizontal pendulum type with two horizontal components set at right angles to each other, developed by H. E. McComb and Edward C. Robison, of the United States Coast and Geodetic Survey. The detail of the pivot at the cup bearing, where the boom carrying the steady mass meets the vertical column, is devised to prevent earthquake damage to the instrument and failure to record. In the Japanese earthquake of 1923, a number of seismographs failed because the boom pulled away from the bearing and then swung back damaging either the pivot or the cup bearing, or the boom stayed out of the bearing entirely, and so rendered the instrument useless. In the displacement meter a guard prevents the pivot from leaving the cup bearing by more than any desired amount such as one-thirty-second of an inch, though the amount is adjustable. The steady mass attached to the boom weighs 1 pound and the optical arrangements are such that the magnification is unity. The pendulum is

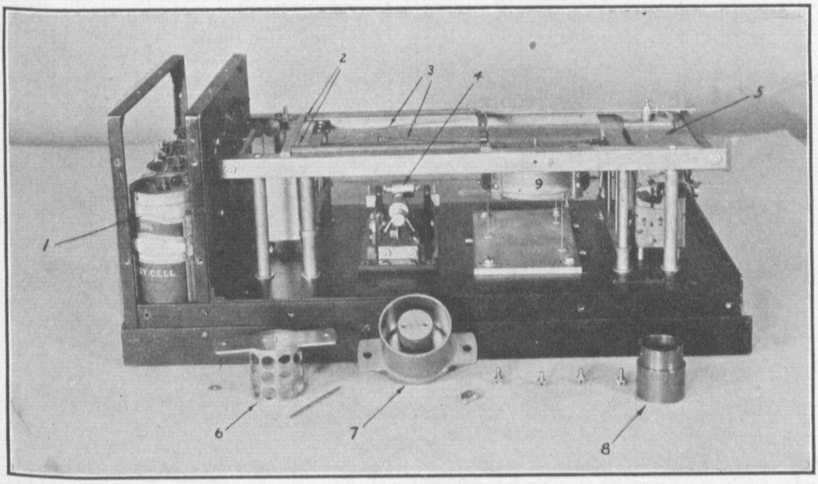
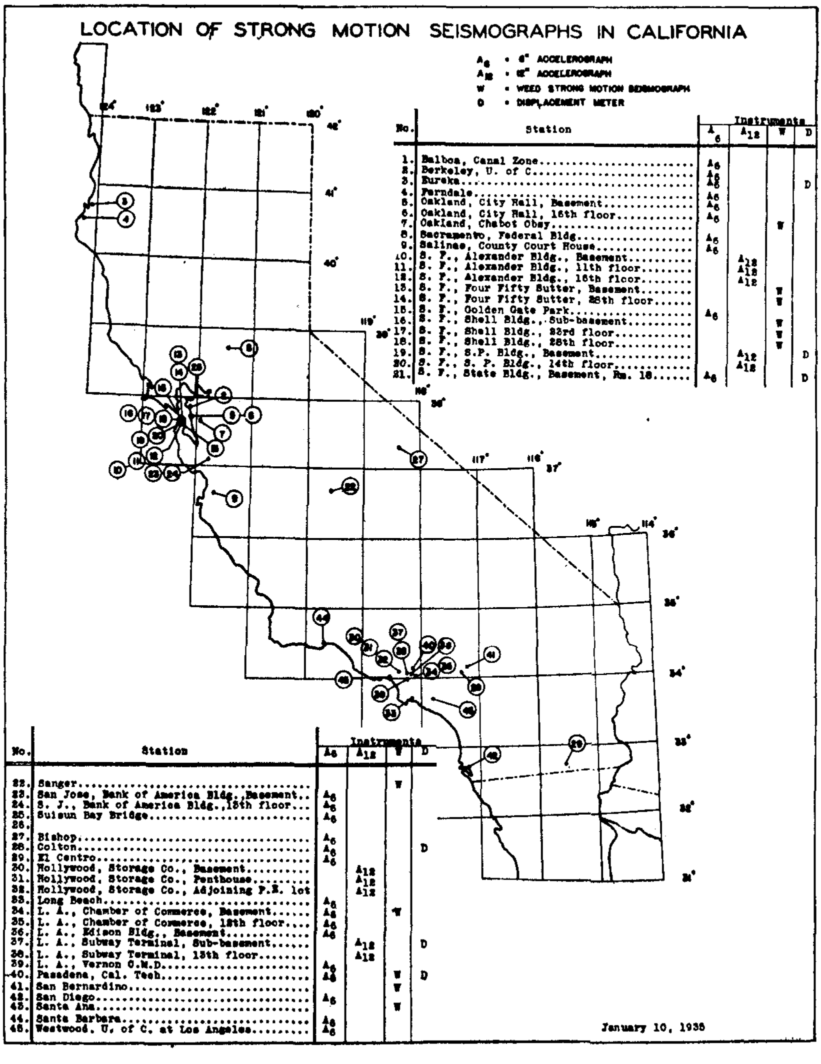


FIGURE 8.—Weed seismograph. (1) Batteries, (2) plate holder, (3) recording arms, (4) Braunlich starters, (5) driving clock, (6) damping vane, (7) oil well, (8) damping adjuster, (9) inverted pendulum showing three wire supports.

oil-damped. A sketch of the assembly is shown in fig. 7. The instrument and all accessories are enclosed in a light-proof case 44 by 40 by 20 inches high.

The starting device in this case is attached directly to the boom and is of the make-circuit type. Contact is made when the outer end of the boom has moved through about one-third inch, though this, too, is adjustable. At present all displacement meters are located at stations provided also with accelerographs, and, to secure simultaneity in the records, the two instruments are wired together so that the displacement meter is started by the pendulum starter of the accelerograph instead of by its own starting device. This arrangement has also the advantage of freedom from spurious starts due to tilts.

A less expensive and somewhat less accurate instrument, which, however, has possibility of great usefulness, has been tested in a model form and a number have been installed. The Weed strong-motion seismograph (fig. 8), designed by Mr. Arthur J. Weed, of the University of Virginia, is very compact, its outside dimensions being 20 inches long by 8 inches wide by 8 inches high. This does not require a dark room and therefore can be used in many places where a more elaborate installation is out of the question. Its essential features are: a cylindrical steady mass of about six pounds, resting on three vertical wires placed in the form of an equilateral triangle, to



No.	Station	Instruments A_6	A_{12}	W	D
1.	Balboa, Canal Zone	A_6			
2.	Berkeley, U. of C.	A_6			
3.	Eureka	A_6			D
4.	Ferndale	A_6			
5.	Oakland, City Hall, Basement	A_6			
6.	Oakland, City Hall, 16th floor	A_6			
7.	Oakland, Chabot Obsy.			W	
8.	Sacramento, Federal Bldg.	A_6			
9.	Salinas, County Court House	A_6			
10.	S. F., Alexander Bldg., Basement		A_{12}		
11.	S. F., Alexander Bldg., 11th floor		A_{12}		
12.	S. F., Alexander Bldg., 16th floor		A_{12}		
13.	S. F., Four Fifty Sutter, Basement			W	
14.	S. F., Four Fifty Sutter, 28th floor			W	
15.	S. F., Golden Gate Park	A_6			
16.	S. F., Shell Bldg., Sub-basement			W	
17.	S. F., Shell Bldg., 23rd floor			W	
18.	S. F., Shell Bldg., 28th floor			W	
19.	S. F., S.P. Bldg., Basement		A_{12}		D
20.	S. F., S. P. Bldg., 14th floor		A_{12}		
21.	S. F., State Bldg., Basement, Rm. 18	A_6			D

No.	Station	Instruments A_6	A_{12}	W	D
22.	Sanger			W	
23.	San Jose, Bank of America Bldg., Basement	A_6			
24.	S. J., Bank of America Bldg., 13th floor	A_6			
25.	Suisun Bay Bridge	A_6			
26.					
27.	Bishop	A_6			
28.	Colton	A_6			D
29.	El Centro	A_6			
30.	Hollywood, Storage Co., Basement		A_{12}		
31.	Hollywood, Storage Co., Penthouse		A_{12}		
32.	Hollywood, Storage Co., Adjoining P.E. lot		A_{12}		
33.	Long Beach	A_6			
34.	L. A., Chamber of Commerce, Basement	A_6		W	
35.	L. A., Chamber of Commerce, 12th floor	A_6			
36.	L. A., Edison Bldg., Basement	A_6			
37.	L. A., Subway Terminal, Sub-basement		A_{12}		D
38.	L. A., Subway Terminal, 13th floor		A_{12}		
39.	L. A., Vernon C.M.D.	A_6			
40.	Pasadena, Cal. Tech.	A_6		W	D
41.	San Bernardino			W	
42.	San Diego	A_6			
43.	Santa Ana			W	
44.	Santa Barbara	A_6			
45.	Westwood, U. of C. at Los Angeles	A_6			

FIGURE 9.—Location of strong-motion seismographs in California.

which an oil damping device is attached. The period is fixed by the weight of the mass and the stiffness and length of the wires. A short rod projects above the mass in the line of its axis. Two slotted levers are coupled to the mass by this means and they in turn each operate a recording stylus. The record is made on the under side of a smoked-glass plate. Since the slots are at right angles to each other, a motion

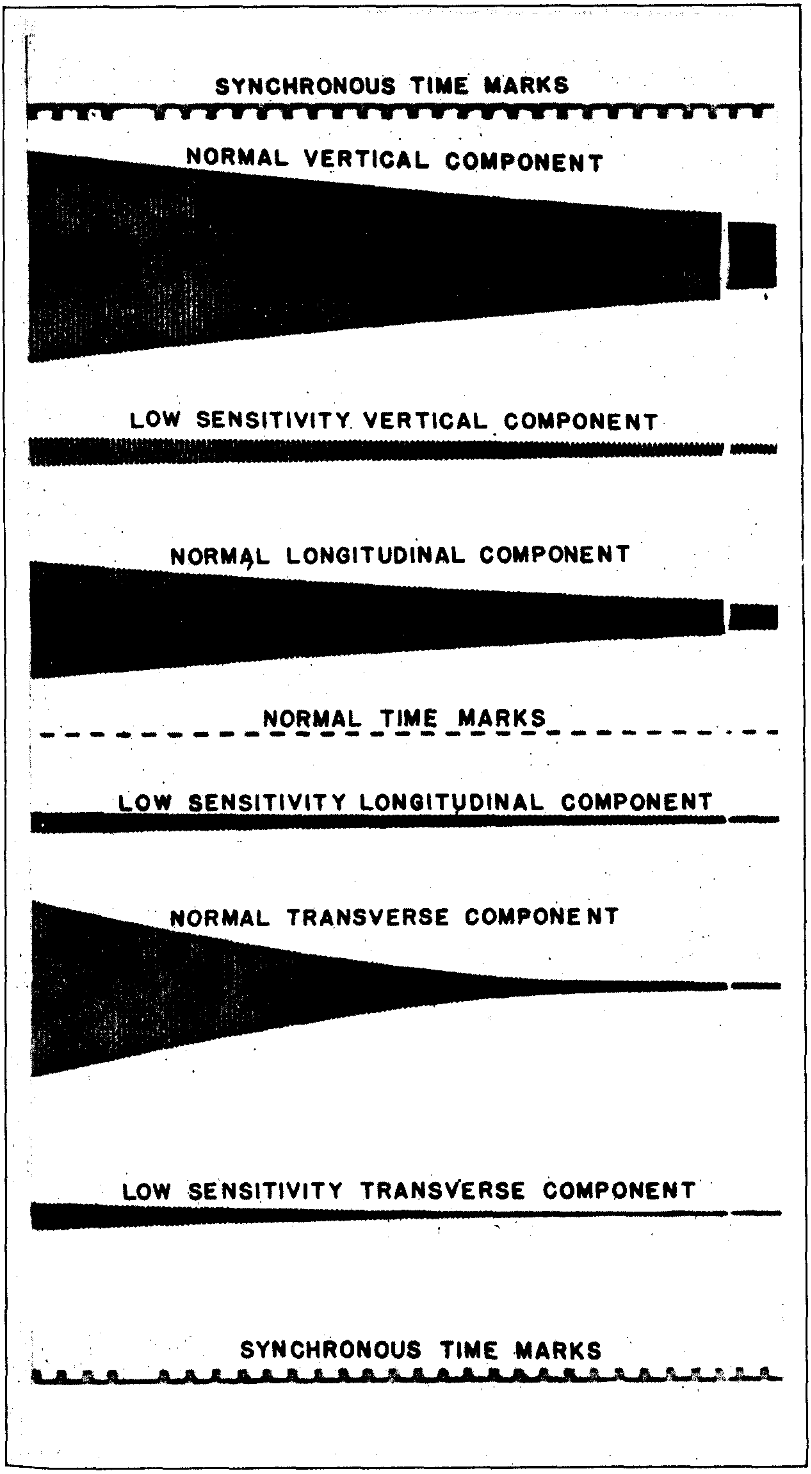


FIGURE 10.—Test record of accelerograph.

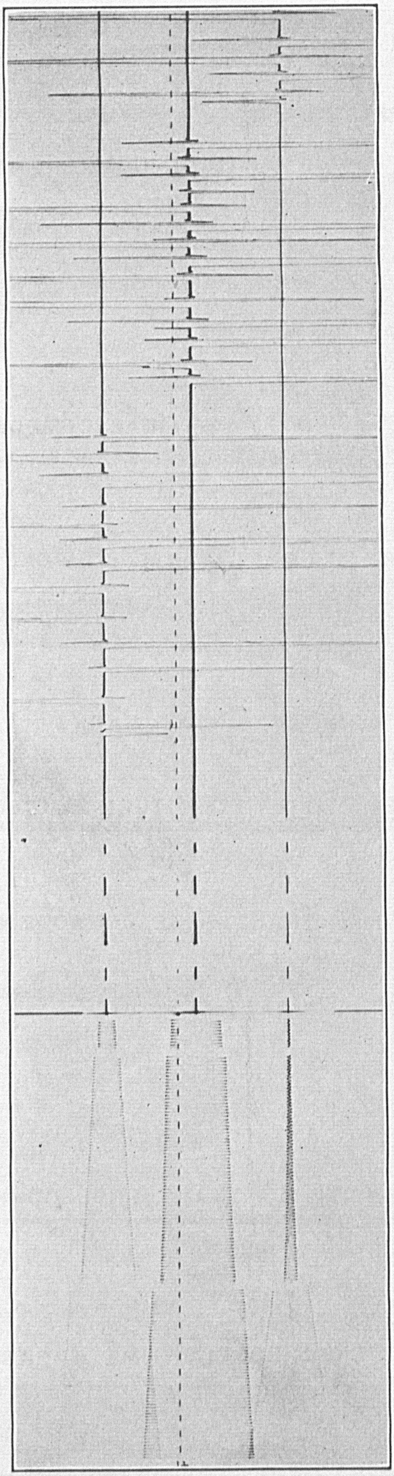


FIGURE 11.—Accelerograph damping and decay tests. The oscillations on the left were recorded with damping magnets removed; those on the right with magnets in normal position.

in two directions is recorded as in the case of two components. The glass plate is moved by clockwork and can be used throughout the length of its travel of 7 inches. Since there are no time marks, re-

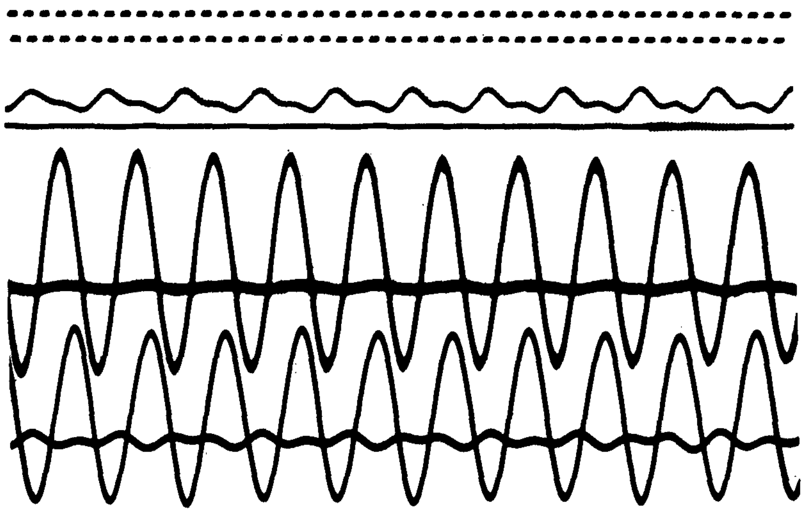
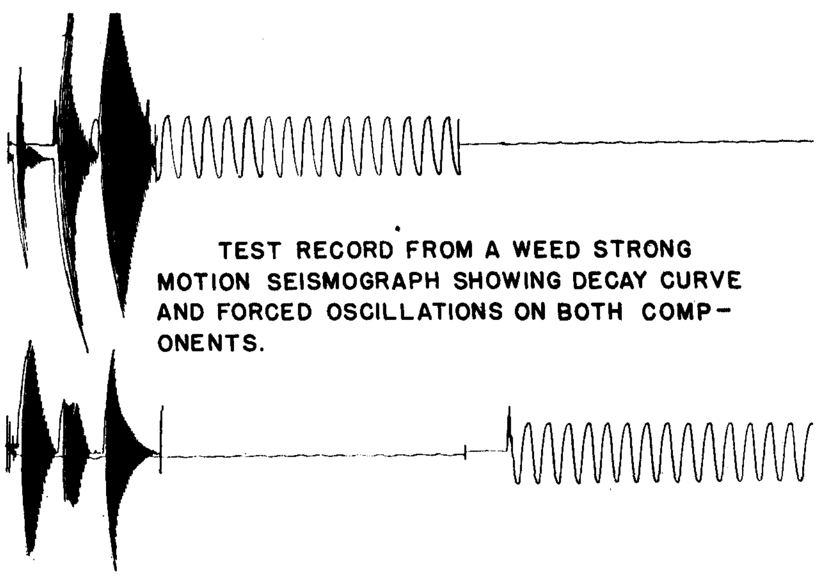


FIGURE 12.—Swinging platform test of three-component accelerograph. The platform motion is elliptical with a small vertical component. From top to bottom, counting two lines to each element, the record shows half-second time marks, vertical motion, east-west motion, and north-south motion.



FIFURE 13.—Test record from Weed strong-motion seismograph.

liance must be placed on the accuracy of the rate of the driving clock, which is at best somewhat uncertain since it starts from rest. The same applies to the drum rate for the other types of instrument, but in their case any irregularity can be measured and allowed for by use

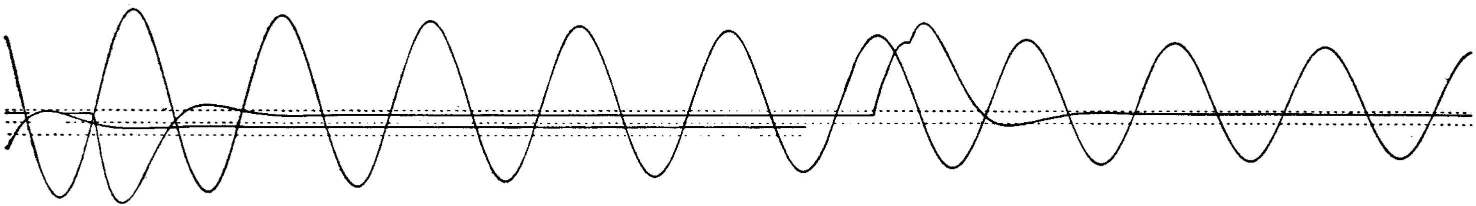


FIGURE 14.—Test record from displacement meter.

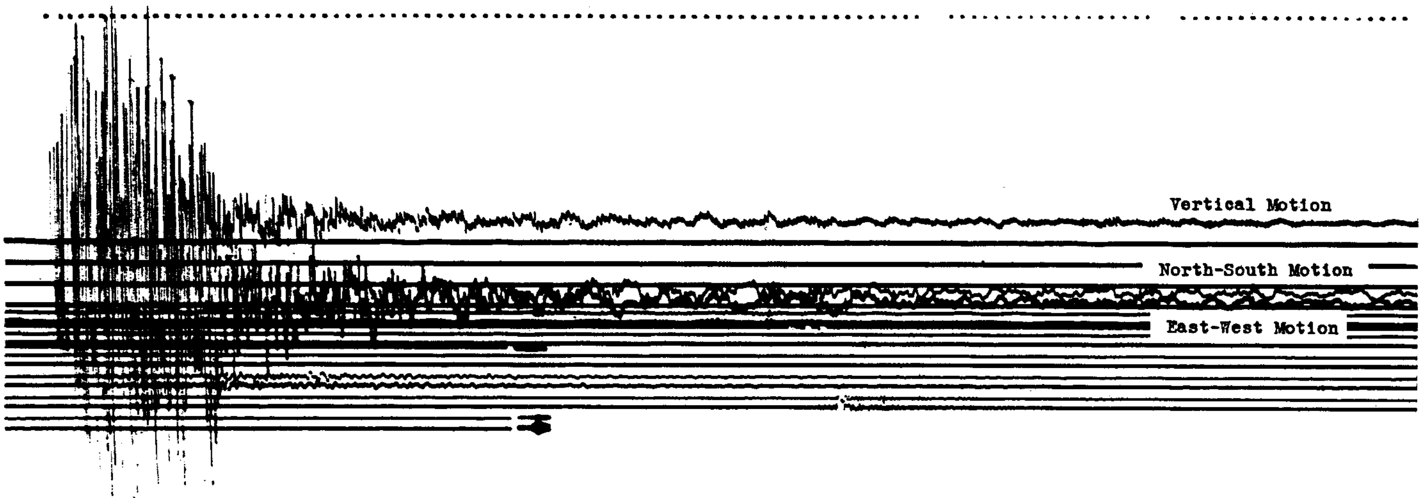


FIGURE 15.—Accelerograph record of the Long Beach earthquake. Record obtained at Long Beach

of the time marks flashed on the record. An attempt was made to provide a mechanical starting device, but this proved uneven in performance and a simple pendulum starter was substituted.

From experience obtained in operating the strong-motion instruments, it has been found advisable to have regular monthly inspection

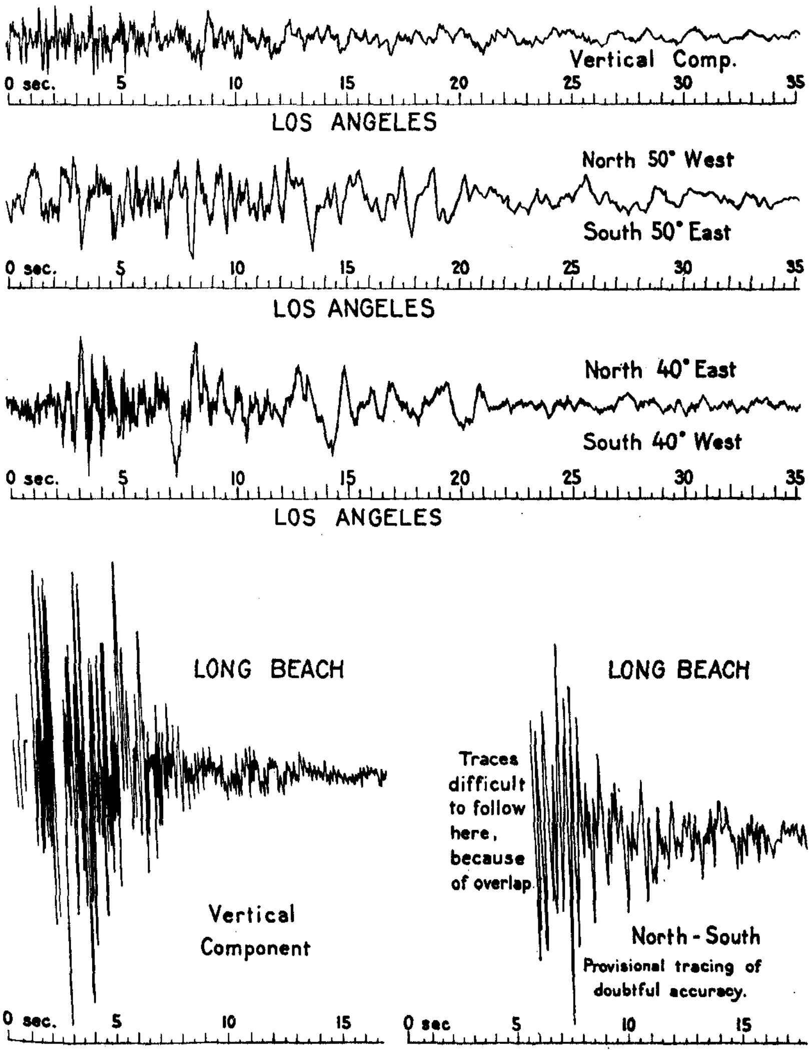


FIGURE 16.—Accelerogram tracings, Long Beach earthquake of March 10, 1933.

of each instrument. Tests are made for free period, decay curve, damping, and parallax between the various traces. The lighting system is also thoroughly checked. Power for displacement meters and accelerographs is furnished by two 6-volt storage batteries. There is a small drain upon the batteries at all times, and at present the

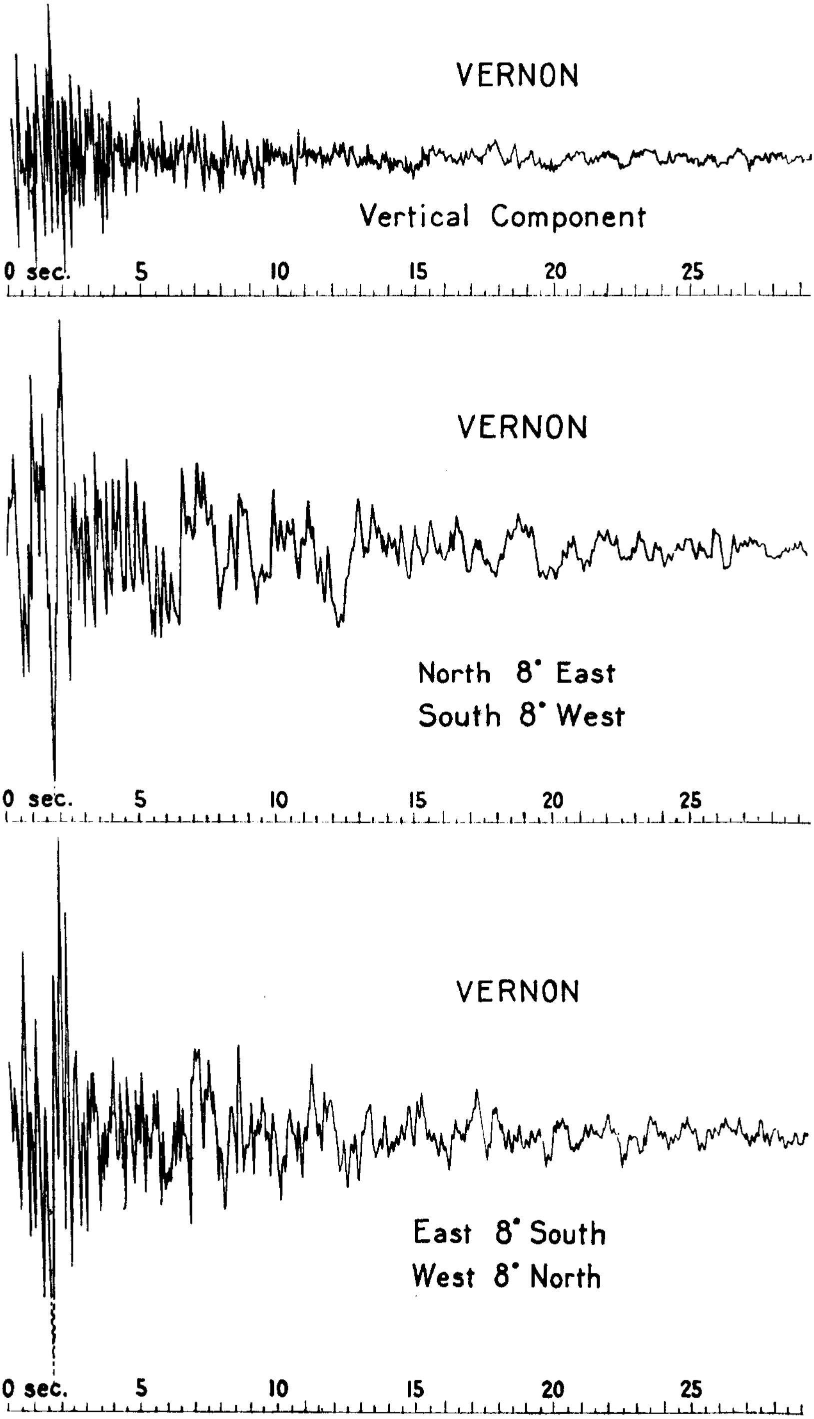


FIGURE 17.—Tracings of Vernon accelerograms, Long Beach earthquake of March 10, 1933.

batteries are kept charged by a microrectifier in continuous operation. These microrectifiers have an output ranging from 25 to 75 milliamperes. It has been found that an output of about 40 milliamperes keeps the batteries well charged.

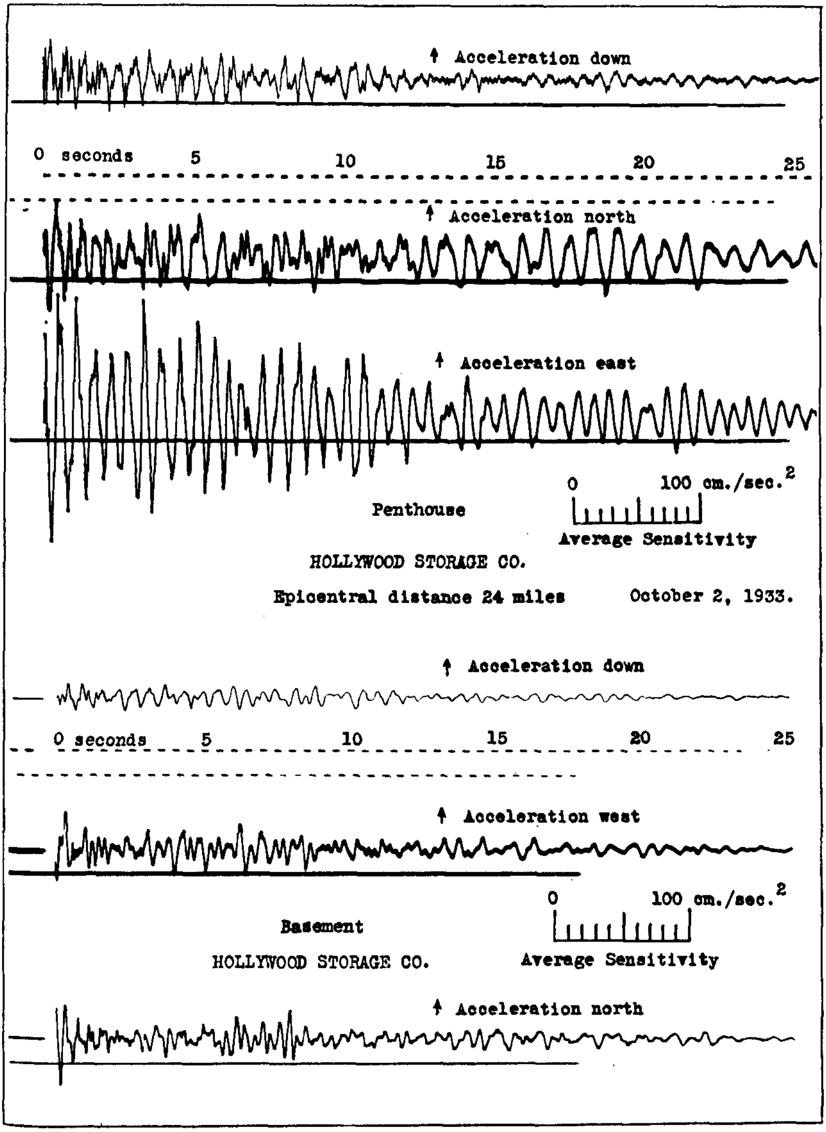


FIGURE 18.—Accelerogram tracings, southern California earthquake of October 2, 1933.

The locations of the instruments that have been installed are shown in figure 9 and listed in table 1. The earthquakes for which strong-motion records have thus far been obtained are listed in table 2. In figures 10 to 14 are shown test records for the 3 types of instrument, and in figures 15 to 20 actual records of earthquakes for the first 2 considerable shocks recorded, those of March 10 and October 2, 1933.

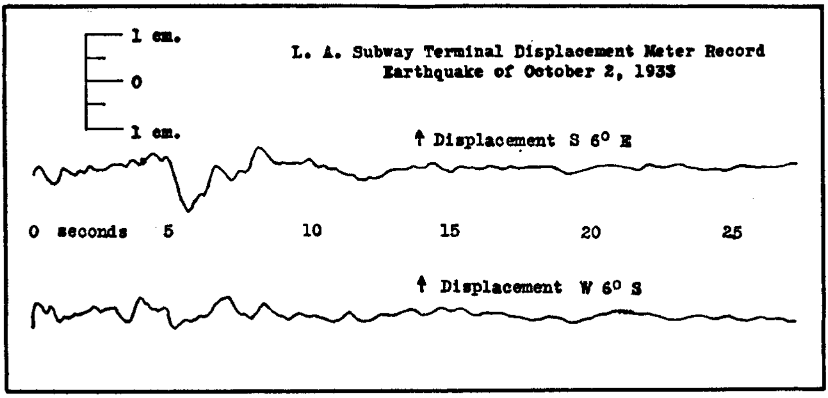


FIGURE 19.—Los Angeles subway terminal displacement-meter record of earthquake of October 2, 1933

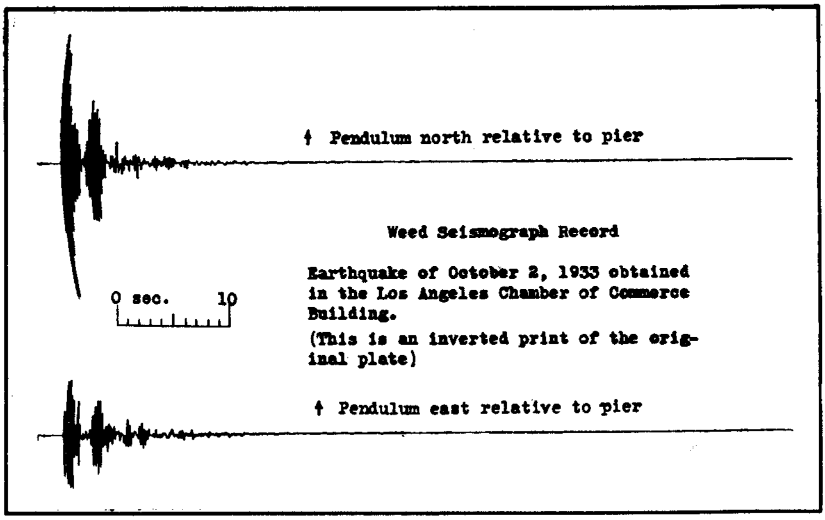


FIGURE 20.—Weed seismograph record, earthquake of October 2, 1933.

TABLE 1.—*Location of 48 strong-motion instruments in California*

Station and foundation	Instrument	Date of installation
NORTHERN CALIFORNIA		
Berkeley:		
University of California: Solid rock	Accelerograph	November 1932.
Eureka:		
Federal Building: Alluvium	Accelerograph and displacement meter.	May 1933.
Ferndale:		
City Hall: Alluvium	Accelerograph	Do.
Oakland:		
City Hall: Alluvium:		
Sixteenth floor	do	November 1934.
Basement	do	June 1933.
Chabot Observatory: Solid rock	Weed seismograph	Do.
Sacramento:		
Federal Building: Alluvium	Accelerograph	Do.
Salinas:		
County Court House: Alluvium	do	Do.

TABLE 1.—*Location of 48 strong-motion instruments in California*—Continued

Station and foundation	Instrument	Date of installation
NORTHERN CALIFORNIA—continued		
San Francisco:		
Alexander Building: Alluvium:		
Sixteenth floor	Accelerograph	November 1934.
Eleventh floor	do	Do.[1]
Basement	do	Do.
Four Fifty Sutter: Rock:		
Twenty-eighth floor	Weed seismograph	Do.
Basement	do	Do.
Golden Gate Park: Rock	Accelerograph	August 1933.
Shell Building: Rock:		
Twenty-eighth floor	Weed seismograph	October 1933.
Twenty-third floor	do	May 1934.
Subbasement	do	October 1933.
Southern Pacific Building: Alluvium and made ground:		
Fourteenth floor	Accelerograph	October 1934.
Basement, Room 18	Accelerograph and displacement meter.	July 1933.
Sanger:		
Residence of Maxwell Allen: Alluvium	Weed seismograph	June 1933.
San Jose:		
Bank of American Building: Alluvium:		
Thirteenth floor	Accelerograph	September 1932.
Basement	do	Do.
Suisun Bay Bridge, S. P. R. R. bridge pier: Rock	do	August 1932.
SOUTHERN CALIFORNIA		
Bishop:		
Office Los Angeles Water Department: Alluvium	do	June 1933.
Colton:		
Southern California Edison substation: Alluvium	Accelerograph and displacement meter.	January 1933.
El Centro:		
Southern Sierras Power Co., substation: Alluvium	Accelerograph	July 1932.
Hollywood:		
Storage Co.: Alluvium:		
Penthouse	do	June 1933.
Basement	do	Do.
Adjoining Pacific Electric lot[2]	do	December 1934.
Long Beach:		
Public Utilities Building: Alluvium	do	July 1932.
Los Angeles:		
Chamber of Commerce: Alluvium:		
Twelfth floor	do	November 1934.
Basement	Accelerograph and Weed seismograph.	June 1933.
Edison Building: Harpan or clay:		
Basement	Accelerograph	December 1934.
Subway Terminal: Hardpan or clay:		
Thirteenth floor	do	Do.
Subbasement	Accelerograph and displacement meter.	August 1932.
Vernon—Central Manufacturing District: Alluvium	Accelerograph	July 1932.
Pasadena:		
California Institute of Technology: Alluvium	Accelerograph and displacement meter.	May 1933.
Do	Weed seismograph (added).	June 1933.
San Bernardino:		
County Court House: Alluvium	Weed seismograph	Do.
San Diego:		
Consolidated Gas & Electric Co.: Alluvium	Accelerograph	July 1932.
Santa Ana:		
County Court House: Alluvium	Weed seismograph	June 1933.
Santa Barbara:		
County Court House: Alluvium	Accelerograph	Do.
Westwood:		
University of California at Los Angeles: Alluvium	do	Do.

[1] The accelerograph on the eleventh floor of the Alexander Building has not been installed to date, but has been retained temporarily at Washington as a model for improvements which seem desirable. It is expected that it will be installed in the near future. This instrument will be at the nodal point for the second mode of vibration, and should give very valuable data on the action of this building during an earthquake.

[2] The instrument on the Pacific Electric Co. lot in Hollywood is in a separate small building several hundred feet from the Hollywood Storage Co. Building, and should provide data which will be free from vibrations set up by the building itself. It is connected with the two accelerographs in the Hollywood Storage Co., making a set of three instruments in one locality operating under different conditions. They are connected electrically for simultaneous starting and time marking.

TABLE 2.—*List of shocks recorded on strong-motion seismographs*

Date, epicenter, and recording station	Records: Accelerograph	Records: Displacement meter	Records: Weed seismograph
Dec. 20, 1932: Western Nevada:			
Long Beach	1		
Southern Pacific Building, San Francisco			1
Mar. 10, 1933: Long Beach earthquake:			
Long Beach	1		
Vernon	1		
Los Angeles Subway Terminal	1		
May 16, 1933: Southern Alameda County:			
S. P. R. R. bridge over Suisun Bay	1		
June 25, 1933: Western Nevada:			
San Jose, Bank of America	2		
S. P. R. R. Building, San Francisco	1	1	
Sacramento	1		
Shell Building, San Francisco			2
S. P. R. R. bridge over Suisun Bay	1		
Oct. 2, 1933: Signal Hill, Calif.:			
Long Beach	1		
Hollywood Storage Co. Building	2		
Los Angeles Subway Terminal	1	1	
University of California in Los Angeles	1		
Vernon	1		
California Institute of Technology	1	1	
Santa Ana			1
Los Angeles Chamber of Commerce			1
Jan. 30, 1934: Western Nevada:			
San Jose, Bank of America	2		
Bishop	1		
June 7, 1934: Temblor Mountains, Calif.:			
Santa Barbara	1		
Hollywood Storage Co. Building	2		
California Institute of Technology	1	1	
July 6, 1934: At sea, northwest of Eureka, Calif.:			
Ferndale	1		
Eureka	1	1	
July 17, 1934: Chiriqui Province, Panama:			
Balboa Heights	1		
Oct. 2, 1934: San Francisco Bay region:			
Golden Gate Park	1		
Oakland City Hall	1		
Oct. 15, 1934: Imperial Valley:			
El Centro	1		
Dec. 30, 1934 (5:52 a. m.): Lower California:			
El Centro	1		
San Diego	1		
Los Angeles Subway Terminal	2	1	
Hollywood Storage Co. Building and adjoining lot	3		
Dec. 30, 1934 (5:55 a.m.): Near Santa Cruz:			
San Jose, Bank of America	2		
Dec. 31, 1934:[1] Lower California:			
San Diego	1		
Long Beach	1		
Los Angeles Chamber of Commerce	1		
Vernon	1		
Los Angeles Subway Terminal	2		
Edison Building, Los Angeles	1		
University of California in Los Angeles	1		
Hollywood Storage Co. Building and adjoining lot	3		
Jan. 2, 1935: Off northern coast of California:			
Eureka	1	1	
Mar. 3, 1935: Off northern coast of California:			
Eureka	1	1	
Ferndale	1		
Total number of records	53	8	5

[1] In cases where there is confusion as to whether a record was made on Dec. 30 or Dec. 31, it is assigned to the stronger shock of Dec. 31.

TILTMETERS

A line of development which has been carried on in Japan, and on which Suyehiro has laid some emphasis, is the measurement of earth tilt as an indication of what may be going on. Several types of tiltmeter have been developed and used in Japan for this purpose. Mr. George E. Merritt, of the Bureau of Standards, has developed tiltmeters using the principle of interferometry. A number of these instruments have been built and tested at Washington, and four of them were installed in the spring of 1933 by the United States Coast and Geodetic Survey on the campus of the University of California, near the Hayward fault. The locations of the tiltmeters and the approximate location of the Hayward fault are shown in figure 25. The sensitivity of the instruments is such that a change of one fringe between crosshairs represents a 1-second change in tilt for instruments nos. 1, 2, and 3, and one-half second for instrument no. 4.

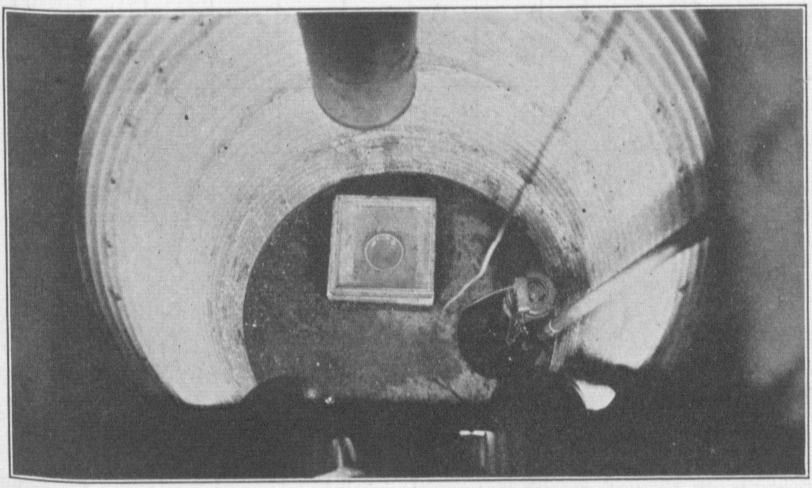


FIGURE 21.—Tiltmeter pit, showing end of telescope and basin of oil on concrete slab at bottom of pit.

The daily readings consist of counts of the fringes between crosshairs in both directions. Figure 21 shows the tiltmeter installation.

The tiltmeter readings have been plotted to show variation in tilt in figures 22 to 24. Considerable fluctuations have been caused by meteorological changes. Rainwater seeping into the tiltmeter pits has caused much trouble and in a number of cases has put the tiltmeters out of operation. Tiltmeter no. 4 is least susceptible to meteorological changes, and, hence, should be accorded more weight than the others.

In figures 22 to 24, the rainfall is shown as well as earthquakes that were felt at Berkeley during this period. Probably the only shock on the Hayward fault occurred on November 5, 1934.

During the time that these observations were made, tiltmeter no. 4 remained quite stable. During 1934 tiltmeter no. 2 recorded considerable fluctuations in tilt for which no definite cause is known. Our knowledge on this subject is evidently still very imperfect.

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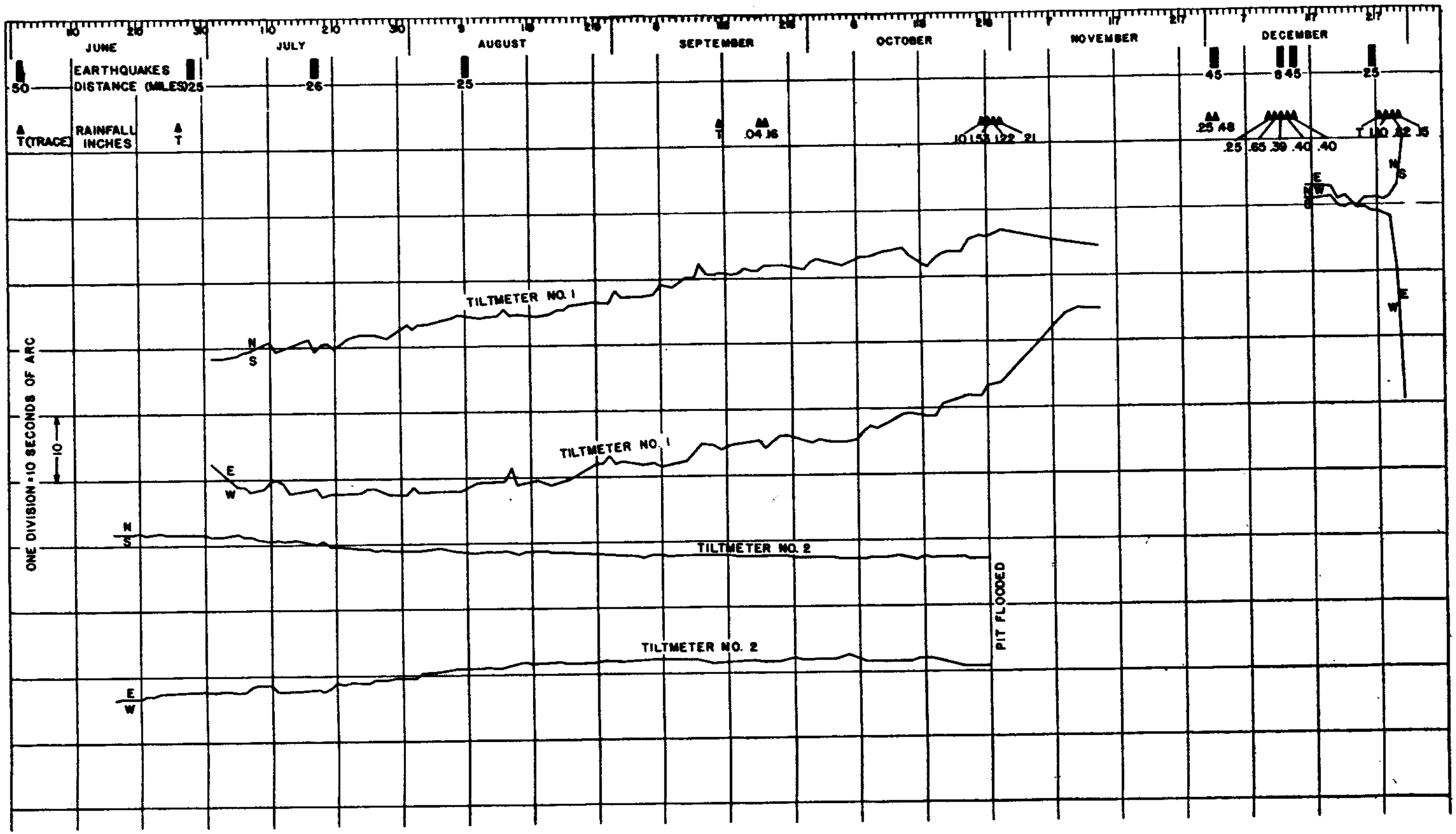


JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
EARTHQUAKES
DISTANCE (MILES)
RAINFALL INCHES
T (TRACE)
TILTMETER NO. 1
TILTMETER NO. 1
TILTMETER NO. 2
TILTMETER NO. 2
PIT FLOODED
ONE DIVISION = 10 SECONDS OF ARC
N
S
E
W

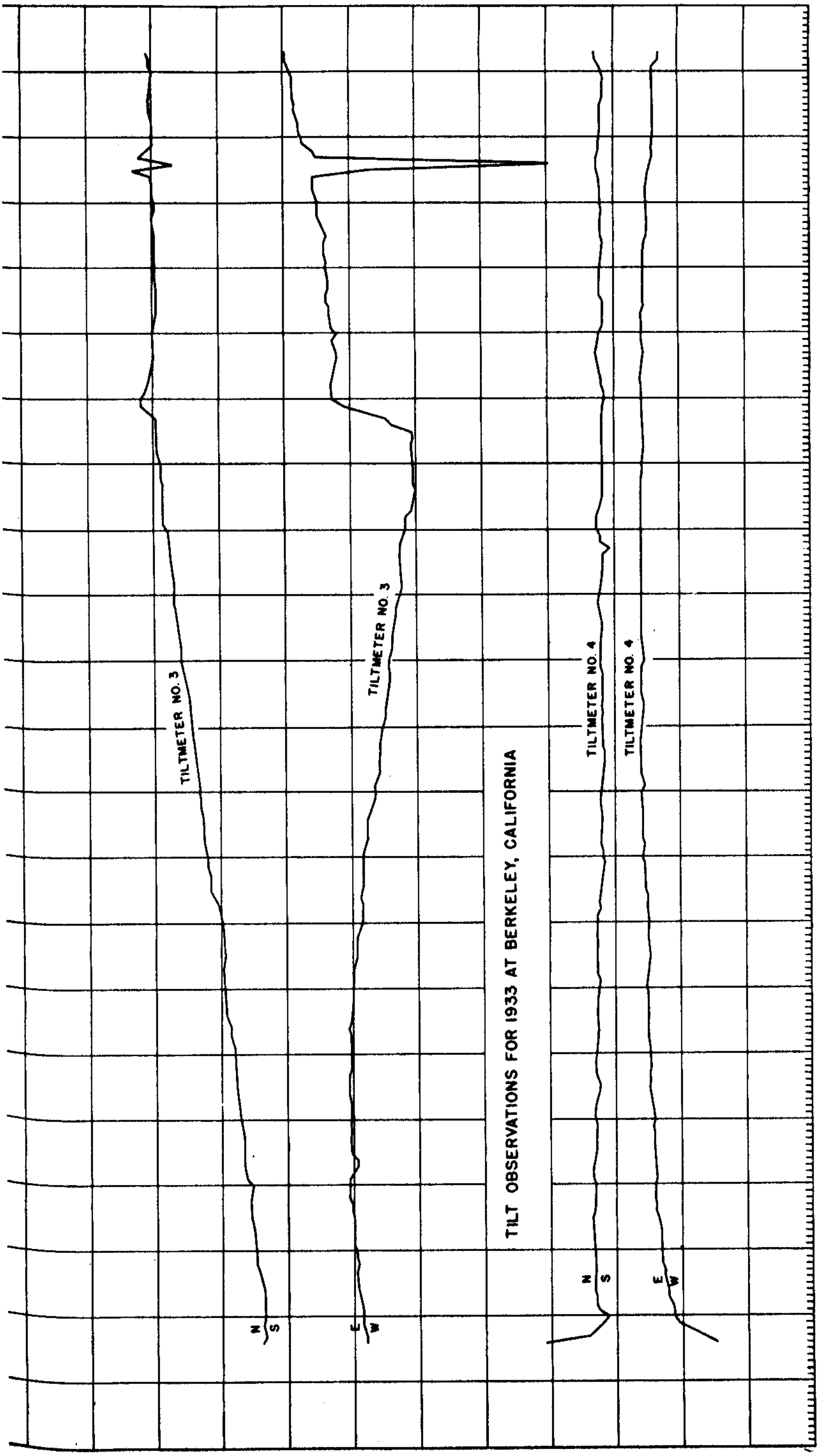
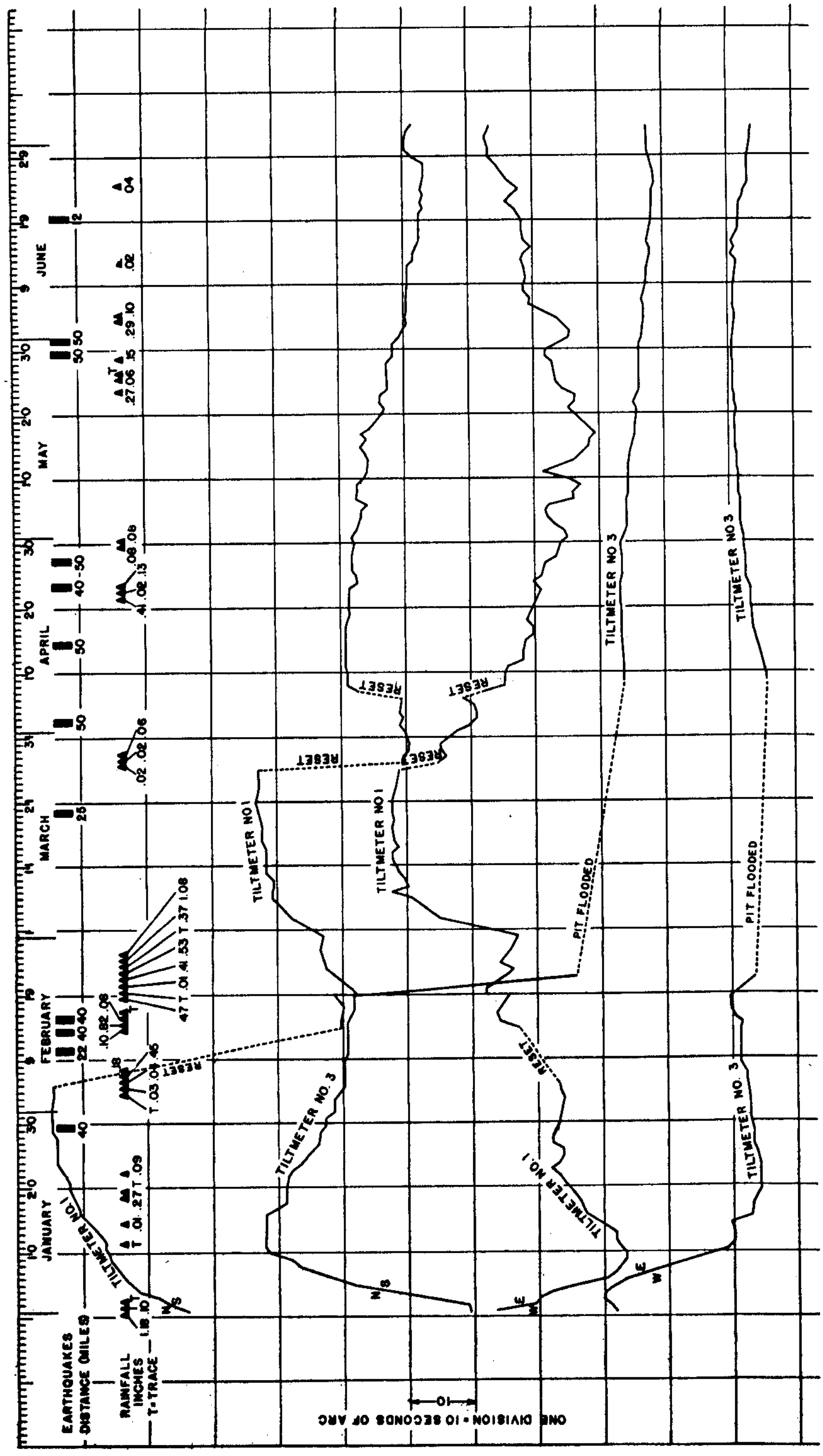


FIGURE 22.—Tilt curves, 1933.


EARTHQUAKES
DISTANCE (MILES)
RAINFALL
INCHES
T=TRACE
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
TILTMETER NO. 1
TILTMETER NO. 3
PIT FLOODED
RESET
N
S
E
W
ONE DIVISION = 10 SECONDS OF ARC

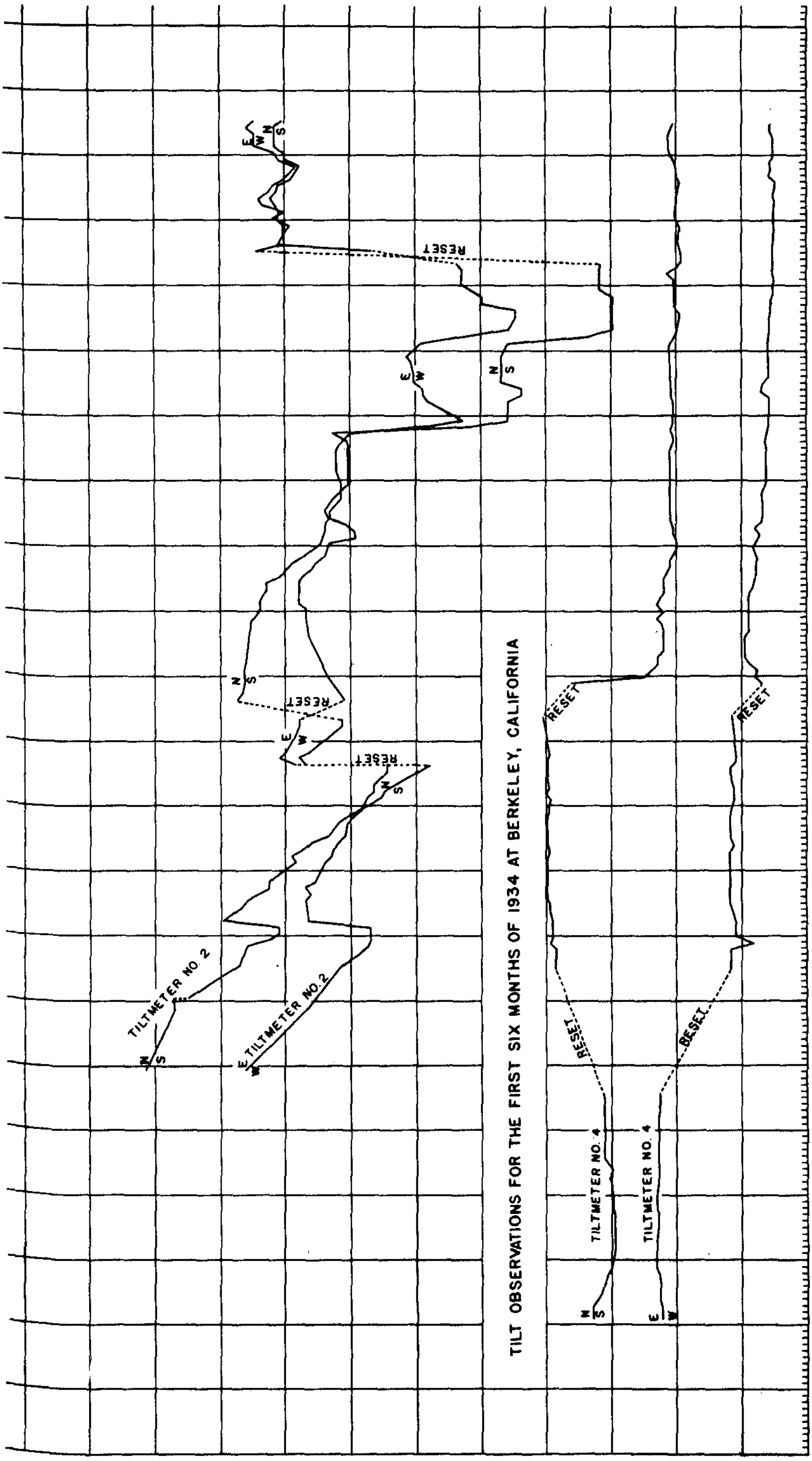
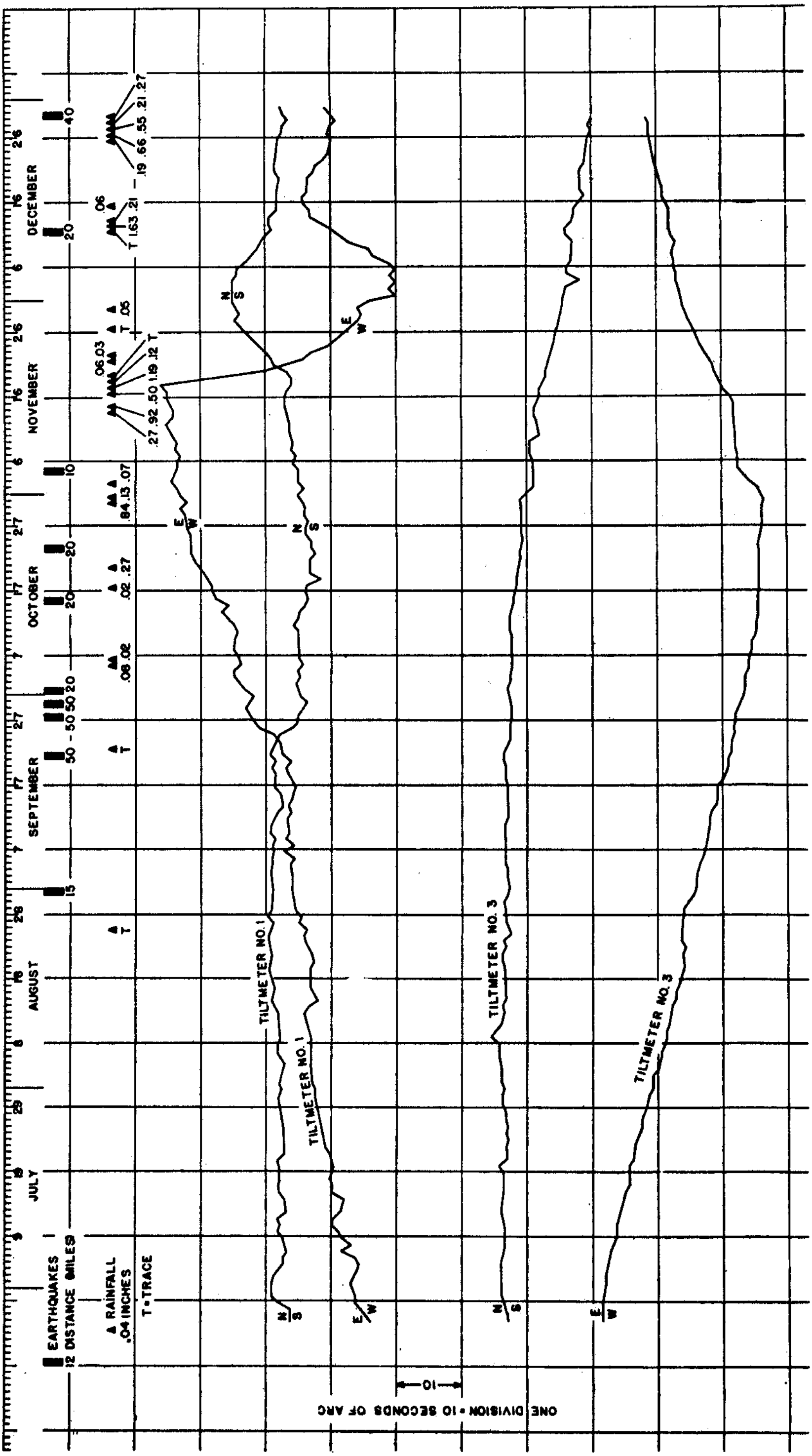


FIGURE 23.—Tilt curves, first 6 months of 1934.


EARTHQUAKES
12 DISTANCE (MILES)
RAINFALL .04 INCHES
T = TRACE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
TILTMETER NO. 1
TILTMETER NO. 3
ONE DIVISION = 10 SECONDS OF ARC

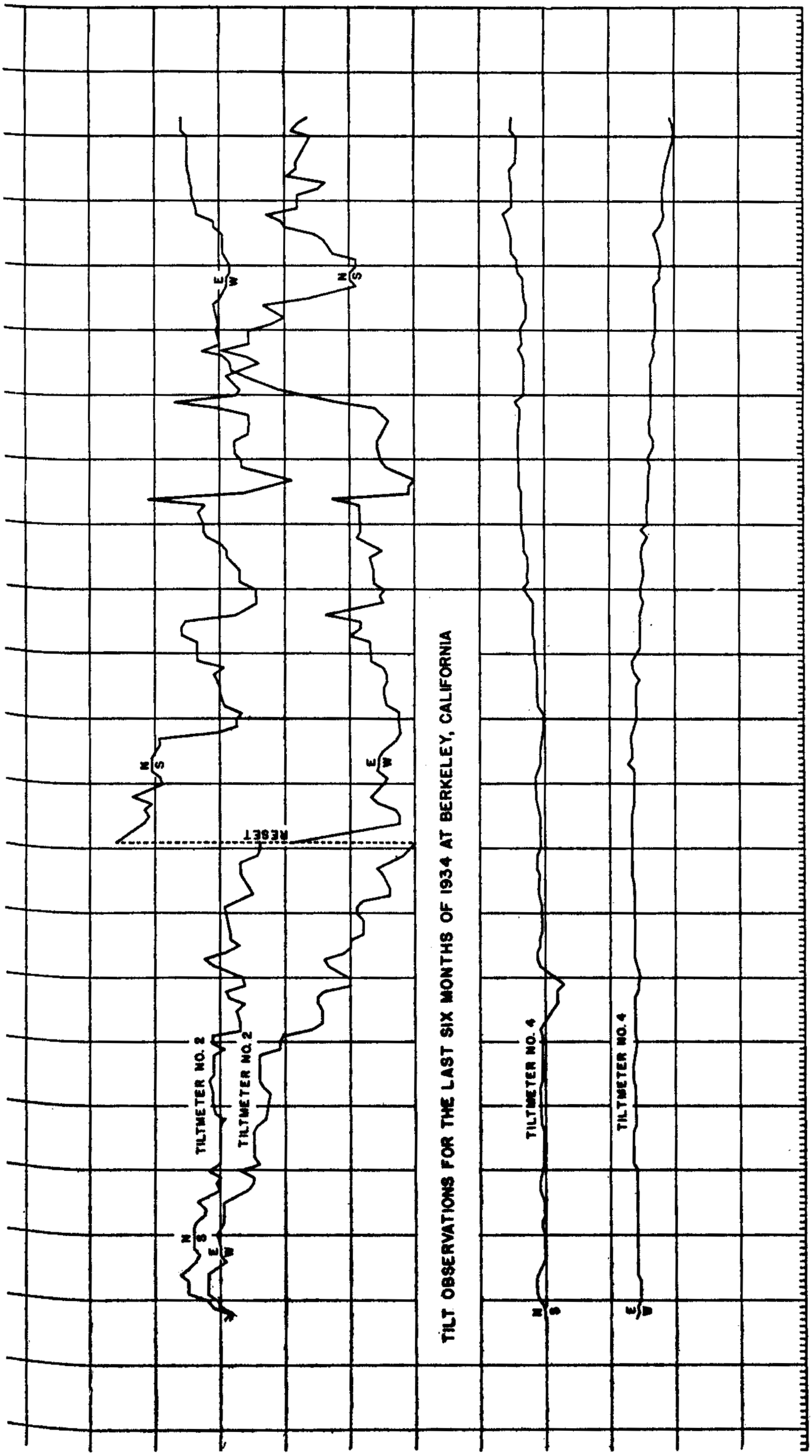


FIGURE 24.—Tilt curves, last 6 months of 1934.

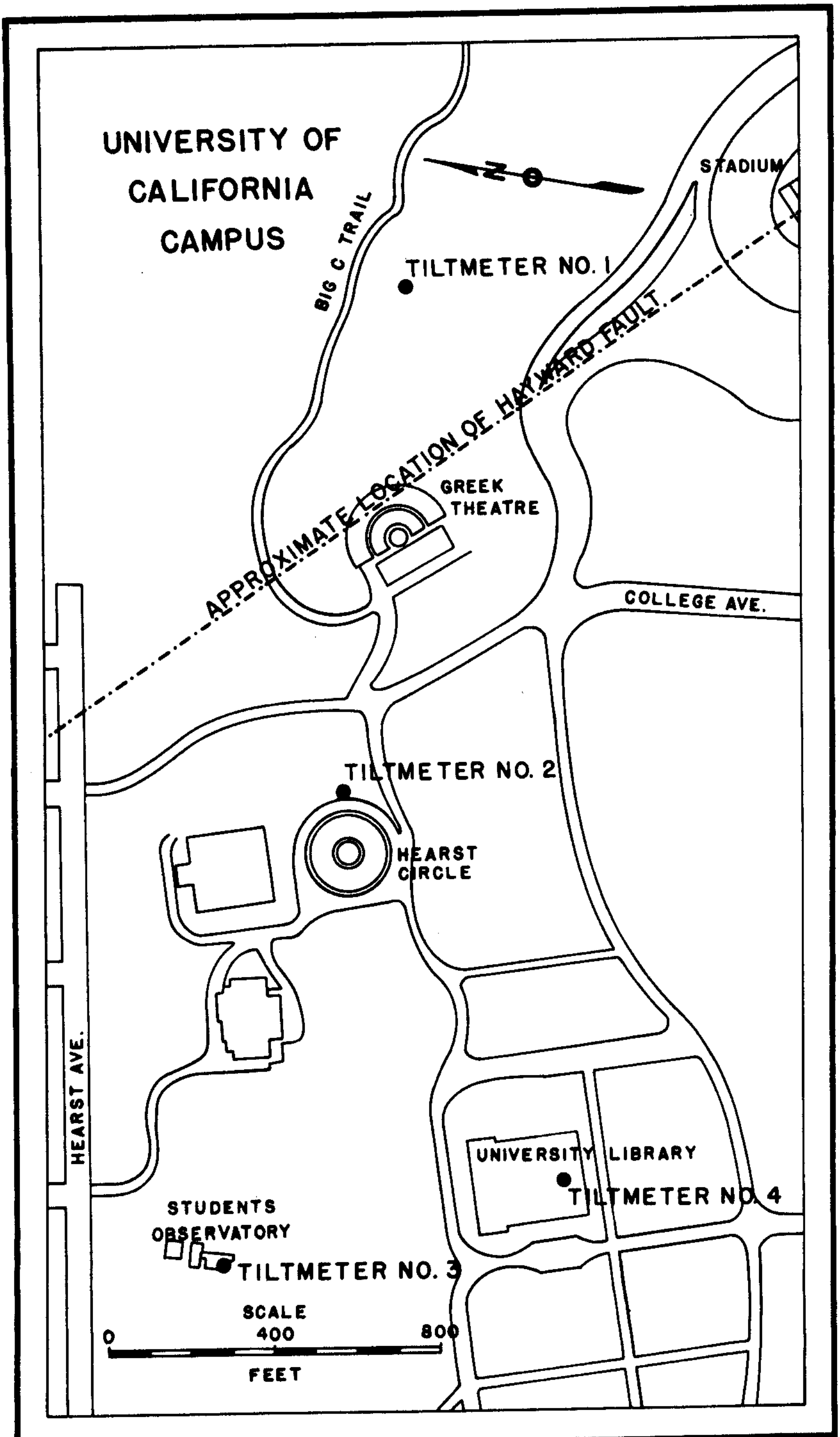


FIGURE 25.—University of California campus.

Chapter 3.—THE ANALYSIS OF RECORDS

F. NEUMANN

Publication of data.—Records obtained from the Coast and Geodetic Survey accelerograph, displacement-meter, and strong-motion seismograph installations in California are analyzed in the Washington office. The results of analyses already completed are published in Serial No. 579, entitled "United States Earthquakes, 1933"; in a mimeographed report entitled "Analysis of Strong Motion Seismograph Records of the Western Nevada Earthquake of January 30, 1934, with Description of a Method of Analyzing Seismograms by Precise Integration"; and in monthly mimeographed statements entitled "Progress in Strong Motion Earthquake Work in California and Elsewhere." Three preliminary reports, covering the earthquakes of March 10, May 16, June 25, and October 2, 1933, are reproduced in Serial 579 just mentioned.

Analyses of vibration records are made by the parties carrying out the various vibration projects in California. That work is discussed in other chapters of this report, but complete results of the work are published only in mimeographed pamphlets which are available at the San Francisco Field Station of the Coast and Geodetic Survey, Customhouse, San Francisco, and from the Director, Coast and Geodetic Survey, Washington.

Recording characteristics of seismographs.—It is not intended to discuss in detail the methods used in analyzing strong motion and vibration records, but it is believed that a broad statement of the principles involved will give a better idea of the possibilities and limitations of seismographic methods. The primary purpose of the instrumental equipment is to measure the motion of the ground, building, or other structure with a pendulum device known as a seismograph. Any object capable of oscillating in the manner of a pendulum is forcibly set into motion if its support moves to and fro, but the motion of the object will necessarily be different from that of its support. The seismograph records this differential movement, and from it it is possible to compute the motion of the instrument foundation, whether it is rhythmic in character, or otherwise. In practice, however, there are important limitations.

First, such a large range of ground and building motion is covered in seismographic measurements that different instruments with varying degrees of sensitivity are necessary to record them all. The double magnification device recently placed on a number of accelerographs in California is a notable step forward because it increases the range of the instrument's usefulness with respect to the number of shocks capable of being satisfactorily recorded, and to the range of amplitudes which can be recorded during a destructive shock. Sensitivity, however, involves not only the so-called *static* or lever magnification of a seismograph, but also the pendulum period, or more precisely, the ratio of the pendulum period to the period of the earth movement, if the motion is of periodic type.

This may be illustrated as follows. Take a plumb line about a foot or two long with a rather heavy bob, and suspend it by one hand. If the hand is moved rapidly through small horizontal amplitudes, the inertia of the bob keeps it practically fixed in space; and if the relative motion between the hand and the bob could be measured, it would be very close to the motion of the hand in space. Now go to the other extreme. Move the hand to and fro very slowly and observe how closely the bob follows the motion of the hand. There is very little differential movement, and if a record could be made of it, the relative motion would be found very small as compared with the motion of the hand in space. This latter case is analogous to that of a long period earth wave (corresponding to the slow motion of the hand) and a relatively short period seismograph pendulum (corresponding to the plumb line). Obviously if the pendulum period is too short, practically no record whatever will be obtained, and this becomes a limiting condition in the satisfactory recording of ground and structural movements.

The harmonic magnification curve.—The ratio between (1) the motion of a pendulum (with respect to its moving support) and (2) the

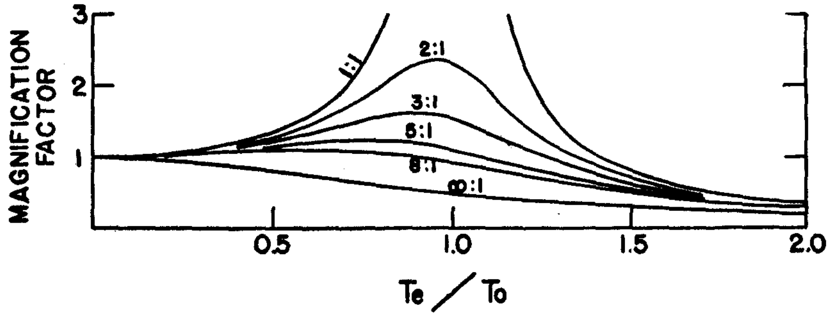


FIGURE 26.—Harmonic magnification curves.

motion of the support itself (corresponding to the motion of the ground or structure) is of fundamental importance in seismogram analysis and can be shown in the form of so-called "harmonic magnification curves." See figure 26.

The relations shown in these curves hold only when the motion impressed on the instrument is of simple harmonic character. The ordinates representing harmonic magnification are expressed in terms of unity, that is, the actual magnification which would be obtained if no magnifying levers were attached to the pendulum and the record were made by a writing element fixed beneath the center of oscillation of the pendulum. If magnifying levers amplify the motion of the center of oscillation, say 100 times, then all ordinates should be multiplied by that factor. The term T_e is the period of the earth wave (time of one complete cycle), which is read directly from the seismogram because the seismograph pendulum is forced to oscillate in the period of the ground wave regardless of its own natural period. T_0 is the natural period of the seismograph pendulum. The shape of the curve depends on the "damping ratio" of the pendulum; that is, the ratio of successive amplitudes of the pendulum after a frictional element, such as oil, or an electromagnetic resistance, has been introduced into the pendulum system to prevent resonance, or "tuning-in" effects.

The equation of the harmonic magnification curve is derived from the differential equation of motion given on page 35, and may be written in the form:

$$M_h=\frac{1}{\sqrt{\left[\left(\frac{T_e}{T_o}\right)^2+1\right]^2+4\left(\frac{T_e}{T_o}\right)^2(h^2-1)}}$$

in which the damping constant h is obtained from the damping ratio ϵ by the formula

$$h=\frac{\log_{10}\epsilon}{\sqrt{1.861+(\log_{10}\epsilon)^2}}$$

Of special interest is the fact that after $\frac{T_e}{T_o}$ becomes numerically greater than about 3 or 4, the ordinates of the curve may be expressed with sufficient accuracy by $\frac{T_e^2}{T_o^2}$, meaning that the amplitude of any impressed displacement or ground movement is minified in that ratio on the seismogram. Substituting this in the well-known expression for acceleration in simple harmonic motion, we have:

$$a=\frac{4\pi^2A}{T_e^2}=\frac{4\pi^2A_t}{T_e^2}\times\frac{T_e^2}{T_o^2}=\frac{4\pi^2A_t}{T_o^2}$$

in which a is acceleration, A amplitude of ground motion, and A_t trace amplitude. If the lever magnification of the seismograph pendulum is V, then V should be placed in the denominator with T_o^2. In this equation it should be noticed that acceleration depends only on the trace amplitude, not on the period, and the instrument therefore functions as an accelerometer. This holds as long as the earth wave period is more than three or four times the pendulum period.

When the impressed periods are less than about one-third the pendulum period it is seen from the curve that the harmonic magnifications are nearly constant, and the seismograph therefore functions as a displacement meter. The range of periods over which this occurs varies greatly with the degree of pendulum damping, and may be much greater than just indicated. When the impressed periods are close to the pendulum period the seismograph records neither displacement nor acceleration directly.

Sensitivity.—Sensitivity when applied to seismographs means sensitivity to acceleration. In very sensitive seismographs large deflections of the recording spot are obtained when the horizontal pendulum is tilted sideways only 1 second of arc, which corresponds to an acceleration of gravity times sin 1″. In the case of very insensitive pendulums (of necessarily short period) they may be turned sideways through 90° so that the full acceleration of gravity is effective in deflecting them. Sensitivity is thus determined directly by subjecting a pendulum to a part of, or the full acceleration of gravity.

Changes in acceleration resulting from changes in tilt are registered quite faithfully by a pendulum as long as they are relatively slow, or of long period, as compared with the pendulum period. This is analogous to the response of a pendulum to purely horizontal acceleration of simple harmonic character as revealed by the harmonic magnification curves, and explains why a short period pendulum can function as an accelerometer for relatively slow irregular motions as well as for slow simple harmonic motion.

The following relation between pendulum period and sensitivity holds true for any pendulum seismometer:

$$\frac{S_a}{T_o^2}=\frac{Va}{4\pi^2}=\text{Constant}$$

in which S_a is the deflection of the seismograph writing point for any arbitrary acceleration a.

Analyses based on the assumption of simple harmonic motion.—As previously stated the key to the analysis of seismograms is the harmonic magnification curve. If the records are of displacement-meter type, or of accelerograph type, the analysis is somewhat simplified, but it must not be forgotten that, after all, such records simply represent conditions at extreme ends of the magnification curve. On displacement meter records the ordinates are directly proportional to displacement as long as the recorded periods are small compared with the pendulum period, say less than one-third, and within these limits the record represents true displacement whether the waves are of simple harmonic character or not. Acceleration can be reliably estimated from a displacement-meter record only when the recorded motion is of simple harmonic character. In that event, the period and amplitude are read directly from the gram, and acceleration can be computed from the well-known expression $a=4\pi^2A/T_e^2$. But such an analysis cannot be accepted as a substitute for an accelerogram because the chances are that some of the most important accelerations in the movement are so poorly defined on the record that they will escape detection. The higher accelerations are always associated with short periods and relatively small displacements but on a displacement-meter record they appear insignificant with respect to the longer period waves. This is quite apparent from any displacement record such as that of figure 19 or the computed curves of figure 27. These facts restrict the field of usefulness of the displacement meter to studies of long period waves only.

When the recorded waves are close to the pendulum period the trace amplitudes represent neither displacement nor acceleration. It is then necessary, if the motion is of simple harmonic type, to go to the magnification curve and measure the harmonic magnification factor from the ratio of the ground and pendulum periods. It is usually less than unity for satisfactorily damped pendulums. This, multiplied by the static magnification of the seismograph, gives the resultant magnification of all impressed waves of the ground period measured. The trace amplitude divided by the resultant magnification gives the displacement of the ground. Acceleration may then be computed from the usual simple harmonic acceleration formula given in the preceding paragraph. In this range of periods neither displacement nor acceleration can be computed precisely, but estimates of displacements are more reliable for the shorter earth-wave periods; for the longer earth-wave periods acceleration is the more reliable. Here again the question of how closely the recorded waves approximate simple harmonic motion enters largely into the value of the results, but not as much as when estimating acceleration from a displacement meter record or displacement from an accelerometer record.

When the seismograph functions purely as an accelerograph, acceleration can be read directly from the seismogram when the sensitivity, or scale, is known. Displacement of the short period waves can be quite accurately computed because period and acceleration are known, and such waves are usually clearly defined on the record. The trace amplitudes of the longer period waves, however, grow smaller so rapidly with increasing period that quite frequently movements of major importance are almost completely masked by the short period waves. For these reasons it is next to impossible to estimate the displacement of long period waves from an accelerogram, especially when motion is not clearly of simple harmonic character. The acceleration is obviously very small under such circumstances.

Analysis by integration.—If a record is important enough to justify extended analysis a second method of treatment may be resorted to, namely, integration. Analysis by integration is based on the fundamental equation of motion of a damped pendulum when subjected to an external acceleration:

$$\ddot{x}=\ddot{y}+2k\dot{y}+p^2y$$

in which x is the instantaneous displacement of the instrument foundation, y the instantaneous displacement of the pendulum relative to the instantaneous position of the foundation (measured directly from the record), k a damping constant, and p^2 an instrument period factor. By integrating the expression two times, successive values of x, the ground displacement, may be obtained as follows:

$$x=y+2k\int_0^t y\,dt+p^2\int_0^t\int_0^t y\,dt\,dt+C_1+C_2t$$

This is a laborious computation, and satisfactory results may be expected only when the records are obtained from high grade seismographs and the integration processes carried through with special equipment. The differential equation for displacement is of such fundamental character that comment on the various terms will not be superfluous. It expresses quantitatively the response of a damped pendulum to any motion impressed on its support, and discussion of it will bring out in detail to just what degree the period of a pendulum controls the character of a record as already evidenced in the case of simple harmonic motion. The theory involved in analysis by integration is not new but its successful application is most difficult because of the high degree of precision required in making measurements on the seismographic record, the fact that the zero line of a record is usually more or less indefinite for such precise work, and the fact that the determination of the constants is a problem in itself, the solution depending largely upon the nature of the record to be analyzed

The three terms containing y have different relative weights in the solution of a record depending largely upon the ratio between the ground-wave periods and the pendulum period of the seismograph writing the records. If the ground-wave periods are short compared to the pendulum period, the two terms containing integrals are insignificant. Then y is by far the most significant term and, under the period conditions just named, the instrument functions as a displacement meter. This is perfectly analogous to the case when the ground movements are assumed to be of simple harmonic character.

The term containing the single integral is most significant when the pendulum and ground periods are about equal, but even then the other y terms cannot be neglected. The instrument functions more than anything else as a velocity meter. The only way to make a true velocity meter out of the pendulum for a resaonable range of ground periods is to make the k factor so large (by overdamping the pendulum) that the remaining terms will have but little relative weight. In practice, however, the lever or static magnification of such a seismograph would need to be greatly increased. The advantage of a velocity record would be that only a single (not double) integration, and a single differentiation of the observed curve would be required to reduce it to terms of displacement and acceleration, respectively. When the double process is undertaken errors of measurement are likely to enter so largely into the results that it is generally considered impracticable, although reasonably satisfactory results have been obtained by the use of precise methods (especially for integration) in the office of the Coast and Geodetic Survey. Good results in the application of the equation to records of so-called intermediate period pendulums have been obtained by quite a number of investigators, but in most work the double integral term plays only a secondary part so that, although appreciable error may enter into the double integration term, it is largely masked by the greater weight of the other terms involving only one integration or none at all.

The term containing the double integral is most significant when pendulum periods are short compared to ground wave periods. When all of the ground wave periods are three or four times as large as the pendulum period, the instrument functions as an accelerometer and the other y terms may be neglected so far as practical results are concerned. This is a most important case so far as the California program is concerned because most of the strong motion instruments are of the accelerometer type. The pendulum periods are 0.1 second so that the accelerographs give a continuous and direct measure of acceleration as long as the ground wave periods or building vibration periods are more than 0.3 or 0.4 second. If they are under these values, acceleration and displacement must be computed on the assumption of simple harmonic motion, or by application of all three y terms in the differential equation of motion. In the critical analysis of an accelerograph record by integration, however, it has been found unnecessary to apply the first two y terms because (1) the resulting amplitudes of the ultrashort period waves are always insignificant compared to those of longer period; and (2) the wave forms of the shorter period waves are shown to better advantage on an accelerogram than on any other type of record, their displacements being estimated quite accurately by assuming simple harmonic motion. Another practical reason for ignoring wave periods under 0.3 second in integrating accelerograph records in which the elemental ordinates are scaled, is that the number of readings would have to be doubled or tripled to obtain a sufficient number of points to define the waves, adding greatly to the laboriousness of the task. This is, of course, not such a great consideration when integration machine methods are employed. Numerical methods of integration can always be carried out to greater accuracy than is obtainable with an integraph, but it may nevertheless be true that mechanical methods will suffice in parts of the work. This is a matter for further study.

The constants of integration in the right-hand member of the equation depend on the initial position of the pendulum and are troublesome factors which can be determined best by trial when it is known that the record does not begin with the pendulum at rest. A method of adjustment is explained in strong motion report no. 4 mentioned in the first paragraph of this chapter. Another difficult problem is the determination of the true axis of the recorded curve throughout its entire length. With this in mind, and the desirability of keeping all measured distances on the seismogram as small as possible to escape paper distortion effects, special base lines have been placed on the records of some of the newer installations so that the average distance

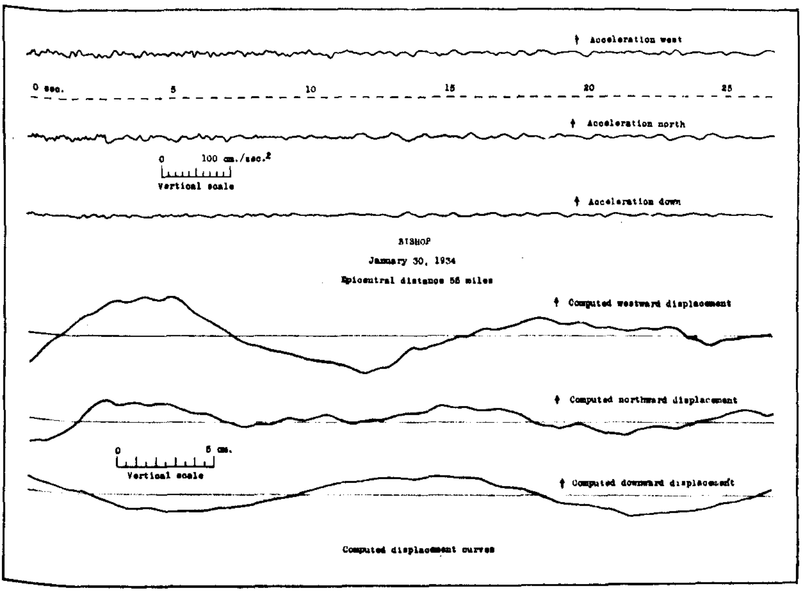


FIGURE 27.—Acceleration and computed displacement curves, earthquake of January 30, 1934.

from base line to the normal trace position will never be more than 1 or 2 centimeters.

Three instrumental constants, pendulum period, damping, and magnification, are involved in the reduction of all records, whether by integration or the assumption of simple harmonic motion. The damping factor depends on an instrumental adjustment, and is most important in the case of the intermediate type of pendulum. It affects accelerograph results appreciably only around the lower range of periods in which the instrument continues to function as an accelerometer. A satisfactory adjustment of damping will enable the accelerometer to function very closely as such for ground periods as low as 0.2 second instead of the customary 0.3 or 0.4 second, but it requires much care to maintain such precise damping adjustment. Examples of integration results as applied to accelerograms are shown in figures 27, 28, and 29.

In view of the results already obtained in analysis by integration there is reason to believe that some very thorough analyses of building

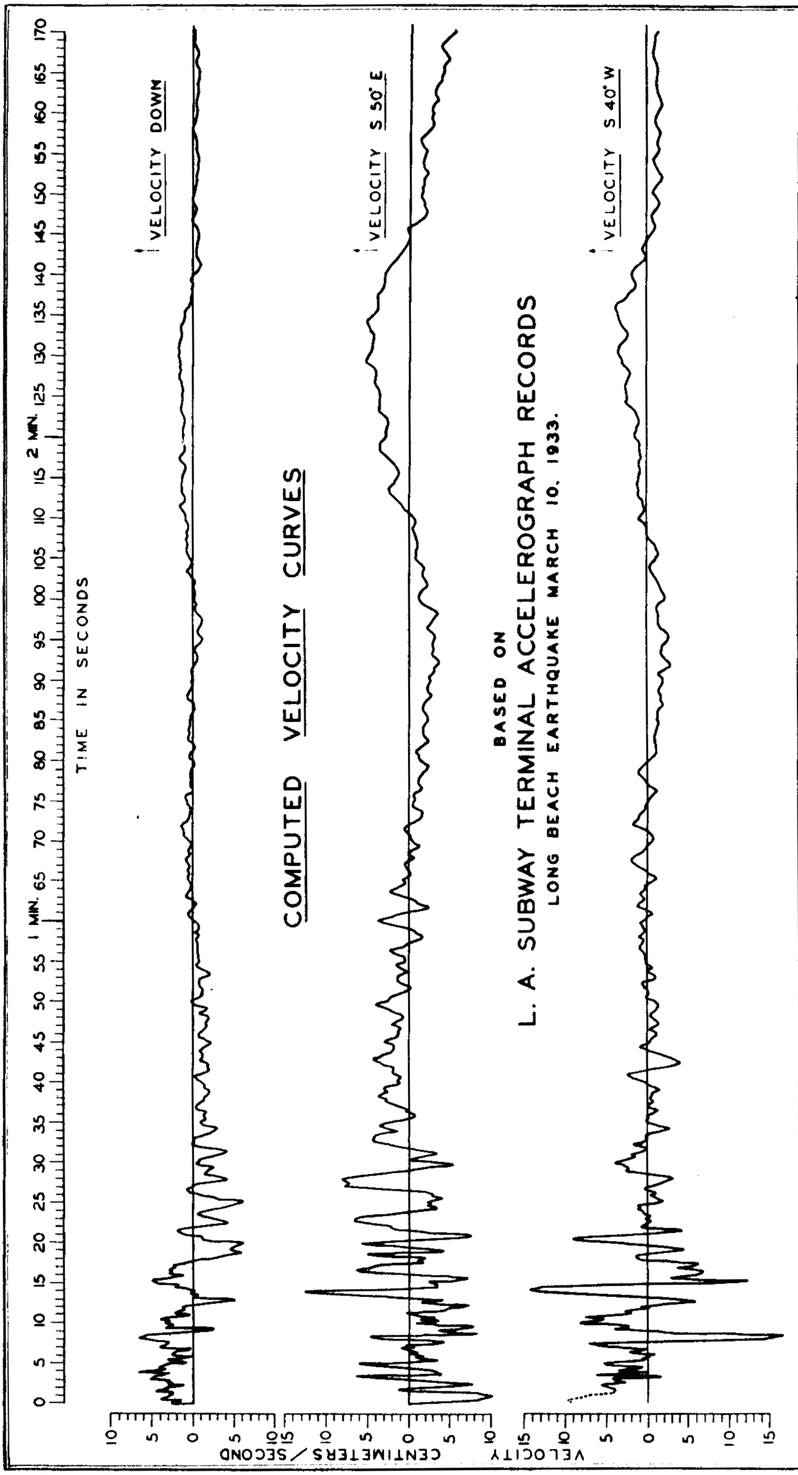


FIGURE 28.—Computed velocity curves, Los Angeles subway terminal record, Long Beach earthquake. See upper half of figure 16 for active portion of original accelerogram.

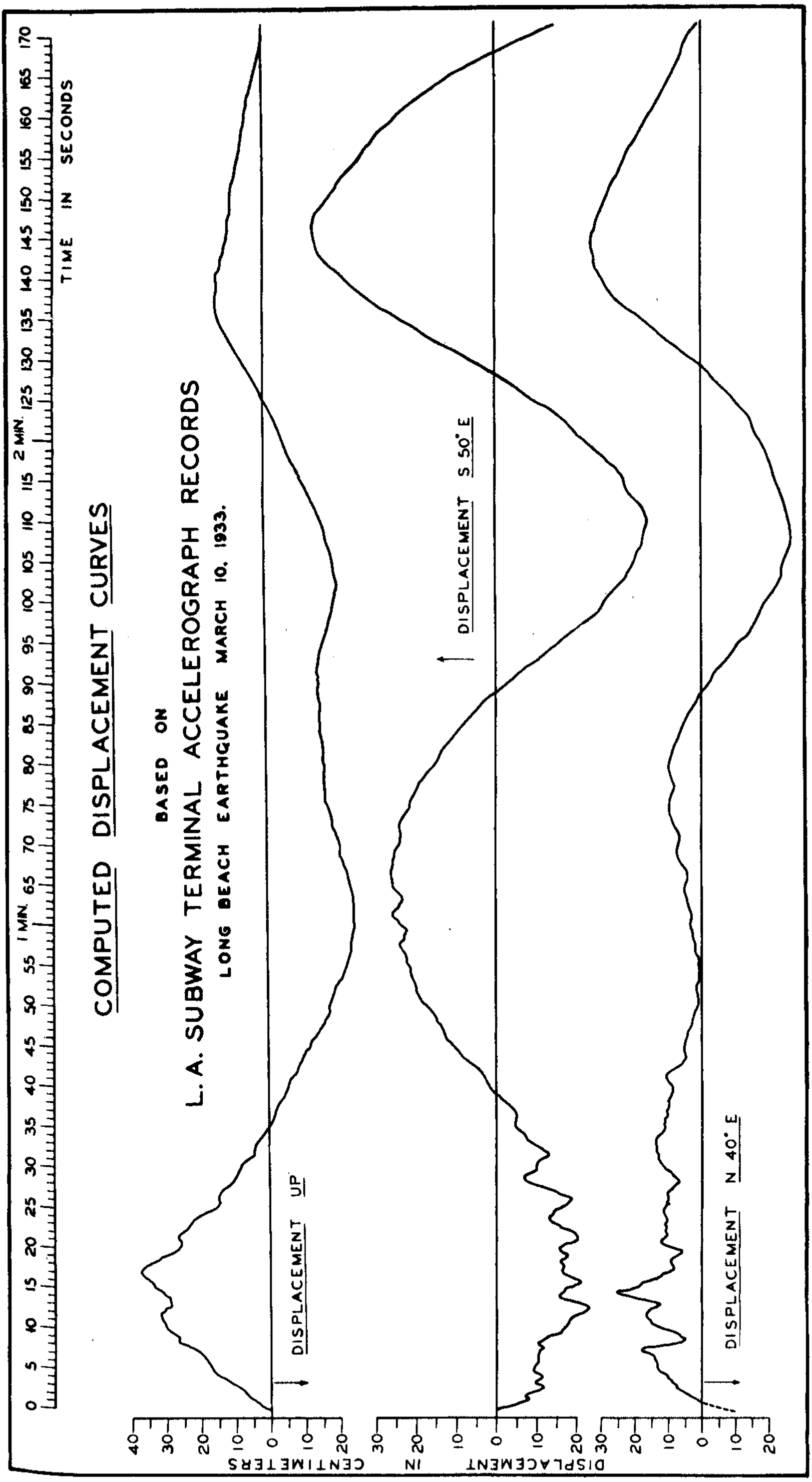


FIGURE 29.—Computed displacement curves, Los Angeles subway terminal record, Long Beach earthquake. See upper half of figure 16 for active portion of original accelerogram.

movements during strong earthquakes will be forthcoming from accelerograph records.

Period analysis.—The engineer is interested not only in the acceleration and displacement of all measured movements but also in the various periods, or frequencies, contained in a given record. The period characteristics of seismic ground movements are of fundamental importance in estimating the distortion to which a structure may be subjected; and the problem is greatly simplified if the seismograph record can be translated into terms of simple harmonic motion. While it is possible, from the theoretical viewpoint, to take a curve of irregular character and break it up into a number of elementary curves of simple harmonic character the usefulness of this Fourier analysis is very limited in the case of seismographic records. Fortunately, in most records certain periods stand out so clearly that they can be read simply by inspection. No computation is involved because every pendulum, regardless of its own period, is forced to oscillate in the period of the impressed vibration; but there may be great differences in phase, and in the relative trace amplitudes of the various periods regardless of actual ground motion. The best the seismologist can do, therefore, is to read off by inspection the periods of those movements which appear to be of simple harmonic character; but the range of visibly recorded periods will depend largely upon the period of the seismometer. Short-period pendulums accentuate short-period waves; long-period pendulums accentuate long-period waves. In accord with this we find accelerometers recording short-period waves best, while displacement meters, with 10 second pendulums, record long period waves best.

Neither a long-period nor a short-period pendulum can visibly record all seismic-wave periods, but it has been shown that the process of double-integrating accelerograph records brings out the entire spectrum of ground waves, the original record emphasizing the short-period waves, the first integration or velocity curve emphasizing waves of intermediate period, and the second integration or displacement curve emphasizing the long-period waves. Long-period waves actually are recorded by high-grade short-period pendulums but the amplitudes are so small (of the order of 0.1 mm in some recent records) that they can be detected only by precise methods of integration. It therefore appears that integration offers one of the most promising lines of attack in detecting hidden movements on accelerograms. Although a greater range of periods is made available for study there still remains the problem of measuring what may be called the elementary periods of the wave trains which produce the record. When it is realized, however, that the periods of even the simplest train of seismic waves change from beginning to end of the train, and that most seismic movements are the result of several trains superposed on each other, the difficulty of the problem becomes increasingly apparent if one hopes to analyze the period characteristics of a record by strictly mathematical methods. The outlook from the point of view of application to building and other structural vibrations is more hopeful because there the vibrations are of more uniform character.

Studies in the response of hypothetical structures.—It has just been stated that the problem of the engineer would be greatly simplified if observed ground movements could be adequately expressed in terms of simple harmonic motion, and also that this is more or less of a hopeless task if only the usual number of seismographic records are available. As the ultimate purpose of the engineer is to compute the response of structures to these motions, the question arises: Why not compute it directly from the observed ground motion using the differential equation of motion discussed elsewhere in this paper? The process would be in effect a reversal of the procedure used in calculating the motion of the ground from the motion of the pendulum. To make such computations it is necessary to know the period of the structure and the degree of damping, elements which must be measured or estimated beforehand. Unfortunately, the formula is very complex and the computation of such laborious character that its usefulness will depend largely upon how expeditiously the work can be done. Mr. A. Blake, of the Coast and Geodetic Survey staff,

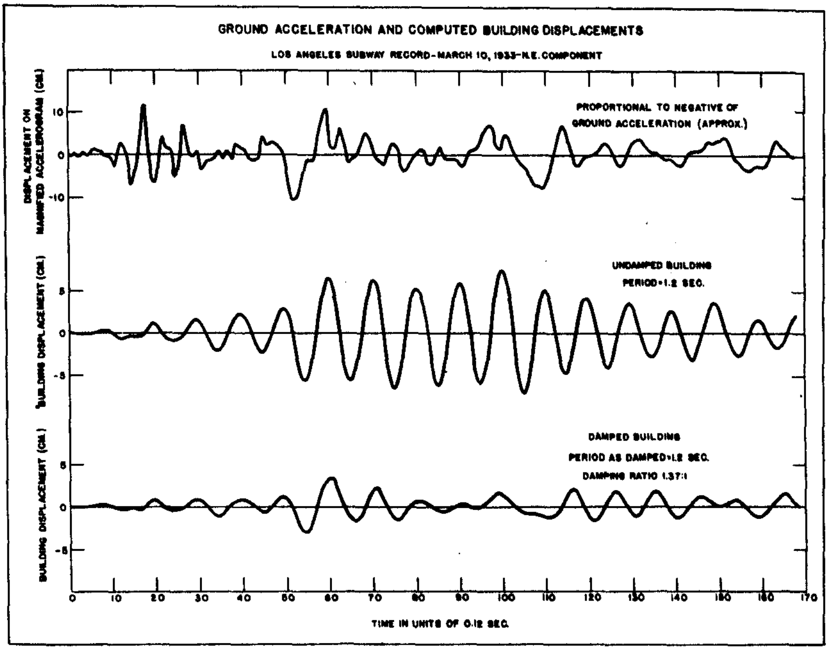


FIGURE 30.—Ground acceleration and computed building displacements.

has already succeeded in reducing the labor considerably. If the method proves practicable it will be possible to compute the response of representative types of structures to any observed ground movement; or the response of some specific structure to such movement. An example of such work is shown in figure 30.

An analysis of this kind corresponds to computing theoretically the response of a system of oscillators of varying period and damping to an impressed motion. A battery of pendulums of similarly variable character, a so-called "harmonic analyzer", would record similar results and it may be that the construction of such an instrument will be more economical than the cost of theoretical studies. It would register automatically at the time of an earthquake.

It is of interest to note that either type of study would aid greatly in detecting dominant ground-wave periods because of resonance effects regardless of whether or not the periods are clearly defined on the seismogram. Resonance will result when ground and undamped pendulum periods coincide and large amplitudes will appear on the response curve. For this reason the computation described would be a valuable tool in period investigation.

Some thought has been given the possibility of measuring the response of a pendulum by actually constructing one with the desired characteristics and imposing upon it the accelerations recorded on the strong-motion instruments by means of tilts, springs, or other forces which would vary directly with the acceleration. As page proof of this publication is given final revision, the author is afforded opportunity to state that a satisfactory solution of the problem has apparently been found in the properties of a simple torsion pendulum. It is based on the principle that if the suspension, or torsion head, of such a pendulum is turned through angles corresponding in magnitude to the instantaneous values of acceleration (as recorded on an accelerogram), then the mass of the torsion pendulum will be deflected through angles corresponding to the deflections of any vibrating system having similar period and damping characteristics. This means that if we have the acceleration record of a ground movement, it will obviate the necessity of building other types of pendulums such as displacement meters and harmonic analyzers. Their response can be readily determined with the torsion pendulum analyzer. This should also apply to the response of engineering structures. A great practical advantage of the torsion pendulum is that earthquake motions and effects can be reproduced in slow motion so that, in the apparatus developed by the author, analysis can be made by moving a tracing point manually over the accelerograph record at one or two hundredths of the speed of the recording drum of the accelerograph. It is obvious from the preceding paragraphs, which were written before the possibilities of the torsion pendulum were investigated, that the device should provide a marked impetus in the study of destructive ground movements.

In conclusion it is quite evident that some rather difficult problems confront the seismologist in his attempts to analyze seismographic records in a way that will be of maximum benefit to the structural engineer. Engineering precision is forcing new standards on the seismologist in both the instrumental and analytical fields and he must work up to them; but progress is being made and it is reasonable to believe that the prospects are brighter than ever before for the successful application of seismological principles to engineering problems.

Chapter 4.—THE QUESTIONNAIRE PROGRAM FOR COLLECTING EARTHQUAKE DATA

P. Byerly and H. Dyk

HISTORICAL

In 1925 the Coast and Geodetic Survey was authorized "to make investigations and reports in seismology, including such investigations as have been heretofore performed by the Weather Bureau."

Prior to 1929 the collecting of reports of felt earthquakes was confined to requests sent out after it was learned, by instrumental or other means, that an earthquake had occurred. This type of work was done principally in California by the University of California, at Berkeley. After the larger northern California earthquakes post-card questionnaires were sent into the region in which the shock was reported felt and the intensities rated from the answered questionnaires. This method yielded few reports unless they were specifically requested and numbers of smaller shocks were liable to pass unreported. Also there was some loss of time since the observer did not report immediately but only on receipt of a request sent after the earthquake.

After consultation with California seismologists it was decided that the Coast and Geodetic Survey could well undertake and expand the program for collecting information on earthquakes felt in the State. After careful consideration it was decided to use a post-card questionnaire form, sending copies to a number of observers who would keep them on hand and furnish reports immediately after the occurrence of a shock. A copy of the questionnaire is shown in figure 31. The wording of the form is based on the current earthquake intensity scales, the details sought being those which are the criteria of the scales. The general form of the card follows that used earlier by the University of California and still earlier by Mr. Maxwell Allen.

The earlier work had shown that postmasters were, in general, faithful reporters. Therefore, the first step of the Coast and Geodetic Survey was to place a supply of questionnaries with each postmaster in California with the request that he report all earthquakes felt in his community. The distribution of questionnaires among the employees of large corporations followed.

In the early part of January 1929 Mr. Arthur S. Jones, manager of the pipe-line department of the Associated Oil Co., was interviewed concerning the possibility of having questionnaires placed at the pumping stations and with field engineers of the company, and under date of January 31, 1929, Mr. Jones offered to cooperate fully in this work. Questionnaire cards were then placed with the members of his organization. This was the first occasion in which the facilities of a large public-service corporation were utilized for the collection of earthquake data.

During the succeeding months, a number of other public-service corporations agreed to cooperate in furnishing reports. In addition, many interested individuals have been secured as regular reporters, it being the policy of the Coast and Geodetic Survey officers in California to supply with questionnaires all who are willing to report.

The work has been under the general direction of Capt. N. H. Heck, Chief of the Division of Terrestrial Magnetism and Seismology of the Coast and Geodetic Survey; the execution of the plan, including

Form 680
Rev. Mar., 1932

An earthquake was **felt, not felt,** on

Date .... Time .... {a. m. / p. m.}

Please **return the card** even if the shock was not felt, as such information is essential.

Place ....

Shook how long ....

Please **underline** the words below which best describe the shock at your locality.

Felt by few, several, many, all; by observer, by others.
In building, wood, brick, ..., strongly, weakly, built; on 1, 2, ... floor, lying down, sitting, quiet, active.
Outdoors, by observer, by others; quiet, active.
Direction of motion felt **outdoors:** N., NE., E., etc.
Ground underneath locality: Rock, soil, loose, compact, marshy, filled in, ...; level, sloping, steep.
Motion rapid, slow, ...; **beginning** gradual, abrupt.
Rattling of windows, doors, dishes, ...
Creaking of walls, frame, ...
Hanging objects, doors, etc., did, did not, swing, N., NE., etc.
Pendulum clocks did, did not, stop; clocks faced N., NE., etc.
Moved small objects, furnishings, ...
Overturned vases, etc., small objects, furniture, ...
Spilled water, oil, etc., from indoor, outdoor containers, tanks, etc., in N., NE., E., ... direction.
Cracked plaster, windows, walls, chimneys, ground.
Fall of knick-knacks, books, pictures, plaster, walls.
Broke dishes, windows, furniture, ...
Twisting, fall, of chimneys, columns, monuments.
Damage, none, slight, considerable, great, total in wood, brick, masonry, concrete, ...
Awakened no one, few, many, all.
Frightened no one, few, many, all.
Trees, bushes shaken slightly, moderately, strongly.
REMARKS:

Signature ....

Address ....

Any additional information will be appreciated.

U. S. GOVERNMENT PRINTING OFFICE: 1932 11—9751

FIGURE 31.—Questionnaire card for reporting earthquakes felt.

arrangements with the heads of various organizations to cooperate in the securing of data and the actual operation of the system, was under the immediate direction of Commander Thos. J. Maher, Inspector of the San Francisco Field Station of the Coast and Geodetic Survey, from the date of inception of the program until 1934, when the entire California Seismological Program was united in one party, under the direction of Mr. Franklin P. Ulrich.

The numbers of card reports received during the various years since 1929 are shown in the following table:

Calendar year:	*Number of reports*
1929	510
1930	1, 747
1931	1, 581
1932	3, 081
1933	2, 065
1934	2, 258
1935 (Jan. 1 to May 15)	285

Prior to 1934, the questionnaire cards received from correspondents were copied in the San Francisco Field Station of the Coast and Geodetic Survey, for its files, so that a complete record would be available for inspection by anyone interested. From the original cards, a report was compiled listing the shocks observed in chronological order; this was issued in mimeographed form to those organizations cooperating in securing reports. The material was later incorporated in the annual report of earthquakes published by the Bureau. Since it had been agreed that the function of the Coast and Geodetic Survey was to collect data only, the study of such reports as were received remained with the two institutions in California actively engaged in studying the earthquake problem; the Seismographic Station of the University of California at Berkeley, and the Seismological Laboratory of the Carnegie Institution of Washington, at Pasadena. In order to avoid confusion and duplication of effort, an arbitrary line dividing the State, previously drawn by the representatives of the two institutions, was adopted and reports of all shocks centered south of that line were sent to the Seismological Laboratory at Pasadena, and of those north of the line to the University of California. The line follows the northern boundaries of San Luis Obispo and Kern Counties, and the western boundaries of Inyo and Mono Counties.

After copies of the questionnaires had been made at the San Francisco Field Station, the cards were forwarded immediately to the appropriate one of the two above-mentioned institutions. Upon the completion of examination of the cards at these places where complete files were also kept, the original cards were then forwarded to the Washington office of the Coast and Geodetic Survey where they formed part of the permanent record.

The files of reports at the University of California are arranged chronologically, where they may be inspected by interested persons; a system of cross-indexing is being established, whereby it will be possible to determine how many shocks have occurred within a given area, and thus appraise the seismicity of that region. Until 1933, complete records were kept in the San Francisco Field Station, and these have been filed according to counties.

PRESENT STATUS

Since June 1934 two copies of each card received have been made in San Francisco. One copy is immediately forwarded to the Washington office, where it forms part of the permanent record; the other is retained in San Francisco, and the original cards are sent to the University of California Seismographic Station or to the Seismological Laboratory at Pasadena. From the copies which are sent to

Washington, D. C., a mimeographed compilation is prepared, and copies distributed to a number of interested parties. These compilations are later incorporated in the printed annual seismological reports of the Bureau.

The post-card reports allow a quick grouping of earthquake intensities as follows:

A. Felt so slightly that observers will not estimate duration of movement or its direction.
B. Observer will estimate duration or direction but windows, doors, etc., do not rattle or walls creak.
C. Windows, doors, etc., rattle and walls creak, but movable objects are not displaced.
D. Movable objects are displaced but not overthrown.
E. Movable objects are overthrown; chimneys and walls may crack.
F. Chimneys are thrown down.

For earthquakes of intensity F or over field studies are usually made by the seismologists and engineers. When a trained observer goes into the field, he may then use the many criteria of a scale such as the Modified Mercalli scale of Wood and Neumann.

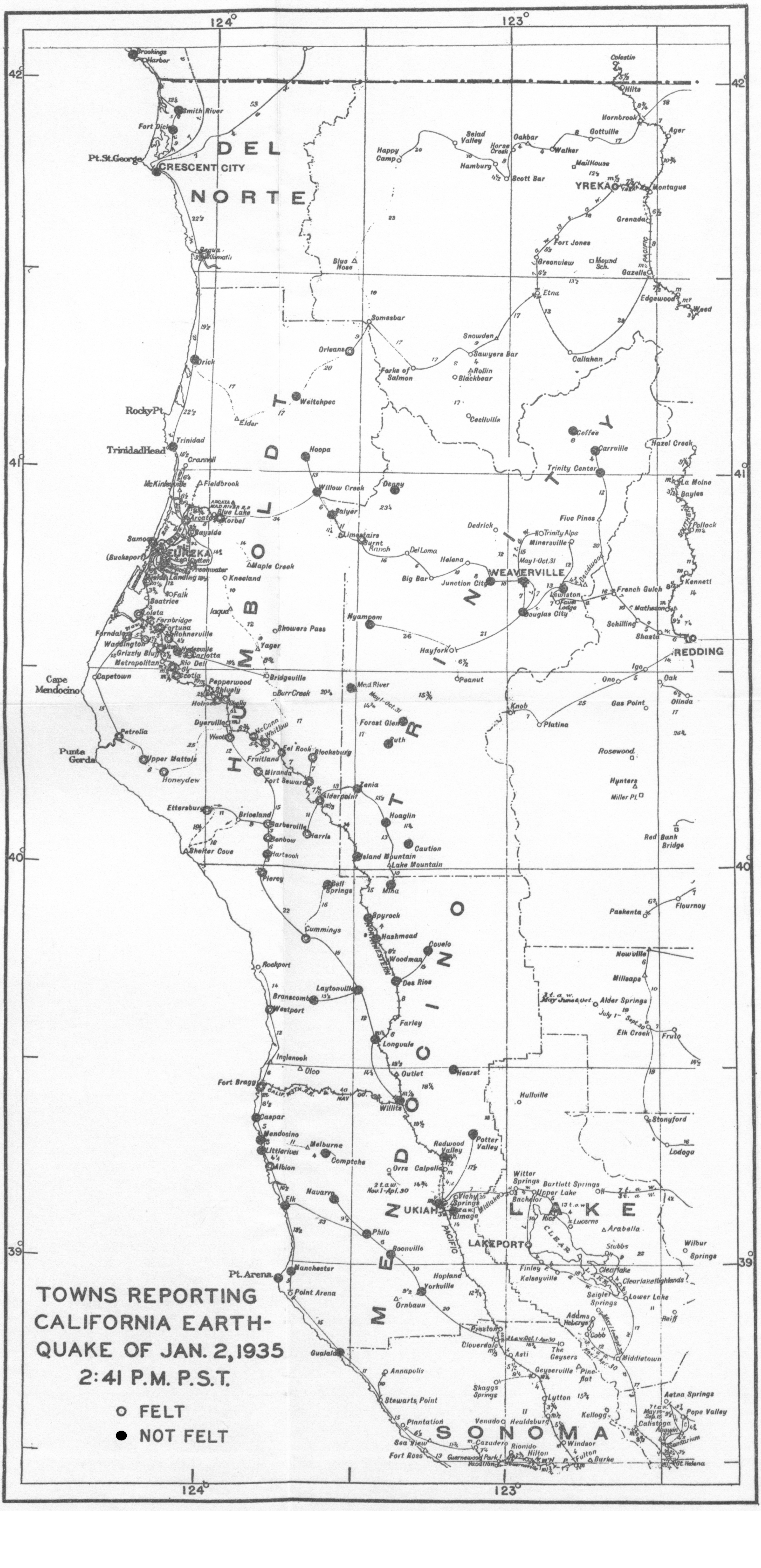
It has been noted in the California practice that regions covered by C above are very wide. The type of construction of California dwellings favors such phenomena strongly.

Seismologists recognize that reports from observers of the visible and felt effects of earthquakes are very important checks on the conclusions reached by purely instrumental means, and, moreover, are the fundamental basis of seismology. On the other hand, the instrumental record may afford a means of estimating the efficiency with which a questionnaire program is operating. An attempt has been made to apply such a criterion to the material at hand, with the following results:

1. In the case of a shock reported by a single observer, there is considerable probability that a disturbance of nonseismic character was mistaken for an earthquake.

2. Complete data regarding northern California shocks, as registered on seismographs, are available for the period from April 1, 1933, to April 1, 1934. During that period there were 85 shocks with epicenters which could be located, and of these, 16 were reported felt. It is the opinion of the writers that of these 85 recorded shocks, the great majority must have been felt, but many only very slightly. Thus, about 20 percent of shocks whose epicenters could be located by instrumental means were reported on the questionnaires. For the calendar year 1934, 600 epicenters were located in southern California. Of these 103 were reported felt, about 17 percent.

This gives a rough measure of the efficiency of the system as applied to the ordinary small shocks which occur relatively frequently throughout the State. It is not a measure, however, of the efficiency of the system as applied to large and important earthquakes; for in such a case, the procedure followed differs from that otherwise employed. As reports come in, the places from which cards are received are plotted on a map in such a way as to differentiate between those localities which report the shock felt and those which report it not felt. If, after a few days have elapsed, there appear to be regions from which no reports have been received, letters are sent to those localities requesting reports on the earthquake in question. As these are received the plotting on the map is continued, and this procedure



TOWNS REPORTING
CALIFORNIA EARTH-
QUAKE OF JAN. 2, 1935
2:41 P.M. P.S.T.
FELT
NOT FELT
124°
123°
42
41
40
39
DEL NORTE
HUMBOLDT
TRINITY
MENDOCINO
LAKE
SONOMA
CRESCENT CITY
EUREKA
WEAVERVILLE
YREKA
REDDING
UKIAH
LAKEPORT
Pt. St. George
Rocky Pt.
Trinidad Head
Cape Mendocino
Punta Gorda
Pt. Arena
Brookings Harbor
Smith River
Fort Dick
Happy Camp
Orleans
Somesbar
Weitchpec
Hoopa
Willow Creek
Trinidad
Arcata
Blue Lake
Fortuna
Ferndale
Scotia
Petrolia
Upper Mattole
Honeydew
Shelter Cove
Garberville
Laytonville
Westport
Fort Bragg
Willits
Mendocino
Albion
Elk
Boonville
Manchester
Point Arena
Gualala
Stewarts Point
Fort Ross
Cloverdale
Geyserville
Healdsburg
Windsor
Calistoga
St. Helena
Middletown
Lower Lake
Kelseyville
Covelo
Etna
Fort Jones
Montague
Weed
Hayfork
Douglas City
Trinity Center
Coffee
Carrville
Platina
Paskenta
Flournoy
Stonyford
Lodoga
Upper Lake
Clearlake Highlands
Aetna Springs
Pope Valley

is repeated until, in the estimation of the Seismological Laboratory at Pasadena or the Seismographic Station of the University of California, the area has been sufficiently well covered.

During the fiscal year 1935, eleven shocks have been so covered.

The following figures regarding specific shocks show the effectiveness of the program in the case of large shocks:

January 2, 1935: Del Norte, Humboldt, Mendocino and Trinity Counties. Reports from 12 different places were received before requests were sent out; requests were sent to 125 places; replies were received from 97 different places; there were some duplications; 110 reports were received in all. (See fig. 32.)

December 30 and 31, 1934: Southern California (these two were considered as a single shock in the tabulation). Reports from 25 different places were received before requests were sent out; 209 requests were sent out, and 157 replies were received.

FUTURE POSSIBILITIES

Since the results to be obtained from the questionnaire program are statistical in nature, the value of the program as a whole increases with the number of reports and with the length of time over which reports are received. It is obvious, therefore, that the continuation of the program is of the utmost importance, not only because of the new information thereby obtained, but in properly interpreting previously received reports. The efficacy of such a program will depend on continued contact with the observers, who must, for the most part, be reminded frequently that the program is still active and that their reports are highly appreciated. As mentioned above, it has been the practice in localities where there were no representatives of cooperating organizations, to call on the postmasters for reports. Their response has been most satisfactory, and many of the postmasters have proved to be interested and efficient observers. Where this is not the case, it is highly desirable that special observers be contacted, who can be relied upon to send in reports of any shocks in their locality, rather than depending upon intermittent requests sent to a comparatively unselected group. A widened group of special observers may be developed in time, individuals who are particularly fitted, and particularly interested, and who will be able to give carefully considered reports. Frequently individuals who spontaneously come forward and show great interest in the subject are not the best observers, since they are concerned with some particular theory as to the cause of earthquakes, rather than the phenomena exhibited by the shock.

SIGNIFICANCE OF THE PROGRAM

Many objections to the rating of earthquake intensities on the basis of earthquake scales such as the Mercalli or Rossi-Forel may be and have been raised. Such ratings are based on the reports of individuals, in many cases not trained in scientific observing, and in the case of larger earthquakes often under emotional strain at the time. The useful items on the scale are those which describe the effects of the shock on inanimate objects, not on people. Buildings are variously constructed and some will creak, wrack, or fall when others will not. This varies by districts. In regions of mild climates houses are not built, in general, as firmly as those in regions of severe climates. Thus, a shock of less energy may produce greater effect on buildings in one place than in another. But from the point of view of human welfare, the important fact about an earthquake is the damage which it does, not its total energy or the maximum acceleration of its wave motion. From this viewpoint the intensity of an earthquake depends on the type of structures in the area concerned as well as on the energy released by the shock.

Strangely enough, it is frequently the engineer who objects most to the rating of shocks by effects on structures. When pushed for a reason for his interest in a physical measure of the energy of a shock in the desert, the engineer usually answers that he wishes to know what would have happened to a city had there been one in the epicentral region. Since to date mankind appears to ignore earthquake hazard in the location of cities, the question appears to be an abstract one. But perhaps with time cities may be selected and buildings constructed with more consideration of earthquake hazard than in the past.

Earthquake motion is so varied that the physical description of it in a given shock may never be given by one number. By such work as that of the strong-motion program of the Coast and Geodetic Survey the engineer will eventually be given a fairly complete picture of the motion of an earthquake throughout its course, and the damage which a shock will cause will depend largely on the coincidence of earthquake periods of vibrations and the free periods of vibration of structures and parts of structures. The physical constants of the shock must be considered together with those of the hypothetical city before a picture of its effects on the city may be obtained.

The rating of intensities by damage done, on standard scales, will always be very useful, particularly to insurance agencies and to engineers. The isoseismals drawn by seismologists from the data presented on such scales are not such smooth curves as would be the case if some measure of energy passing the observing station were available, but from many viewpoints they are much more interesting.

There is no more important activity in the field of seismology than the systematic accumulation of the type of field data which is sought by the questionnaire. The data collected by the questionnaires will lead eventually to earthquake catalogs the value of which to the insurance man in making rates, to the engineer in choosing sites for structures, and to the geologist in determining which portions of the State are geologically active, is generally recognized. In the long run the value of this program to the members of these professions and to builders cannot be overestimated. And, if prediction of earthquakes ever becomes possible, all present indications are that it will depend, to a large extent, on the frequency of earlier shocks, as well as on the results of geodetic surveys.

Chapter 5.—VIBRATION OBSERVATIONS

D. S. CARDER

INTRODUCTION

L. S. JACOBSEN

The work that has been done in California during the past year by the several parties of the California Seismological Program of the United States Coast and Geodetic Survey is of such fundamenal importance to the engineering profession that it marks a distinct milestone in our quest for a better knowledge of how to build earthquake-resistant structures.

The following introduction and discussion of the test results is arbitrarily limited to the work covered by the subparties engaged in measuring the vibrational periods of buildings and other structures, and will not touch upon the important work done by the other parties.

It is, of course, true that a knowledge of the periods of a building is not sufficient for expressing a rational opinion as to whether or not the building is "earthquake proof" or even relatively safe. However, before such an opinion can be formulated, the analytical study as well as the tempered practical considerations require, without exception, a fairly precise knowledge of the building's vibrational characteristics. The numerous and carefully-conducted tests reported on in this communication furnish the stepping stones for linking theory and practice together in the problem of vibrational properties of structures. Without these or similar tests, we should be in the dark as to whether or not certain proposed theories have experimental confirmation.

Let us confine ourselves, for the present, to a discussion of the vibrational characteristics of a symmetrical building of approximately constant cross-sectional mass and rigidity. This will also include buildings in which the variation of mass and rigidity with height is uniform.

At first we assume a mere abstraction, namely, that the foundations of the building are infinitely rigid. Under these circumstances, we may inquire about the relative values of the natural periods of the building, about the types of distortions concomitant with the modes of vibration, and about the relative importances of taking these into consideration in future period studies.

It is obvious that if the height-to-width ratio of the building is very large, flexural distortions are of importance and shear distortions may play a small role; but just how large must this ratio be before shear may be neglected?

A theoretical answer to this question is possible, but it is not entirely acceptable until borne out by actual observations. It can be shown theoretically that for the fundamental translational modes of vibration, shear and flexural distortions are of equal importance in computing the period if the height-to-width ratio of the building is between 3 and 4. For smaller ratios, shear distortions predominate, and for larger ratios flexural distortions assume the important role. If the fundamental translational period of the building of the above proportions, as computed from shear distortions, be 1 second, another computation from flexural considerations will yield the same period, while a more accurate computation involving both types of distortion will give a period of 1.34 seconds. But what about the period corresponding to the second mode of vibration? In this case, computations from shear distortions will give a second period of 1/3 second, while flexure will yield a period of 1/6.2 second and the more accurate computation involving both types of distortion will give 1/2.7 second. For the third mode of vibration, shear predominates until the height-to-width ratio is nearly 9. Since most tall buildings on the Pacific coast have height-to-width ratios smaller than 4 or even than 3, we conclude that flexure of the building as a whole is of importance only for the fundamental period and in a few cases for the second mode period.

The two translational fundamental periods of the symmetrical building correspond to the two independent modes of vibration in planes at right angles. The fundamental torsional mode of vibration of the building about its vertical axis involves torsional or shear distortions only, and its period is quite readily computed from the constants and dimensions of the building. The second torsional period offers no difficulty and for the symmetrical, uniform building on a rigid foundation it will be one-third of the fundamental torsional period. The third mode period will be one-fifth of the fundamental, and so on.

We are now ready to consider the effect of a yielding foundation on the periods of the symmetrical building. *A priori*, we should expect that the foundation will yield in three ways: First, translatory yielding; second, rotatory; and third, torsional. It can be shown, however, that the first and third, the translatory and the torsional yielding, depend mainly on deformation constants of the ground in the horizontal direction; these can be simply related if the building is relatively isolated from other structures, and if the ground is uniform. The rotatory yielding of the foundation depends mainly on a vertical deformation constant of the ground and is almost independent of the other two types of yielding.

The effect of the translatory and rotatory yielding of the foundations on the fundamental translational period of the building is to make it longer. Thus, when each of these types of yielding is responsible for one-fourth increase in the deflection at the top of the building (height-to-width ratio of building between 3 and 4) the period changes from 1.34 seconds to 1.78 seconds. Again, if the translational yielding is responsible for one-fourteenth increase and the rotational for 1/2.8 increase of the deflection at the top of the building, the period changes from 1.34 seconds to 2.28 seconds. The yielding of the foundation affects the fundamental periods of translation as well as those of torsion very appreciably. As the yielding increases, the periods increase until the building overturns.

The second mode periods are also affected by the yielding foundation, but not to as high a degree as the fundamental. Thus, if we consider shear and translation only, the period increases from one-third second on the rigid foundation to one-half second on the completely yielding foundation, but in this case the motion is unstable.

The third mode period for shear alone on the rigid foundation gives one-fifth second while the same building on the completely yielding foundation will have a period of one-quarter second; moreover, in this case the motion is stable. Similar considerations apply to the higher modes of torsional vibration.

The conclusions that we may draw from this introduction are that experimental studies of the vibrational periods of buildings as undertaken by the California Seismological Program will help us to settle the question of foundation influence as well as the one of the relative importance of shear and flexural distortions. Thus, if shear predominates, the ratios of the observed translational and torsional fundamental periods to higher mode periods for buildings on rigid foundations should be near 3, 5, 7, etc. If flexure predominates, the translational period ratios should be near 6.2, 17.5, 34.6, etc. Since the influence of the foundation on the periods of most actual buildings is appreciable—in a few cases it has been evaluated[1]—we should expect that if shear predominates, the period ratios would be of the order 3+; 5+, 7+; while if flexure predominates, the translational periods should be in the ratios of 6.2+, 17+, 35+.

A reading of the report will convince one that shear distortions are predominant in most Pacific coast buildings.

The diversity of buildings tested makes it a great task to analyze the results, and this report does not attempt to do that. It is hoped that the data will be used extensively for serious studies of the problem by competent investigators, and that premature conclusions resting on irrational fundamental conceptions may be few.

So far we have considered the vibrational periods of a symmetrical structure. If the building possesses one-fold symmetry, as for instance an E-shaped and a U-shaped building, translational vibrations parallel to the axis or plane of symmetry will be coupled with the torsional vibrations of the building. In L-shaped buildings and other asymmetrical structures, the three fundamental modes of vibration will be coupled and therefore interdependent. This is, of course, also true of all the higher mode vibrations.

In order to find the three fundamental periods, it is necessary to consider the location of the principal gravity axes or planes of the building for which the dynamic moments of inertia are maximum and minimum. When this has been done, three simultaneous differential equations of the second order can be set up, and the frequency equation of the third degree resulting from the simultaneous solution of the three differential equations will then yield three roots that may be identified with the three fundamental frequencies or corresponding periods. Even for a one-story asymmetrical structure, the theoretical considerations are rather complicated, and so is its observed behavior. It is, therefore, not strange if a study of the test results from the unsymmetrical buildings is rather perplexing.

The work done on the free vibrations of tank towers is especially interesting on account of the relatively great uniformity in their design and construction. Whenever departures from the usual behavior occur, these can be explained, at least qualitatively, by one or several factors, namely:

[1] See thesis by J. A. Blume and H. L. Hesselmeyer, Stanford University, 1934.

(*a*) Ground conditions.
(*b*) Loose or only slightly stressed tie rods; unsymmetrical structures.
(*c*) Resonance phenomena between the gravitational vibration properties of the mass of water and the tower-tank system.

Factors (*a*) and (*b*) are quite well understood, but more obscurity surrounds factor (*c*). In order to clear up the question of the effect of the mobile water in the tank, the following discussion is submitted.

Assume that a rectangular tank is placed on a solid foundation and that the tank walls are extremely rigid. If the tank be given a rather sudden horizontal displacement, the water surface on one side of the tank will be elevated and on the other side it will be depressed. This means that two gravitational waves are started, one of elevation and another of depression. Each of these waves will travel toward its opposite side with a velocity of propagation that depends mainly on the square root of the average depth of water in the tank. The time it takes each of the waves to reach the opposite side is of course $\frac{L}{v}$. This time is called one-half the gravitational period of the water body in that particular tank. The full period is given by $T_w=\frac{2L}{v}=\frac{2L}{\sqrt{gd}}$, where L is the length of the tank, d is the average depth of water and g is the gravitational constant. It is thus seen that the lower the depth of water in a particular tank, the longer the gravitational period.

It is obvious that a similar effect occurs in a tank of circular or any other cross section, only the evaluation of T_w will not be as simple as in the case of the rectangular tank. If the average depth of water in the tank be constant, $T_w\sim\frac{1}{\sqrt{d}}$, whatever be the cross section of the tank, but if d varies, as is the case when the bottom is hemispherical, a more complicated functional relationship holds. Whatever this relation is, the fact remains that the gravitational period of the water in the tank increases with a decrease in depth of water.

Consider next the inertia effect of the water in the rectangular tank. It has been shown by several investigators that it is possible to substitute for the mobile water a dynamically equivalent rigid mass which depends in magnitude mainly on the square of the depth of the water. When the cross section of the tank is circular instead of rectangular and the bottom is hemispherical, the problem is somewhat more complicated, but the same tendency is present, namely, that the equivalent translational mass of the water increases rapidly with an increase in depth.

We may now consider the combination of tower, tank, and equivalent mass of water, and compute the translational period from the known rigidity of the tower construction, the mass of the tower, and tank proper, and the equivalent mass of the water. Let the period thus computed be T_t. This period will generally be different from T_w, but since $T_w\sim\frac{1}{\sqrt{d}}$ and $T_t\sim\sqrt{c+d^2}$ it is sometimes possible to have the two periods approach each other with a consequent exhibition of resonance effect. If T_t and T_w are nearly equal, very pronounced beats should occur in the motion.

REPORT OF VIBRATION OBSERVATIONS

Before a formal presentation of vibration data collected under the California Seismological Program is given, it seems appropriate to list briefly the instruments which are being used to obtain such data. These instruments, which are about equally divided between the subparties operating in northern and southern California, are listed as follows:

5 survey vibration meters (later type).
5 electric recorders.
2 spring recorders.
1 electric recorder, equipped to use motion-picture film.
1 survey vibration meter (experimental type).
2 Wood-Anderson seismometers (loaned by the Seismological Laboratory of the Carnegie Institution).
2 Taylor anemometers (capacity 3,000 feet per minute).
Accessory equipment such as lamp assemblies equipped with single-filament 6-volt lamps, timing clocks, electric timers, and sheets of black cloth to protect recording equipment from the light.

The first instruments used by the subparty working in northern California were the experimental vibration meter and spring recorder. The subparty working in southern California used a Wood-Anderson seismometer with a spring recorder. In each case, the second component was measured by resetting the instruments at right angles to the former position. Upon the arrival of the new vibration meters (fig. 33) provision was made by both parties to record both directions at the same time. Later, the arrival of new recorders (fig. 34) made simultaneous recording possible. No provision has yet been made to record vertical vibrations. With the present equipment, optical distances are kept fixed for a given type of work; about 100 cm for routine building vibration work, and about 60 cm for recording tank and bridge tower vibrations where the amplitudes are greater and space is more at a premium. If greater sensitivity is needed, a Wood-Anderson seismometer replaces the Survey meters.

Normally the instrumental period is set well above the expected period to be measured. The trace amplitudes are then approximately proportional to the displacement of the ground, which is computed directly by taking account merely of the static magnification of the instrument. This method gives results which are theoretically accurate to 10 percent when the period of motion of the structure is almost as high as the period of the instrument, provided the damping ratio of the pendulum is taken near 10:1.

If the amplitudes of vibration of a structure are great, say 1 mm or more as in the case of forced tank-tower and normal bridge-tower vibrations, an effective method of reducing the trace amplitude without a serious change of the instrumental constants is to set the instrumental period somewhat below that of the structure. When this method is used, or whenever greater accuracy is desired than can be obtained by taking account of the static magnification alone, the harmonic magnification formula is used. It is applicable to all cases of sustained simple harmonic motion. The assumption that the motion is of this type is not far wrong in most of the cases dealt with here. It gives the effective magnification of the seismograph in terms of its static magnification, V, in the following form:

$$\frac{V}{\sqrt{(1+u^2)^2-4u^2(1-h^2)}}$$

in which u is the ratio of the period of the vibration being recorded, to that of the seismometer; and h is the damping constant of the seismometer. The damping constant h is computed from the damping ratio ϵ by the formula

$$h=\frac{1}{\sqrt{1+\left(\frac{\pi}{\log_e \epsilon}\right)^2}}$$

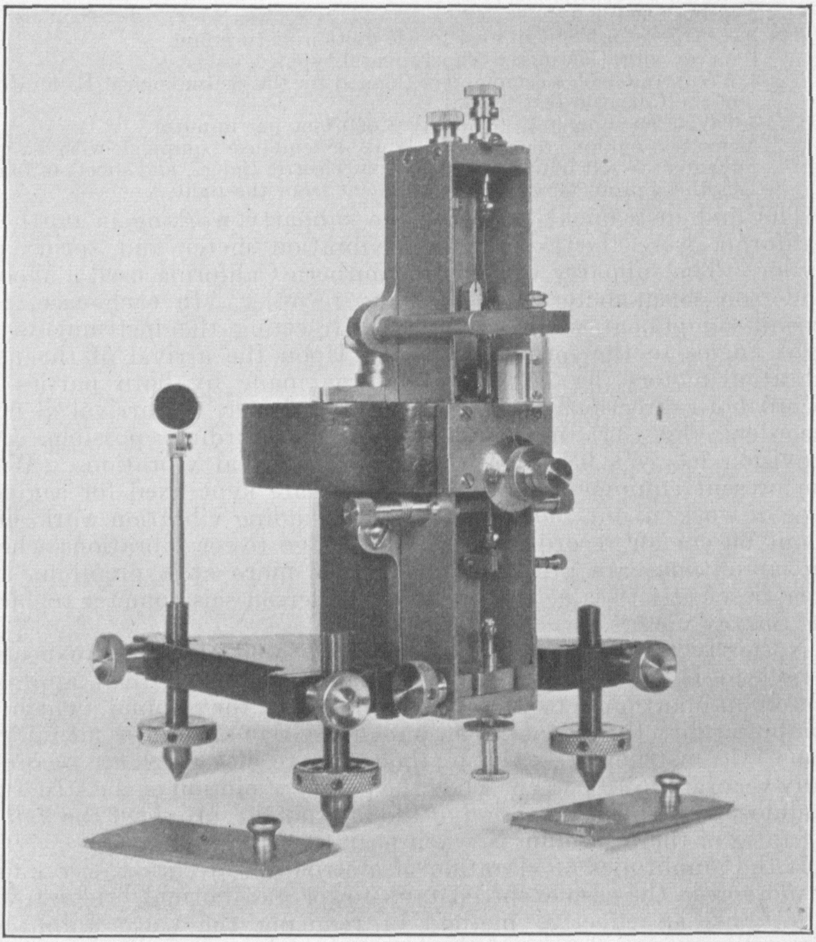


FIGURE 33.—Vibration meter.

BUILDING VIBRATIONS

Investigation of building vibrations has been carried out according to the following plan:

(*a*) Measurement of periods of buildings of all types with record of points of observations, such observations to be filed at the Washington office so that comparisons may be made in case buildings are subjected to a future severe earthquake. This will also give useful information on the variation of period with height. In most cases, observations to be made only at one place in a building and preferably at the top, or several measurements at different parts of an upper floor in a building of complex cross section.

TABLE 3.—*Summary of building vibration data, May 1, 1934, to Mar. 31, 1935*

EXPLANATIONS OF ABBREVIATIONS AND SYMBOLS

[Reference numbers refer to explanations in footnotes]

Bk. Brick. Brick walls are curtain walls unless otherwise noted.
C. Cross, or short dimension of building.
Comb. Combined.
Conn. Connected horizontally (interconnected).
Cont. Continuous.
F. Facing.
fl. Floor.
L. Wood lath; also used before period to indicate long direction of building.
Mez. Mezzanine floor.
ML. Metal lath.
P. Plaster.
Pent. Penthouse.
RC. Reinforced concrete.
St. Steel. Always refers to structural steel; not reinforcing bar.

S. Stone.
s. Street sides. Refers to brick, tile, or terra cotta face on walls.
TC. Terra cotta.
W. Wood.
x. Used in dimensioning L-shaped buildings. Tabulated figures are outside dimensions. x indicates width of wings.
* Asterisk indicates an approximate value.
⊥ Perpendicular.
† The interpretation of the periods on the records made in these buildings is believed to be particularly doubtful and will require additional tests to separate translation, torsion, or extraneous motions.
§ In flat-iron type buildings the longitudinal direction is defined as the direction of the bisector of the vertex angle.

‡ Beaufort scale of wind force:
Light = 0 to 7 m. p. h.
Gentle = 8 to 12 m. p. h.
Moderate = 13 to 18 m. p. h.
Fresh = 19 to 24 m. p. h.
Strong = 25 to 38 m. p. h.
Gale = 39 to 54 m. p. h.
Whole gale = 55 to 75 m. p. h.
Hurricane = above 75 m. p. h.
To convert miles per hour to feet per minute, multiply by 88. Then the wind-velocities used in this table will be approximately the following:
Very light = less than 100 feet per minute.
Light = about 900 feet per minute.
Moderate = about 1,400 feet per minute.
Fresh = about 1,900 feet per minute.
Strong = about 2,800 feet per minute.

RECTANGULAR BUILDINGS

No.	Building, city, date of observation	Soil	Foundation type and loading	Materials: Frame	Materials: Walls	Materials: Partitions	Materials: Floor	Dimensions: Length	Dimensions: Width	Dimensions: Height	Dimensions: Height	Vibration data: Position	Component. Direction or street	Periods: Fundamental: Translation	Periods: Fundamental: Torsion	Periods: Other	Periods: Extraneous	Maximum building displacement	Wind velocity	Remarks
			Lbs. per sq. ft.					*Feet*	*Feet*	*Stories*	*Feet*			*Seconds*	*Seconds*	*Seconds*	*Seconds*	*0.001 inch*	*Beaufort scale or ft./min.‡*	
1	Addams, Pasadena Junior College. Sept. 4, 1934.	Gravel	RC. footing. 8,000	RC	None	None	RC	132	90	2	41	2 fl.	NS EW	0.55 0.54		0.29 0.3*		0.07 0.07	275	Being rebuilt. No walls or partitions.
2	Agassiz, Pasadena Junior College. Sept. 4, 1934.	do	do	RC	None	None	RC	132	90	2	41	2 fl.	NS EW	0.55 0.55		0.29 0.29		0.07 0.07	Light	Same as No. 1.
3	Alexander, San Francisco. Oct. 17, 1934.	Sand	do	St	Bk	Tile	RC	68	60	15	196	16, 14, 12, 11, 10, 5 fl.	Montg Bush	1.25 1.33	0.80	0.40, 0.22 0.40, 0.22		1.30 1.00	Gentle	No thirteenth floor.
4	Ambassador Apartments, Berkeley. Dec. 18, 1934.	Alluvium	RC. footing	RC	RC	W. and P.	RC. and W.	88	45	6	80	6 fl.	Bancroft Union	C. 0.45 L. 0.25*		0.28, 0.08 0.08		0.15 0.05	Light	
5	American Trust Co., Berkeley. Dec. 18, 1934.	Gravel and clay	RC. footing. 8,000	St	Bk. and TC.	ML. and P.	RC	65 [1]	65 [1]	12	163	12 fl.	Center ⊥Center	1.05 1.25	0.70	 0.4*		0.50 1.50	Light	1-story wing on west.
6	American Trust Co., San Jose. Jan. 21, 1935.	Sand and clay, soft.	Grillage. Footing on RC. mat.	St	Bk., S. F. s.	ML. and P.	RC	75	58	7	108	7 fl.	First San Fernando	0.90, 0.84 0.79, 0.84		0.79, 0.12 0.11		0.50 0.50	Light	
7	Bank of America, Berkeley. Dec. 18, 1934.	Alluvium	RC. footing	St	Bk	ML. and P.	RC	76	40	6	85	6 fl.	Center ⊥Center	0.50 0.45		 0.28		0.25 0.25	Light	Street fronts not exactly at 90° angle.
8	Bank of America, Market and New Montgomery, San Francisco. Jan. 11, 1935.	Sand and clay	do	St	RC., S. F. s.	Tile	RC	92	55	14	175	14 fl.	New Montg Market	1.30 1.53	0.84	0.11* 0.11*		0.50 1.50	Light	
9	Bank of America, San Jose. Sept. 26, 27, 1934.	Sand and clay, soft.	RC. footing on piles.	St	RC., Bk. F.	ML. and P.	RC	125	54	13	175 250 [2]	Tower, 13, 11, 9, 7, 5, 2 fl.	First Santa Clara	1.20 1.31	0.80	0.75, 0.42, 0.33 0.36, 0.24	0.12 0.12	0.70 0.50	500	
10	Barker Bros., Los Angeles. Feb. 15, 1935.		RC. footing	St	Bk. and TC.	Tile	RC	318	108	11	150	11 fl.	Flower Seventh	1.85 1.23		0.2*, 0.1* 0.2*		1.00 0.40	Light	
11	Belmont Fire Station, Long Beach. Oct. 10, 1934.	Fill	Concrete	W. and St.	Bk. veneer	L. and P.	W			2		2 fl. and ground	⊥Second Second	(?) (?)		0.08, 0.04 0.17, 0.08, 0.04	0.33 0.33	0.07 0.07	Light	Period on ground 0.32. 0.07 may be building period. Steel beams and column wood bearing studs.†
12	Buffum Hotel, Long Beach. Oct. 6, 1934.	Adobe overlying sand.	RC. footing	W.[4] and St.	Bk	L. and P.	W. on St. beams.	150	50	4	42	4 fl. and basement.	⊥Third Third	0.25* 0.23		0.10* 0.10*	0.33 [3] 0.33 [3]	0.15 0.15	Light	Ground period may be 0.33.†
13	California, Oakland. Dec. 15, 1934.	Hard clay	RC. footing. 6,000	RC	RC	ML. and P.	RC			10	110	10 fl.	Franklin Nineteenth	C. 0.49 L. 0.35		0.1* 0.2*		0.25 0.20	Light	

See footnotes at end of table.

TABLE 3.—*Summary of building vibration data, May 1, 1934, to Mar. 31, 1935*—Continued

RECTANGULAR BUILDINGS—Continued

No.	Building, city, date of observation	Soil	Foundation type and loading	Materials: Frame	Materials: Walls	Materials: Partitions	Materials: Floor	Dimensions: Length	Dimensions: Width	Dimensions: Height	Dimensions: Height	Vibration data: Position	Vibration data: Component. Direction or street	Periods: Fundamental: Translation	Periods: Fundamental: Torsion	Periods: Other	Periods: Extraneous	Maximum building displacement	Wind velocity	Remarks
			Lbs. per sq. ft.					*Feet*	*Feet*	*Stories*	*Feet*			*Seconds*	*Seconds*	*Seconds*	*Seconds*	*0.001 inch*	*Beaufort scale or ft./min.*‡	
14	California and Hawaiian Sugar Refining Co., Crockett:																			
	(a) Charhouse. Aug. 3, 6, 7, 20, 1934.	Shale ledge under piles.	Cont. RC. arches on piles.	St	Bk	None	RC	([5])	([5])	9	140	9 fl.	NS EW	0.94 0.92		0.20 0.28	1.00, 0.3, 0.17, 0.05. 1.00, 0.3, 0.1, 0.05.	0.20 0.20	Light to moderate.	1.00 and 0.30 sec. periods when plant is running.
	(b) Refinery. Aug. 6, 7, 1934.	do	do	St	Bk	None	RC			7		7 fl.	NS EW	0.50 0.50		0.26 0.26	0.15, 0.05. 0.40, 0.35, 0.06.	0.10 0.10	do	Plant not running.
	(c) Packing House No. 1. Aug. 6, 7, 1934.	do	do	St	Bk	None	RC	([5])	([5])	7		7 fl.	NS EW	0.50 0.52		0.05(?) 0.25(?)	2.7, 0.10. 2.7, 0.40, 0.05.	0.05 0.05	do	
	(d) Packing House No. 2. Aug. 6, 20, 1934.	do	do	St	Bk	None	RC			4		4 fl.	NS EW	0.95 0.95		0.28 0.28	2.5(?)[6], 0.1. 0.1.	0.20 0.20	do	
15	California Pacific, San Francisco. Nov. 5, 1934.	Sand and clay, firm.	RC. footing. 8,000	St	RC., Bk. F. s.	ML. and P.	RC	60	35	11	140	11 fl.	Montg. Sutter	C. 1.16 L. 0.88, 0.54[7].		0.5*, 0.17.		0.80 0.25		
16	Campanile, University of California, Berkeley. July 19, 1934.	Gravel	RC. slab	St	RC. and granite.	None	RC[8]	36	36	8[8], and tower.	302	Between 7 and 8, 6, 4 fl.	NS EW	1.19 1.19		0.27 0.27		0.25 0.43	Light	Ornamental tower. Near Hayward fault
17	Campbell Apartments, Long Beach. Aug. 29, 1934.	Alluvium, soft	RC. footing. 6,000	St.[9]	RC	Tile	RC	148	54	10	124	10 fl.	NS EW	1.00 1.00	0.56	0.35 0.35		0.30 0.15	250	†
18	Chevrolet-Fisher Body Co., Oakland. Nov. 9, 1934.	Adobe clay	RC. footing	RC	Bk. filler	None	RC			2	24	Roof	Hillside 73d	0.35* 0.26 0.26*			0.1* 0.10[10]	0.10 0.05	200	Steel tank tower rests on building columns.
19	City Hall, Long Beach. Oct. 17, 1934.	Adobe over sand	do	St	RC		RC			6		6 fl.	Pacific Broadway	0.30 0.36		 0.25(?)		0.07 0.07	700	
20	City Transfer, Long Beach. Aug. 27, 1934.	Adobe	RC. footing. 5,500	RC	RC	Few	RC	106	60	7	71	7 fl.	⊥ Anaheim Anaheim	0.32 0.34		0.15(?) 0.2(?)		0.15 0.30	Very light	After earthquake repairs. See table 7.
21	Claus Spreckles, San Francisco. Oct. 18, 1934.	Sand	St. Grillage mat. 4,500.	St	Stone	Tile and W.	RC. and S.	75	75	19	266	18, 15, 14, 13, 12, 11, 10, and 5 fl.	3d Market	2.20 1.95		0.66, 0.40, 0.20. 0.60, 0.36, 0.30, 0.27, 0.20.		0.8 1.0	Light	
22	Coit Tower, San Francisco. Oct. 22, 1934.	Sandstone	RC. mat[11]	RC	RC	None	RC. few				173	Top	NS EW	0.405 0.405				0.40 0.40	Mod	Tubular concrete tower. Very smooth motion.
23	Commerical, San Jose. Jan. 21, 1935.	Sand and clay, soft.	RC. conn. footing on piles.	St	RC. and Bk.	ML., P., and tile.	RC	69	50	10	142	10 fl.	1st Santa Clara	0.66 0.56	0.37	0.13 0.11		0.25 0.25	Light	
24	Cooper, Los Angeles. Mar. 4, 1935.		RC. footing. Some piles (?).	RC	RC	Tile	RC	150	150	11	150	11 fl.	Los Angeles 9th	0.96 0.78		0.86, 0.50. 0.50		0.25	1,500	
25	Cunard, San Francisco. Jan. 11, 1935.	Sand and marine clay.	RC. girder and slab.	St	RC., Bk. F. s.	ML. and P.	RC	80	30	10	113	10 fl.	1st Market	L. 0.53 C. 0.97	 0.40*	0.11 0.11		0.25 1.00	Gentle	
2 6	De Young Museum, San Francisco. Oct. 24, 1934.	Sand	RC. footing	RC	Bk	Tile	RC			5[12]		In tower	NW	0.45		0.03		0.25	Light	
27	Easton, Oakland. Dec. 17, 1934.	Sand and clay	do	St	Bk., S. F. s.	ML. and P.	RC	105	52	11	160	11 fl.	Broadway 13th	1.67 1.18	0.71	0.1* 0.1*		1.50 1.00	360	
28	English, Long Beach Junior College. Sept. 26, 1934.	Alluvial silt	Concrete	W	W	W	W., RC. on ground.			1	15	Attic	NS EW	0.32[13] 0.32[13]		0.16		0.01 0.01	200	Under construction Tile on roof, no stucco on walls.†
29	Exchange Garage, San Francisco, Dec. 26, 1934	Fill overlying marine clay.	Concrete piles	RC	RC	None	RC	137	137	6	80	6 fl.	Battery Pine	0.78 0.25	0.53	0.2*	 1.0*, 0.4*[14]	0.25 0.05	Mod	Abuts no. 30 and no. 158.
30	Exposition, San Francisco. Nov. 30, 1934.	do	RC. conn. footing on piles.	RC	RC	RC[15]	RC	137	44	8	99	8 fl.	Battery Pine	C. 0.43 L. .308		0.32	0.8(?)	0.25 0.25	Mod	Abuts no. 29.

See footnotes at end of table.

TABLE 3.—*Summary of building vibration data, May 1, 1934, to Mar. 31, 1935*—Continued

RECTANGULAR BUILDINGS—Continued

No.	Building, city, date of observation	Soil	Foundation type and loading	Materials: Frame	Materials: Walls	Materials: Partitions	Materials: Floor	Dimensions: Length	Dimensions: Width	Dimensions: Height		Vibration data: Position	Vibration data: Component. Direction or street	Periods: Fundamental: Translation	Periods: Fundamental: Torsion	Periods: Other	Periods: Extraneous	Maximum building displacement	Wind velocity	Remarks
			Lbs. per sq. ft.					*Feet*	*Feet*	*Stories*	*Feet*			*Seconds*	*Seconds*	*Seconds*	*Seconds*	*0.001 inch*	*Beaufort scale or ft./min.‡*	
31	Famous Department Store, Long Beach. Mar. 23, 1934.	Adobe overlying sand.	RC. footing	RC	RC. and Bk.[16]	None	RC	100	100	3	35	3 fl	Pine 6th	0.31 0.30		 0.09		0.15 0.07	520	
32	Farmers' and Merchants' Bank, Long Beach. Dec. 5, 1934.	Adobe over sand	do	RC	Bk	Tile	RC	100	54[17]	10	136	10 fl	Pine 3d	1.00 0.87		0.8*, 0.3* 0.7*, 0.3*		0.60 0.50	1,200	
33	First National Bank, San Jose. Jan. 21, 1935.	Soft sand and clay	RC. footing on piles	St	RC. and Bk	Tile	RC	100	58	9 and Mezz.	130	9 fl	1st Santa Clara	0.86 0.74	0.41	0.25* 0.26, 0.10		0.25 0.35	Light	
34	Franklin, Oakland. Dec. 13, 1934.	Sandy clay	RC. footing	St	RC	Tile	RC	150	53	11 and Pent.	165	12 fl. Pent	Franklin 16th	0.89 0.49		0.2* 0.6*		0.75 0.25	Light	
35	Garment Capital, Los Angeles. Mar. 8, 1935.		do	RC	Bk	Tile, few	RC	124	55	12	150	12 fl	Santee 8th	0.84 1.24	0.50	 0.4*, 0.25*		0.50 2.00	Light	
36	Graphic Arts, Los Angeles. Nov. 26, 1934.		do	RC	RC. and Bk	Tile	RC	137	78	6	78	6 fl	⊥Pico Pico	0.48 0.35				0.15 0.10	Very light	Large windows—all walls.
37	Great Western Power, Oakland. Nov. 7, 1934.	Sandy clay	RC. footing. 8,000	St	RC	ML. and P	RC	57	46	10	126	10 fl	Broadway 19th	1.00 1.00		0.35 0.44		0.80 0.40	Light	
38	Griggsby, Long Beach. Nov. 23, 1934.	Adobe and sand	RC. footing	St	RC	None	RC	100	100	2		2 fl	Pine 4th	(?) (?)		0.10 0.09	0.35† 0.35†	0.07 0.02	Very light	Ground period may be 0.35 sec. †
39	Harris Newmark, Los Angeles. Mar. 8, 1935.		do	RC	RC. and Bk	Tile, few	RC	160	138	12	150	12 fl	Los Angeles 9th	0.85 0.92		0.30 0.50		0.25 0.25	Light	
40	Haas, Los Angeles. Mar. 29, 1935 and July 8, 1935.		do	St	Bk. and TC	Tile	RC	150	53	12	150	12 fl	Broadway 7th	1.20, 1.10 1.20, 1.10	0.98	1.06, 0.40, 0.2*, 0.08. 0.5*, 0.40, 0.21		0.75 0.75	Light	
41	Heartwell, Long Beach. Mar. 21, 1935.	Soft alluvium	RC. footing. 7,700	RC	Bk	Tile	RC	159	50	12	130	12 fl	Pine Ocean	L. 0.76 C. 1.10, 1.15.	 1.03	0.5*		0.30 0.70	400	Has not been repaired since 1933 earthquake.
42	Hollywood Storage, Hollywood. Aug. 13, 1934.	do	RC. comb. & column footing on piles.	RC	RC	Tile, few	RC	217	51		141	Pent., 10, 7 fl	NS EW	1.20 0.49		0.5?, 0.03 1.19, 0.06		1.00 0.20	Light	Long and narrow. No. partitions above first floor.
43	Insurance Center, San Francisco. Nov. 30, 1934.	Fill overlying sand and clay.	RC. comb. footing on piling.	St	RC., Bk. F. s.	ML. and P	RC	85	45	15	178	13 fl	Sansome Pine	C. 1.45 L. 1.10	 0.70	0.25*		1.5 1.0	Mod	
44	Insurance Exchange, Long Beach. Nov. 22, 1934.	Adobe over sand	RC. footing	St	Bk., S. F. s.	Tile	RC	74	50	7	96	7 fl	Locust Broadway	0.95 0.85	0.73	0.17*, 0.1* 0.32, 0.2*, 0.1*		0.30 0.35	Very light	
45	Insurance Exchange, Los Angeles. Jan. 9 and Apr. 23, 1935.		do	RC	Bk	Tile	RC	156	86	12	150	12 fl	9th Olive	1.06 1.25	0.74	0.37, 0.2* 0.43, 0.25		0.20 0.20	Light	
46	International Mart, Los Angeles. Mar. 4, 1934.		do	RC	BK. filler	Tile	RC	150	100	12	150	12 fl	⊥Washington Washington	0.94 0.75		0.50, 0.19 0.24, 0.10		0.20 0.20	1,000	
47	Jergin's Trust Co., Long Beach. Aug. 30, 1934.	Soft alluvium	RC. footing. Comb.[18]	St	Bk	Tile	RC	179	135	9	112	8 fl	⊥Ocean Ocean	0.74 0.70		0.15* 0.15*		0.15 0.20	Light	
48	Lagunita Court, Stanford Univeristy. Sept. 25, 1934.	Gravel	RC	Welded St	ML. and Stucco.	St.,[19] ML. and P.	W., St. joist	209	42	2	33	Attic	NS EW	0.14 0.13		0.026 0.080		0.03 0.03	Light	Designed for 10 percent horizontal force. Finished building. See table 7.
49	Lloyd and Casler, Los Angeles. Nov. 26, 1934.	Recent alluvium	RC. footing	RC	RC., Bk. F. S.	Tile	RC	120	108	3	40	3 fl	Pico Wall	0.27 0.27			0.05[20] 0.05[20]	0.04 0.03	Very light	
50	Luckenbach, Los Angeles. Feb. 23, 1935.		do	RC	RC	Tile	RC	67	32	8	88	8 fl	Hill 3d	1.23 0.50		0.38 0.3[21]		1.50 0.25	450	
51	Lumbermen's, San Francisco. Feb. 5, 1935.	Fill overlying marine clay.	RC. footing on piles	St	Bk	ML. and P	RC	114	46	7	90	6 fl	Drumm Pine	0.27 0.72[22]		0.07 0.5(?), 0.07		0.10 0.25	Gentle	
52	Lyon Storage, Long Beach. Nov. 15, 1934.	Adobe overlying sand.	RC. footing	RC	RC.[23]	None	RC					4 fl	NS EW	0.28 0.30		0.18* 0.18		0.20 0.07	Light	

See footnotes at end of table.

TABLE 3.—*Summary of building vibration data, May 1, 1934, to Mar. 31, 1935*—Continued

RECTANGULAR BUILDINGS—Continued

No.	Building, city, date of observation	Soil	Foundation type and loading	Materials: Frame	Materials: Walls	Materials: Partitions	Materials: Floor	Dimensions: Length	Dimensions: Width	Dimensions: Height (Stories)	Dimensions: Height (Feet)	Vibration data: Position	Component. Direction or street	Periods: Fundamental: Translation	Periods: Fundamental: Torsion	Periods: Other	Periods: Extraneous	Maximum building displacement	Wind velocity	Remarks
			Lbs. per sq. ft.					*Feet*	*Feet*	*Stories*	*Feet*			*Seconds*	*Seconds*	*Seconds*	*Seconds*	*0.001 inch*	*Beaufort scale or ft./min.*‡	
53	I. Magnin and Co., Oakland. Nov. 7, 1934.	Fill overlying sandy clay.	RC. Conn. footing on RC. piles.	St.	RC., tile F.	None	RC	113	100	4	74	4, 1 fl.	Broadway	0.43			0.09 [14]	0.15	Light	
													20th	0.37				0.15		
54	I. Magnin and Co., San Francisco. Oct. 25, 1934.	Sand, some clay	RC. footing	RC	RC	Few	RC	160	95	8	110	7, 8 fl.	Geary	0.70	0.40			0.50	Light	
													Grant	0.58		0.7*		0.35		
55	May Co., Los Angeles. Jan. 19, 1935 and July 12, 1935.		do	St.	Bk. and TC.	None	RC	355	322	9	130	9 fl.	Broadway	2.08	1.6	0.44, 0.09		0.70	200	
													8th	1.80		0.53, 0.44, 0.09		0.70		
56	Maxfield, Los Angeles. Mar. 8, 1935.		do	RC	RC	Few	RC	90	81	12	145	12 fl.	Santee	0.93		0.31, 0.15*		0.50	Light	
													⊥Santee	0.50				0.25		
57	Medical, Los Angeles. Mar. 1, 1935.		do	RC	RC	St. and P.	RC	100	50	8	90	8 fl.	Lucas	0.50		0.25*, 0.1		0.25	Light	
													6th	0.41				0.20		
58	Medico—Dental, Los Angeles. Feb. 26, 1935.		do	RC	Bk.	Tile	RC	100	60	13	156	13 fl.	Francisco	1.22		0.50		0.50	600	
													8th	1.06		0.10*		0.50		
59	Medico—Dental, San Jose. Jan. 21, 1935.	Blue clay	RC. Conn. footing on piles.	St.	RC	ML. and P.	RC	106	42	11	138	11 fl.	6th	C. 0.92	0.50	0.11*		0.45	Light	
													Santa Clara	L. 0.64		0.2*		0.25		
60	Morgan's Professional, Berkeley. Dec. 18, 1934.	Clay, firm	RC. footing	St.	Bk.	ML. and P.	RC	75	51	6	80	6 fl.	⊥Univ.	L. 0.36		0.08		0.10	Light	
													Univ.	C. 0.63		0.08		0.50		
61	Newhall, San Francisco. Dec. 26, 1934.	Fill overlying marine clay.	RC. mat on piles	St.	Bk.	Tile	RC	100	60	10	130	11 fl (Pent.)	Battery	0.77		0.3*, 0.1		0.50	Mod.	
													Calif.	0.81		0.28		0.50		
62	Oceanic, San Francisco. Feb. 5, 1934.	do	RC. footing on piles.	RC	RC	Tile, ML. and P.	RC	102	60	8	100	8 fl.	Davis	0.72		0.07		0.25	Light	
													Pine	0.79		0.35, 0.08		0.25		
63	130 Bush St., San Francisco. Jan. 9, 1935.	Mud over firm clay.	do	St.	RC	None	RC	80	20	11	122	11 fl.	Bush	(?)		0.80*	1.88 [15]	1.00	Gentle	Very narrow building between Adam Grant (202) and Shell (128). See fig. 53.†
													Battery	1.05		0.25?, 0.1*	1.89 [15]	1.00		
64	Pacific Commerce, Los Angeles. Mar. 18, 1935.		RC. footing	RC	RC	Tile	RC	100	40	10	150	10 fl.	4th	L. 0.50	0.40	0.16		0.25	500	
													Hill	C. 0.80		0.07		0.50		
65	Pacific Gas and Electric Co., Oakland. Nov. 7, 1934.	Sandy clay	do	RC. and St.	RC	ML. and P.	RC	100	50	8	135	Roof	Clay	1.16		0.34		1.00	Light	
													17th	0.93		0.27		1.00		
66	Pacific National Bank, San Francisco. Jan. 9, 1935.	Sand and clay, firm.	RC. footing. 5,000	St.	RC., Bk. F. s.	Tile	RC			18	220	11 fl. and Pent.	Montg.	1.65		0.36, 0.29, 0.1*		6.00	Gentle	
													Calif.	1.00		0.37, 0.15*		0.50		
67	Patriotic Hall, Los Angeles. Mar. 12, 1935.		RC. footing	St.	Bk.	Tile, ML. and P.	RC	100	80	10	130	10 fl.	Figueroa	0.50				0.25	Light	
													18th	0.50		0.32		0.10		
68	Printing Center, Los Angeles. Nov. 24, 1934.	Recent alluvium	do	RC	RC., Bk. F. s.	Tile	RC	141	89	12	150	12 fl.	Maple	0.91	0.72	0.25*		0.20	Light	
													⊥Maple	0.85		0.25*		0.20		
69	Public Utilities, Long Beach. Oct. 3, 1934.	Adobe over sand	do	RC	RC	Tile	RC	118	59	3	51	3 fl. and basement.	⊥Broadway			0.15	0.32†	0.02	430	May be ground period of 0.31–0.32. Building very rigid.†
													Broadway			0.16	0.31†	0.02		
70	J. W. Robinson, Los Angeles. Jan. 18, 1935.		do	RC. and St.	Bk. and TC.	Few	RC	329	213	7	110	7 fl.	Grand	0.80	0.49	0.1*		0.30	Light	
													7th	0.60		0.1*		0.20		
71	Royal Insurance Co., San Francisco. Nov. 26, 1934.	Fill over firm, clay	RC. footing on piles	St.	Bk.	Tile	RC	87	67	10 and Mezz.	157	11 fl. and Pent.	Sansome	0.70		0.23, 0.08*		0.25	Mod.	
													Pine	0.75	0.41	0.27		0.25		
72	Sachs, San Francisco. Mar. 7, 1935.	Sand and clay	RC. footing	St.	RC	Tile, ML. and P.	RC	120	40	10	130	10 fl.	Grant	L. 0.46		0.06		0.10	Mod.	
													Geary	C. 0.96	0.39	0.09		0.40		
73	St. Clair, San Francisco. Dec. 26, 1934.	Fill over marine clay.	RC. Conn. footing on piles.	RC	RC	Tile, ML. and P.	RC	70	61	8	102	8 fl.	Drumm	0.50	0.42	0.25, 0.1*		0.25	Mod.	
													Calif.	0.61		0.25?, 0.1*		0.25		
74	Santa Marina, San Francisco. Feb. 5, 1935.	do	RC. footing on piles.	RC	RC	Tile	RC	137	90	8	99	8 fl.	Drumm	0.70	0.56	1.0?, 0.3, 0.20, 0.06.		0.25	Light	
													Calif.	0.74		0.30, 0.8, 0.06		0.25		

See footnotes at end of table.

TABLE 3.—*Summary of building vibration data, May 1, 1934, to Mar. 31, 1935*—Continued

RECTANGULAR BUILDINGS—Continued

No.	Building, city, date of observation	Soil	Foundation type and loading	Materials: Frame	Materials: Walls	Materials: Partitions	Materials: Floor	Dimensions: Length	Dimensions: Width	Dimensions: Height		Vibration data: Position	Component. Direction or street	Periods: Fundamental: Translation	Periods: Fundamental: Torsion	Periods: Other	Periods: Extraneous	Maximum building displacement	Wind velocity	Remarks
			Lbs. per sq. ft.					*Feet*	*Feet*	*Stories*	*Feet*			*Seconds*	*Seconds*	*Seconds*	*Seconds*	*0.001 inch*	*Beaufort scale or ft./min.*‡	
75	Security Bank, Oakland. Feb. 25, 1935.	Sand and clay	RC. footing. 8,000	St	Bk	ML. and P.	RC	100	53	7	100	7 fl	Broadway 11th	C. 0.82 L. 0.62		0.3*, 0.08 0.75?, 0.38		0.75 0.25	Gentle	
76	Southern California Gas Co., Los Angeles. Feb. 28, 1935.		RC. Comb. footings	RC	RC	Tile, few	RC	84	50	7	85	7 fl	Broadway 10th	0.51 0.27 [26]		0.61 0.60		0.45 0.10	60	([26]).
77	Sovereign Apartments, Long Beach. Feb. 1, 1935.	Adobe and sand	RC. footing	RC	RC. and tile [27].	Tile	RC	99	60	13	150	12 fl	⊥Ocean Ocean	0.88 0.98		0.09 0.31		0.50 0.60	700	
78	State, Los Angeles. Mar. 14, 1935.	Gravel	do	St	RC. and Bk.[28].	Tile	RC	131	81	13	162	13 fl	Spring 1st	0.91 0.63		0.3*, 0.1* 0.22		0.30 0.25	500	
79	Stock Exchange, San Francisco. Dec. 28, 1934.	Fill over sand	RC. comb. footing on piles.	St	Bk. and S. F.[29].	Tile	RC	157	41	12	175	12 fl	Sansome Pine	1.15 1.20	0.65	0.08* 0.40, 0.12*		0.25 0.25	Gentle	
80	Telephone, Oakland. Dec. 13, 1934.	Sandy clay	RC. footing under mat.	St	Bk	ML and P	RC	155	64	11	165	11 fl	Franklin ⊥Franklin	C. 1.44 L. 0.95		0.1* 0.86, 0.74, 0.25*.		1.50 0.50	Light	
81	Textile Center, Los Angeles. Mar. 8, 1935.		RC. footing	RC	RC	Tile	RC	95	70	12	150	12 fl	Maple 8th	0.81 0.98	0.50	 0.79, 0.25		0.50 0.30	Light	
82	37th St. School, Los Angeles. Sept. 14, 1934.		RC	([30])	Bk.[30]	Bk	RC. and W.			3		3 fl	NS EW	0.21 0.22		 0.05		0.04 0.04	Light	
83	Title Insurance and Trust Co., Los Angeles. Mar. 12, 1935.		RC. footing	St	Bk. and TC.	Tile	RC	241	154	10	150	Pent	Spring 4th	0.86 0.91		0.50 0.31		0.25 0.50	Light	
84	Transportation, Los Angeles,. Mar. 5, 1935.		do	RC	RC	Tile	RC	112	80	13	145	12 fl	⊥7th 7th	0.83 0.89	0.50	0.10 0.12		0.25 0.40	Light	
85	200 Bush St., San Francisco. Dec. 27, 1934.	Sand over clay	RC. footing on precast piles.	St	Bk	Tile	RC	132	65	15	164	14 fl	Sansome Bush	0.95 1.00	0.65 0.65	0.15* 0.37, 0.13		0.30 0.35	Gentle	Abuts Stock Exchange. 0.65 period is common to both.
86	233 Sansome St., San Francisco. Nov. 6, 1934.	Fill over firm clay.	RC. footing on piles.	St	RC., TC. F.	Tile	RC	86	40	12	167	Pent	Sansome Pine	0.78 0.52		0.35*, 0.24 0.2*	 1.00	0.50 0.25	Mod	1.00 period from abutting Insurance Exchange No. 143.
87	University Club, Los Angeles. Jan. 17, 1935.		RC. footing	RC	RC	Tile	RC			7	110	6 fl	Hope 6th	0.36 0.39		0.1* 0.1		0.25 0.25	Light	
88	Walker's Department Store, Long Beach. Nov. 23, 1934.	Adobe over sand	do	RC	RC	Tile	RC	150	150	4	62	4 fl	Pine 4th	0.45 0.43*		0.08* 0.08	0.36 (?) 0.36 (?)	0.07 0.07	Very light	† 0.36 period most prominent. May be ground period.
89	West Coast Life, San Francisco. Jan. 11, 1935.	Sand	do	St	RC., Bk. F., s.	Tile and ML. and P.	RC	135	45	14	147	Pent	2d Market	L. 0.83 C. 1.18		1.1?, 0.4*, 0.11 0.85*, 0.4?, 0.12.		0.60 1.90	Light	
90	Wilshire Medical, Los Angeles. Jan. 17, 1935.		do	St	Bk	Tile	RC	150	67	13	150	13 fl	Westlake Wilshire	1.18 0.93		0.3*, 0.1*		0.25 0.75	Light	
91	Wiltshire Hotel, San Francisco. Mar. 7, 1935.	Sand and clay	RC. invert. T beams.	St	RC	ML. and P.	RC	80	41	16	162	Roof	Stockton Post	0.75 0.60		0.35 0.42		0.20 0.40	1,000	Abuts Drake Hotel No. 103.
92	Wurlitzer, Los Angeles. Mar. 20, 1935.		RC. comb. footing	RC	Bk. and TC.	Tile	RC	150	50	12	150	12 fl	Broadway 8th	C. 1.12 L. 0.59 [31]		0.87, 0.65, 0.5*, 0.10. 1.1*		1.00 0.30	Light	
93	—Residence, 4508 Fleming Ave., Oakland. Sept. 22, 1934.	Clay	W. on concrete	W	W	L. and P.	W			2	25	2 fl	Fleming ⊥Fleming	0.21 0.20				0.25 0.25	Mod	Wood frame house.

See footnotes at end of table.

TABLE 3.—*Summary of building vibration data, May 1, 1934, to Mar. 31, 1935*—Continued

L-SHAPED BUILDINGS

No.	Building, city, date of observation	Soil	Foundation type and loading	Materials: Frame	Materials: Walls	Materials: Partitions	Materials: Floor	Dimensions: Length	Dimensions: Width	Dimensions: Height		Vibration data: Position	Component. Direction or street	Periods: Fundamental: Translation	Periods: Fundamental: Torsion	Periods: Other	Periods: Extraneous	Maximum building displacement	Wind velocity	Remarks
			Lbs. per sq. ft.					*Feet*	*Feet*	*Stories*	*Feet*			*Seconds*	*Seconds*	*Seconds*	*Seconds*	*0.001 inch*	*Beaufort scale or ft./min.*‡	
94	Allied Crafts, Los Angeles. Nov. 27, 1934.	Recent alluvium	P.C. footing	RC	RC., Bk. F. s.	Tile	RC	142	96	10		10 fl	Maple Pico	0.91 0.91				0.50 1.00	Very light	
95	Balboa, San Francisco. Nov. 21, 1934.	Sand over blue clay.	Spread grillage	St	Bk	ML. and P	RC	100 x 44	79 x 41	10	139	9 fl. wings	2d Market	0.98 0.90		0.41, 0.18* 0.4*, 0.2*		2.5 2.5	Gentle	
96	Bank of America, 485 California, San Francisco. Nov. 22, 1934.	Hard clay	RC. cont. footing. 6,000.	St	Bk. and S., tile F.	Tile	RC	138 x 43	67 x 44	10 and 2 mezz.	165	12 fl. corner	Montg Calif	0.83 1.50		0.25* 0.80*, 0.25		0.50 1.00	Gentle	Abuts Merchant's Exchange on east (No. 206).
97	Bank of America, 108 Sutter, San Francisco. Nov. 3, 1934.	Sand and clay	RC. footing. 6,000	St	Bk	Tile, ML. and P.	RC	103 x 40	60	10	130	10 fl. corner	Kearny Sutter	0.75 1.21		1.2*, 0.4* 0.43		0.35 1.90	Gentle	
98	Bendix, Los Angeles. Nov. 20, 1934.	Soft alluvium	RC. footing	RC	RC	Tile	RC	129 x 81	141 x 86	11		11 fl	Maple 12th	0.74 0.74	0.58*	0.23* 0.23*		0.07 0.15	Light	
99	Building and city not named. Feb. 18, 1935.	Fill over marine clay.	RC. footing on piling.	St	RC. and Bk.	ML. and P	RC	125	62	13	181	12 fl	NS EW	1.34 0.99	0.72	0.3 0.31		1.00 0.50	Light	
100	Breakers Hotel, Long Beach. Oct. 11, 1934.	Beach sand	RC. piling	RC	RC	Tile	RC	144 x 60	83 x 50	13 and tower.	179, 250[32]	14 fl. tower	⊥Ocean Ocean	1.23 0.73		0.3* 0.27		0.15 0.15	500	
101	Central Bank, Oakland. Nov. 8, 1934.	Sandy clay	RC. footing. 6,000	St	Bk	Tile, ML. and P.	RC	149 x 44	101 x 44	15	220	13 fl. wings	Broadway 14th	1.32 1.12, 1.05	 0.80	1.0(?), 0.21 0.23, 0.15*		0.7 0.7	Light	
102	Cooper Arms, Long Beach. Mar. 21, 1935.		RC. footing	RC	Bk	Tile	RC	184 x 52	95 x 51	12	130	12 fl	Linden Ocean	0.94 0.94		0.30† 0.50, 0.30†		0.25 0.50	1,050	(†).
103	Drake Hotel, San Francisco. Mar. 6, 1935.	Sand and clay	do	RC	RC., Bk. F. s.	ML. and P	RC	150 x 30	50 x 70	7	84	7 fl	Stockton Post	0.75* 0.60	0.43	0.6*		0.10 0.10	Gentle	Abuts Wiltshire Hotel No. 91.
104	Financial Center, Oakland. Nov. 7, 1934.	Sandy clay	RC. footing. 8,000	St	Bk. and RC.	Tile	RC	100	73	16	219	14 fl. wings	Franklin 14th	1.12 1.34	0.82	0.34		0.70 1.00	Light	
105	Financial Center, San Francisco. Sept. 5, 1934.	Sand and clay	do	St	Bk	Tile	RC	137 x 47	137 x 47	15	200	12, 13 fl. 3 pos	Montg Calif	1.30 1.24		0.20 0.20		1.00 1.00	420	
106	Leamington Hotel, Oakland. Dec. 13, 1934.	Hard clay	RC. footing. 6,000	RC	RC	ML. and P	RC	135	135	10	110	11 fl. pent. wings	Franklin 19th	0.59[33] 0.56[33]		0.51[33], 0.2*, 0.08*. 0.51[32], 0.1*, 0.08*.		0.15 0.10	Light	
107	Matson Nav. Co., San Francisco. Oct. 24, 1934.	Fill over sand and mud.	RC. slab over piling.	St	Bk. and RC.	Tile	RC	137	137	17	218, 320[34]	16 fl. 3 pos	Main Market	1.47, 1.40 1.28, 1.31	 0.92	0.28, 0.23 0.33, 0.28		0.5 0.5	Gentle	Abuts No. 109.
108	Medical, Oakland. Dec. 13, 1934.	Sandy clay	RC. footing	RC	RC., Bk. F. s.	Tile	RC	110 x 48	64 x 30	9	120	9 fl	Franklin 19th	0.65 0.65		0.10* 0.2*, 0.10*		0.5 0.5	Light	
109	Pacific Gas and Electric, San Francisco. Oct. 25, 29, 1934.	Fill over sand and mud.	RC. Conn. footing on piling.	St	Bk	Tile	RC	137 x 48	120 x 44	17	247	17 fl. 3 pos	Beale Market	1.50 1.30	1.0–1.1	0.43*, 0.32, 0.04. 0.35, 0.04		1.0 0.5	Gentle	Abuts No. 107, Matson.
110	Pacific Telephone and Telegraph, San Francisco. Oct. 31, 1934.	Sand clay hardpan	RC. slabs. 8,000	St	Bk., TC. F	Tile	RC	160 x 60	147 x 50	26	369	26 fl. 3 pos	New Montg Mission	2.20 2.25	 1.30[35]	0.40 0.40		2.0 2.5	Light	
111	Press Telegram, Long Beach. Feb. 1, 1935.	Adobe	RC. footing. 8,000	RC	Bk	Tile and W	RC	150 x 50	150 x 50	4	64	4 fl. corner	Pine 6th	0.39 0.39		0.32*, 0.09 0.09		0.25 0.25	350	
112	Sharon, San Francisco. Jan. 11, 1935.	Sand	RC. footing. 6,000	St	Bk. and RC.[36]	Tile	RC	148 x 20	148 x 48	8	108	8 fl	New Montg Stevenson	0.50 0.54	0.39	0.41, 0.35, 0.13. 0.12		0.25 0.25	Gentle	
113	Standard Oil Co., San Francisco. Sept. 4, 1934.	Sand and sandy clay.	RC. mat.[37] 4,000	St	Bk., TC. F	Tile	RC	206 x 50	137 x 59	21	309	20 fl. 3 pos	Sansome Bush	1.80 1.50	1.20	0.6*, 0.4 0.5(?), 0.4		2.0 1.0	600. Irregular.	
114	Tapscott, Oakland. Dec. 5, 1934.	Sandy clay	RC. footing. 8,000	RC	RC., Bk. F. s.	Tile and ML. and P.	RC	150 x 48	93 x 45	4	48	4 fl. corner	Broadway 19th	0.68 0.28		0.28		0.10 0.10	Fresh	

See footnotes at end of table.

TABLE 3.—*Summary of building vibration data, May 1, 1934, to Mar. 31, 1935*—Continued

L-SHAPED BUILDINGS—Continued

No.	Building, city, date of observation	Soil	Foundation type and loading	Materials				Dimensions				Vibration data							Wind velocity	Remarks
				Frame	Walls	Partitions	Floor	Length	Width	Height		Position	Component. Direction or street	Periods				Maximum building displacement		
														Fundamental		Other	Extraneous			
														Translation	Torsion					
			Lbs. per sq. ft.					*Feet*	*Feet*	*Stories*	*Feet*			*Seconds*	*Seconds*	*Seconds*	*Seconds*	*0.001 inch*	*Beaufort scale or ft./min.*‡	
115	Welsh, San Francisco. Dec. 26, 1934.	Fill over mud and clay.	Grillage beams on piling.	St	Bk	Tile	RC	127 x 30	54 x 47	7	107	7 fl. corner	Battery	0.48		0.78*, 0.24, 0.1*.		0.10	Mod	
													Calif	0.81		0.47*, 0.24, 0.13.		0.50		
116	Willmore Apartments, Long Beach. Sept. 6, 1934.	Soft alluvium	RC. footing	RC	RC	Tile	RC	146 x 46	141 x 46	11	132	11 fl	⊥3d	0.62				0.25	250	After repairs. See table 7.
													3d	0.62				0.25		
117	Wilson, Huntington Park. Feb. 14, 1935.	Recent alluvium	do	St	RC	ML. and P.	W	150 x 43	105 x 53	4		4 fl. 3 pos	Pacific	0.33		0.5*, 0.10		0.25	Very light	
													Slauson	0.33		0.10		0.25		

HIGH TOWER AND "SET-BACK" BUILDINGS

No.	Building, city, date of observation	Soil	Foundation type and loading	Frame	Walls	Partitions	Floor	Length	Width	Height (stories)	Height (feet)	Position	Component. Direction or street	Translation	Torsion	Other	Extraneous	Maximum building displacement	Wind velocity	Remarks
118	Bank of America, Oakland. Feb. 25, 1935:																			
	(*a*) Tower	Sandy clay	RC. footing. 8,000	St	Bk	Tile, ML. and P.	RC	51 [38]	45 [38]	18	270	8, 18 fl	Broadway	1.22		0.2*	0.52	0.75	500	18-story attached to 8-story at one side.
													12th	1.22		0.45, 0.2*		1.25		
	(*b*) Old building	Sandy clay	RC. footing	St	Bk	Tile, ML. and P.	RC	150	100	8		8 fl	Broadway	0.50		0.41, 0.2*	1.20	0.25		
													12th	0.60		0.49, 0.2*	1.22	0.25		
119	City Hall, Los Angeles. Oct. 16, 1934 and Apr. 3, 1935.	Blue shale	RC. footing mat under 25th floor.	St	Bk., Tile F.	Tile	RC	318, 75 [39]	75, 75 [39]	25, 10	400	24, 20, 16, 12, 9 fl	Main	2.10		0.80, 0.61, 0.22.		1.50	Light	10-story wings on each side of tower part.
													1st	2.25		0.80, 0.63, 0.5, 0.41, 0.28*, 0.22.		1.50		
120	City Hall, Oakland. Oct. 12, 1934.	Firm sandy clay	RC. mat	St	Bk., S. F.	Tile	RC	175	125	16	310	16, 14, 12, 10, 8, 6, 3 fl.	Washington	1.10		0.45, 0.33		0.50	500	Rectangular shape set-back walls and high tower.
													14th	1.20		0.4*, 0.33		0.30		
121	City Hall, San Francisco. July 27, and Oct. 26, 29, 1934.	Sand	RC. footing. 6,000	St	Bk., S. F.	Tile	RC	408	288	4 and dome.	65, 300 [40]	4 fl. dome	Polk	0.69	0.40	0.25, 0.17, 0.11.		0.1	Mod	
													McAllister	0.65		0.32, 0.27, 0.17, 0.12.		0.1		
122	Elks Club, Carillon Hotel, Oakland. Dec. 4, 1934.	Alluvium	RC. footing. Piles under tower.	St	RC	Tile, and ML. and P.	RC	150	104	14	270	16 fl.[41] 12 fl	Broadway	0.97				0.50	1,750	High tower building connected to 4-story part.
													19th	0.71		0.15*		0.25		
												4 fl. part	Broadway	0.97 [42]		0.43, 0.21				
													19th	0.70 [42]		0.3*ca, 0.22				
123	Ferry, San Francisco. Oct. 22, 1934.	Mud over clay	Piling under RC. piers.	St	Bk. and RC.	Few	RC			2 and tower.	54, 234	9, 12 fl	Embarcadero	0.69		0.15, 0.3(?)		0.50	Mod	Low wings and central tower.
													Market	0.81		0.10, 0.3(?)		1.50		
												2nd fl. wings	Embarcadero	1.14 [43]		0.3*, 0.1*		0.25		
													Market	1.00 [43]		0.3*, 0.22, 0.1*.		0.25		
124	Hobart, San Francisco. Nov. 5, 1934.	Sand and clay	RC. conn. footing	St	RC	ML. and P.	RC	93	42	21	297	21 fl	⊥Market	1.70		1.2*, 0.4*, 0.3*.		2.0	Gentle	High tower with L-shaped twelfth floor extension.
													Market	1.25		1.5*, 0.2*		1.5		
												12th fl. part	Montg	0.49		0.23	1.5 [44]	0.25		
													Sutter	0.75		0.23	1.3 [44]	0.25		
125	Ocean Center, Long Beach. Aug. 28, 1934.	Beach sand over clay.	RC. piles	RC	RC	Tile	RC	264, 82 [45]	78, 39 [45]	11 and 3 [46].	186	10 fl	⊥Pike	0.51		0.3*, 0.2*		0.20	700	E-shaped to sixth floor.
													Pike	0.78		0.6*, 0.50, 0.2(?), 0.13*.		0.20		
126	Russ, San Francisco. Nov. 3, 1934.	Firm sand and clay.	Cont. footing mat under thirtieth floor. 8,000.	St	Bk., TC. F.	Tile	RC	275, 137 [47] 108 [48].	160, 137 [47] 6 [48].	30 and mezz.	409	16, 22, 30 fl	Mont	1.74	1.2*	0.8, 0.6, 0.3*		0.70	Mod	E-shaped to sixteenth floor. T-shaped to twenty-second floor.
													Pine	1.90		0.8*, 0.3, 0.18(?).		1.00		
127	Building and city not named. Jan. 8, 1935.	Firm clay	RC. caissons	St	Bk	Tile	RC			21		21, 10 fl	NS	1.14		0.64, 0.40	1.25 [49], 0.5* [49]	0.50		Adjacent to No. 199. †
													EW	1.38		0.40*	0.70 [49]	0.75		

See footnotes at end of table.

TABLE 3.—*Summary of building vibration data, May 1, 1934, to Mar. 31, 1935*—Continued

HIGH TOWER AND "SET-BACK" BUILDINGS—Continued

No.	Building, city, date of observation	Soil	Foundation type and loading	Materials: Frame	Materials: Walls	Materials: Partitions	Materials: Floor	Dimensions: Length	Dimensions: Width	Dimensions: Height	Dimensions: Height	Vibration data: Position	Vibration data: Component. Direction or street	Periods: Fundamental: Translation	Periods: Fundamental: Torsion	Periods: Other	Periods: Extraneous	Maximum building displacement	Wind velocity	Remarks
			Lbs. per sq. ft.					*Feet*	*Feet*	*Stories*	*Feet*			*Seconds*	*Seconds*	*Seconds*	*Seconds*	*0.001 inch*	*Beaufort scale or ft./min.‡*	
128	Shell Oil Co., San Francisco. May 3, 4, 7, 8, 9, 14, 16, 1934.	Hard clay and gravel.	RC. caissons. 8,000	St	Bk., TC. F	Tile	RC	137, 80 [50]	117, 80 [50]	29	381	All fl.	Bush	1.90		0.67, 0.40, 0.35, 0.21.		2.8	Gentle to strong.	Tenth floor wings extend at right angles from main tower.
													Battery	1.92		0.66, 0.45, 0.40, 0.35, 0.25.		2.8		Do.
129	Wm. Taylor Hotel, San Francisco. Nov. 19, 1934.	Sand	RC. footing. 8,000	St	Bk	ML. and P.	RC	137	137	26	300	26 fl.	Leavenworth	1.40		0.6*		1.2	Gentle	Thirteenth floor wings extend at right angles from main tower.
130	Tribune Tower, Oakland. Feb. 25, 1935:												McAllister	1.40		0.63		0.8		Do.
	(*a*) Tower	Sandy clay	RC. mat	St	RC	Tile	RC	100	40	20	308	19, 7 fl.	Franklin	1.55		0.77 [51], 0.52, 0.08.		1.50	Gentle	High set-back building attached to older 6-floor RC. building.
													13th	1.97		0.79 [51], 0.58, 0.33.		2.00		Do.
	(*b*) Old building	do	RC. footing	RC	RC	ML. and P.	RC	100	60	6	85	6 fl.	Franklin	0.52	0.34	0.085	0.77 [51]	0.75		
													13th	0.58		0.085	0.75*	0.25		
	U- AND E-SHAPED BUILDINGS																			
131	Architects, Los Angeles. Dec. 12, 15, 27, 1934.		RC. footing	RC	RC	Tile	RC	157	57	12	150	12 fl.	Figueroa	C. 0.75	0.45	0.24, 0.1*		0.07	Light	Long narrow U.
													5th	L. 0.53		0.2*		0.07		
132	Associated Realty, Los Angeles. Mar. 14, 1935.		do	RC	Bk	Tile	RC	201	105	12	150	12 fl.	Olive	1.14		0.38		0.40	Light	U-shaped.
													6th	1.00		0.3, 0.5*		0.30		
133	Balfour, San Francisco. July 16, 1934.	Fill	RC. piers on piles	St	Bk	Tile	RC	129	89	15	219	15 fl. wings and center.	Sansome	1.17		0.52, 0.43(?), 0.22, 0.15, 0.10(?).		1.0	Light	U-shaped. Abuts Dollar Annex, No. 158, whose Calif. St. period is same.
													Calif.	1.08, 1.14	0.85 [52]	0.21, 0.14(?), 0.11.		1.0		Do.
134	Bank of America, Montgomery and Clay, San Francisco. Feb. 18, 1935.	Hard clay	RC. caisson [53], RC. footing.	St	Bk. and RC.[53]	ML. and P.	RC	137	100	9	102	9 fl. and roof	Montg	0.59	0.48	0.27			Gentle	U-shaped. Old and new buildings attached.
													Clay	0.65		0.4(?), 0.27				Do.
135	Calif. Comm. Union, San Francisco. Jan. 10, 1935.	Firm sand and clay.	RC. footing. 6,000	St	Bk	Tile	RC	137	125	15	220	15 fl. and pent	Montg	1.39		0.46, 0.4*, 0.25.		1.25	Light	U-shaped.
													Calif	1.17		0.45*, 0.25, 0.11.		0.75		
136	Central Mfg. Dist. Terminal, Vernon. Dec. 14, 1934.	Recent alluvium	RC. footing	St.[54] and RC.	RC	None	RC	402	254	6	80	6 fl. wings and center.	Loma Vista	0.60		0.5*, 0.3*, 0.16.		0.15	Light	U-shaped.
													Loma Vista	0.60		0.36, 0.1		0.07		
137	Central, Los Angeles. Dec. 26, 1934.		do	St	Bk	Tile	RC	130	120	11	140	10 and 11 fl.	Main	1.40 †	1.02	0.40		0.15	Light	U-shaped.†
													6th	1.58 † 1.50 †	1.06	0.07		0.15		
138	Chamber of Commerce, Los Angeles. Mar. 6, 1935.		do	St	Bk. and TC.	Tile	RC	240	205	8	120	8 fl. 4 pos.	Broadway	1.07	0.93	0.1*		0.35	Light	E-shaped.
													12th	1.06		0.50, 0.43, 0.1		0.50		
139	Crocker First National Bank, San Francisco. Jan. 11, 1935.	Sand and clay	do	St	RC., S. F.	Marble and glass.	RC	160	75	12	184	12 fl.	Montg	1.30		0.19		0.75	Mod	U-shaped.
													Post	1.10		0.16		0.40		Do.
140	Edison, Los Angeles. Aug. 7, 1934.	Blue shale	RC. footing. 6,000	St	RC	ML. and P.	RC	171	164	12	150	12, 10, 6, 4, 3, 2, 1 fl.	Grand	0.66		0.3*, 0.26		0.13	450	U-shaped. Designed for horizontal force of 10 percent gravity.
													5th	0.77		0.3*, 0.26		0.13		
141	Federal Reserve Bank, San Francisco. Sept. 8, 1934.	Fill over sand and mud.	RC. footing on piles.	St	Bk. and S.	St. and glass.	RC			7	140	7 fl. wings and center.	Sansome	0.7, 0.8		0.2*		0.25	Light	U-shaped.
													Sacramento	0.65		0.2*, 0.60		0.25		
142	Howard, San Francisco. Mar. 7, 1935.	Sand	RC. footing. 6,000	St	Bk., TC. F.	Tile	RC	120	66	12	160	12 fl. ends	Grant	1.20	0.86			0.50	600	Do.
													Post	1.25, 1.35		1.08		0.75		

See footnotes at end of table.

TABLE 3.—*Summary of building vibration data, May 1, 1934, to Mar. 31, 1935*—Continued

U- AND E-SHAPED BUILDINGS—Continued

No.	Building, city, date of observation	Soil	Foundation type and loading	Materials: Frame	Materials: Walls	Materials: Partitions	Materials: Floor	Dimensions: Length	Dimensions: Width	Dimensions: Height	Dimensions: Height	Vibration data: Position	Vibration data: Component. Direction or street	Periods: Fundamental: Translation	Periods: Fundamental: Torsion	Periods: Other	Periods: Extraneous	Maximum building displacement	Wind velocity	Remarks
			Lbs. per sq. ft.					*Feet*	*Feet*	*Stories*	*Feet*			*Seconds*	*Seconds*	*Seconds*	*Seconds*	*0.001 inch*	*Beaufort scale or ft./min.*‡	
143	Insurance Exchange, San Francisco. Nov. 22, 1934.	Sand and clay	RC. footing on piles.	St.	Bk. and RC. TC. F. s.	ML. and P.	RC.	125	105	10 and mezz.	159	10 fl.	Sansome Calif.	0.90 1.00	0.50	0.3* 0.3*		0.25 0.50	Gentle	U-shaped. Abuts 233 Sansome, no. 86.
144	Kress, Long Beach. Nov. 23, 1934.	Adobe over sand	RC. footing	RC.	RC.	Tile	RC.	150	50	8	95	8 fl.	Pine 5th	C. 0.58 L. 0.44		0.08 0.07		0.20 0.07	Very light	Long narrow U.
145	Latham Square, Oakland. July 21, 1934.	Clay	RC. footing. 6,000	St.	RC., Bk. F. s.	Tile, ML. and P.	RC.	105	100	15	180	14 fl. wings and center.	Telegraph 16th	1.00 1.06, 0.91, 0.86[55].	0.52	0.40, 0.14, 0.10 0.15, 0.13, 0.11		0.25 0.45	540	U-shaped.
146	Law, Los Angeles. Feb. 28, 1935.		RC. footing	RC.	Bk.	Tile	RC.	120	45	12	120	12 fl.	Broadway ⊥Broadway	1.28 0.72		1.00, 0.4*, 0.25* 0.22, 0.1		0.15 0.10	Very light	Do.
147	Merchants Exchange, Los Angeles. Mar. 18, 1935.		do	RC.	RC.	Tile few	RC.	135	118	12	150	12 fl.	Los Angeles 7th	0.87 0.72		0.50*, 0.27 0.1*		0.25 0.25	500	Do.
148	Metropolitan Water District, Los Angeles. Mar. 29, 1935.		do	St.	Bk.	Tile	RC.	113	64	12	150	10 fl.	NS EW	1.26 1.35		0.40 0.40, 0.2*				Long narrow U. High building above theatre.
149	Ming Quong School, Oakland. Nov. 9, 1934.	Gravel	RC.	RC. walls	RC.	L. and P.	RC.			2	25	2 fl.	NW-SE	0.10*		0.05*		0.003	Very light	U-shaped. Near Hayward fault.
150	Omar L. Hubbard, Long Beach. Oct. 26, 1934.	Adobe over sand	RC. footing	RC.	RC.	Tile	RC.	144	46	11		11 fl.	⊥Broadway Broadway	C. 0.68 L. 0.44		0.45, 0.1* 0.1*		0.20 0.20	Light	Long narrow E.
151	One Eleven Sutter, San Francisco. Nov. 1, 1934.	Firm sand and clay.	RC. mat. 6,000	St.	Bk.	Tile	RC.	160	100	24	305	22 fl.	Montg Sutter	1.50 1.32		0.45, 0.05 0.40		0.75 0.75	Mod	U-shaped.
152	Pacific Southwest, Long Beach. Sept. 11, 1934.	Adobe over sand	RC. footing	RC.	Bk.	Tile	RC.	100	100	12	134	12 fl.	American Broadway	1.42 1.22	1.05	0.31, 0.19, 0.07 0.43, 0.31, 0.07		1.0 1.0	Light	Damaged in 1933 earthquake. Has not been repaired. U-shaped.
153	Palace Hotel, San Francisco. July 12, 1934.	Sand	Grillage footing	St.	Bk.	Tile and Bk.	RC.	343	264	9	130	9 fl.	Montg Market	0.72 0.72	 0.55	0.41, 0.20, 0.11 0.82?, 0.30, 0.20		0.30 0.20	Mod	U-shaped.
154	Postal Telegraph, San Francisco. Sept. 5, 1934.	Fill over marine clay.	RC. footing on piles.	St.	Bk.	Tile	RC.	90	80	11	150	11 fl.	Battery Pine	1.03 0.86		0.86, 0.4*, 0.07 0.41, 0.07		0.50 0.50	Light	Do.
155	Petroleum Securities, Los Angeles. Dec. 15, 1934.	Recent alluvium	RC. footing	RC.	Bk.	Tile	RC.	176	161	11	150	11 fl.	Flower ⊥Flower	0.93 1.00, 0.93		0.3* 0.50, 0.3*		0.15 0.15	Light	Do.
156	Procter & Gamble Finished Prod., Long Beach. Oct. 20, 1934.	Fill	RC. piles	RC.	RC.	None	RC.	254	127	6	111	6 fl.	⊥7th 7th	0.73 0.53		0.33* 0.33		0.07 0.07	Light	Do.
157	Professional, Long Beach. Oct. 26, 1934.	Adobe over sand	RC. footing	RC.	RC.	Tile	RC.	150	50	8	94	8 fl.	Pine 8th	0.60 0.34				0.20 0.07	Variable	Long and narrow U.
158	Robert Dollar, San Francisco. July 14, 1934.	Fill	RC. Cont. footing on piles.	St.	RC.	Tile	RC.	104	89	11	139	10 fl. wings and center.	Battery Calif.	0.80 1.08	0.50	0.15 0.21, 0.12, 0.17		0.35 0.25	Light	U-shaped. Connected to Dollar Annex which abuts Balfour, no. 133.
159	Roosevelt, Los Angeles. Dec. 4, 1934.	Recent alluvium	RC. footing [56]	St.	Bk., TC. F. s.	Tile	RC.	229	133	12	150	12 fl.	Flower 7th	1.42 1.41		0.45*		0.35 0.50	Very light	E-shaped.
160	School Administration, Long Beach. Aug. 29, 1934.	Soft alluvium	RC. footing. Some Cont. 5,000.	RC.	RC.	Tile	RC.	146	75	4	48	4 fl.	Locust ⊥Locust	C. 0.37 L. 0.31		0.1* 0.23		0.15 0.15	Light	Long and narrow U. Damaged by 1933 earthquake.
161	Security, Long Beach. Oct. 9, 1934.	do	RC. footing	St.	Bk.	Tile	RC.	150	75	13	167	13 fl.	Pine 1st	1.49 1.24		0.4* 1.0*, 0.35*		1.00 0.35	500	U-shaped. No apparent damage in 1933 earthquake.
162	Security Title, Los Angeles. Mar. 15, 1935.		do	St.	Bk.	Tile	RC.	130	120	13	156	13 fl.	Grand ⊥Grand	1.15 1.04		0.51, 0.3* 1.14, 0.38		0.25 0.30	Light	
163	Shreve, San Francisco. Mar. 6, 1935.	Sand	St. beam grillage. 8,000.	St.	Bk., S. F. s.	Tile	RC.	120	70	11	150	11 fl. ends	Grant Post	1.32 1.45	0.84	 0.14		1.00 2.00	1,600	U-shaped. Large windows.

See footnotes at end of table.

TABLE 3.—*Summary of building vibration data, May 1, 1934, to Mar. 31, 1935*—Continued

U- AND E-SHAPED BUILDINGS—Continued

No.	Building, city, date of observation	Soil	Foundation type and loading	Materials: Frame	Materials: Walls	Materials: Partitions	Materials: Floor	Dimensions: Length	Dimensions: Width	Dimensions: Height	Dimensions: Height	Vibration data: Position	Vibration data: Component. Direction or street	Periods: Fundamental: Translation	Periods: Fundamental: Torsion	Periods: Other	Periods: Extraneous	Maximum building displacement	Wind velocity	Remarks
			Lbs. per sq. ft.					*Feet*	*Feet*	*Stories*	*Feet*			*Seconds*	*Seconds*	*Seconds*	*Seconds*	*0.001 inch*	*Beaufort scale or ft./min.‡*	
164	Southern Pacific, San Francisco. June 11, 12, 13, 1934.	Fill over mud and blue clay.	RC. cyl. footings on piles.	St.	Bk.	ML. and P.	RC.	275	209	14	197	14, 11, 10, 7, 4 fl.	Spear Market	1.13 1.20	1.00[57]	0.45, 0.30, 0.21. 0.64, 0.35, 0.27, 0.20.		1.50 1.50	Mod.	E-shaped. Long piles through soft ground.
165	State, San Francisco. Nov. 19, 1934.	Sand	RC. footing	St.	Bk. and S. F., RC.[58]	Tile	RC.	375	110	6	90	6 fl.	Larkin McAllister	0.31 0.46		 0.27		0.15 0.15	Light	E-shaped.
166	Syndicate, Oakland. Dec. 17, 1934.	Sandy clay	do	St.	RC. and Bk.	ML. and P.	RC.	123	69	10	135	10 fl. wings and center.	Broadway 14th	0.56 0.77		0.4 * 0.35, 0.3 *	1.30[59] 0.98[59]	0.25 0.50	Light	U-shaped.
167	Title Guarantee, Los Angeles. Mar. 18, 1935.		do	St.	Bk.	Tile	RC.	119	86	12	150	10 fl.	Hill 5th	1.19 1.00		0.28, 0.1 * 0.50, 0.09			900	Do.
168	Washington, Los Angeles. Dec. 6, 1934.	Recent alluvium	do	St.	Bk.	Tile	RC.	130	85	13	150	13 fl.	Spring 3rd	1.18 0.94		1.4 *, 0.38 1.12, 0.38		0.50 0.50	Very light	Do.

FLAT-IRON TYPE BUILDINGS

No.	Building, city, date of observation	Soil	Foundation type and loading	Frame	Walls	Partitions	Floor	Length	Width	Stories	Feet	Position	Component	Translation	Torsion	Other	Extraneous	Max. displacement	Wind velocity	Remarks
169	American, Oakland. Dec. 5, 1934.	Sandy clay	RC. mat	St.	RC.	ML. and P.	RC.	80	69	11	145	11 fl.	Long. § Cross	0.89 1.06	0.5 *	0.27 0.33		0.50 1.00	1,150	Angle about 65°.
170	Broadway, Oakland. Dec. 17, 1934.	Sand and clay	RC. footing	RC.	RC., TC. F. s.	ML. and P.	RC.	135 sides.		8	115	8 fl.	Long. § Cross	0.66 0.76	0.45	0.22		0.25 0.50	Light	Angle about 40°.
171	Crocker, San Francisco. Nov. 5, 1934.	do	Spread footings	St.	Bk.[60]	Tile	RC.			11	189	11 fl.	Long. § Cross	0.83 1.00		0.25 * 0.3 *		0.50 0.25	Light	Do.
172	Federal Realty, Oakland. Dec. 17, 1934.	do	RC. Comb. footing.	St.	RC., TC. F. s.	ML. and P.	RC.	104	35,[61] 9[62]	13	177	12 fl.	Long. § Cross	1.06 1.70	 0.9 *	1.7 *, 0.25 * 0.10 *		0.50 1.50	Light	Angle about 15°.

T-, H-, AND IRREGULAR-SHAPED BUILDINGS

No.	Building, city, date of observation	Soil	Foundation type and loading	Frame	Walls	Partitions	Floor	Length	Width	Stories	Feet	Position	Component	Translation	Torsion	Other	Extraneous	Max. displacement	Wind velocity	Remarks
173	Barbara Worth, El Centro. Feb. 20, 1935.	Silt	RC.	RC.[63]	RC.[63], Bk.[63]	Tile and W.	W.			5 and mezz.		4 and 5 fl.	7th Main 7th Main	0.37 0.37 0.26 0.28				0.20 0.20 0.20 0.20	Very light	New part. New part. Old part. Old part.
174	Bekins Storage, Oakland. Dec. 21, 1934.	Clay	RC. footing	RC.	RC.	Few	RC.	175	60	7	85	7 fl.	⊥22d 22d	0.55 0.28		0.22		0.05 0.05	Light	5-sided building. Skew corner.
175	Exchange Block, San Francisco. Jan. 10, 1935.	Sand and clay	Footing on brick piers.[64]	RC.	RC., Bk. F. s.	Tile, ML. and P.	RC.			8	99	8 fl.	Montg Pine	0.54 0.61		0.10 0.10		0.50 0.35	Light	H-shaped.
176	Fife, San Francisco. Dec. 26, 1934.	Fill over marine clay.	Steel grillage on piles.	St.	RC., Bk. F. s.	ML. and P.	RC.	80	50	12	155	12 fl.	Drumm Calif	1.04 1.35		0.35 * 1.0 *, 0.2 *		0.50 1.00	Mod.	Front and back parallel, 1 side at right angles, other side at 45°.
177	Four Fifty Sutter, San Francisco. May 24, 25, 26, 31, 1934; June 1, 2, 1934.	Sand and clay	RC. footing. 8,000.	St.	RC., TC. F. s.	Tile	RC.	188	138	26	340	All fl.	Powell Sutter	1.36 1.55	1.00	0.45, 0.25 0.48, 0.27, 0.20.		1.00 1.00	Light to mod.	Rectangular up to seventh floor. T-shaped from seventh to top.
178	Foxcroft, 68 Post, San Francisco. Nov. 1, 1934.	Sandy clay	RC. footing	St.	Bk.	Tile	RC.	123	78	8	105	8 fl.	Kearny Post	0.67 0.85	0.48	0.13, 0.09 * 0.75, 0.12	0.5[65] 0.5 *	0.25 0.25	Mod.	Very irregular shaped.
179	Horace Mann, Pasadena Junior College. Aug. 24, 1934 and Mar. 1, 1935.	Gravel	do	RC.	None	None	RC.			3		3 fl.								Being rebuilt. See table 7.
180	Kohl, San Francisco. Dec. 28, 1934.	Sand and clay	do	St.	Bk., S. F. s.	ML. and P.	RC.	100	100	11	140	11 fl. ends	Montg Calif	0.97† 1.05 †	0.66 † 0.7 †	0.11		0.50 0.60	Gentle	H-shaped. Very irregular motion.
181	Lakeside Comm. Apartments, Oakland. Dec. 5, 1934.	Silt over hard clay.	Cofferdam to clay	St.	RC.	Tile	RC.			12	140	Roof	Harrison ⊥Harrison	0.57 0.51		0.10 0.08		0.25 0.25	1,500 to 3,000	H-shaped.

See footnotes at end of table.

TABLE 3.—*Summary of building vibration data, May 1, 1934, to Mar. 31, 1935*—Continued

T-, H-, AND IRREGULAR-SHAPED BUILDINGS—Continued

No.	Building, city, date of observation	Soil	Foundation type and loading	Materials				Dimensions				Vibration data							Wind velocity	Remarks
				Frame	Wells	Partitions	Floor	Length	Width	Height		Position	Component. Direction or street	Periods				Maximum building displacement		
														Fundamental		Other	Extraneous			
														Translation	Torsion					
			Lbs. per sq. ft.					*Feet*	*Feet*	*Stories*	*Feet*			*Seconds*	*Seconds*	*Seconds*	*Seconds*	*0.001 inch*	*Beaufort scale or ft./min.$*	
182	Los Angeles Athletic Club, Los Angeles. Mar. 18, 1935.		RC. footing	St.	Bk.	Tile	RC	156	145	12	150	12 fl.	Olive 7th	1.31 1.31		0.22 0.47		1.00 0.75	500	Every other floor is a mezzanine.
183	Municipal Auditorium, Long Beach. Aug. 30, 1934.	Fill	RC. piles	St. and RC.	RC	Tile and RC.	RC	277	172	9	105	Top fl. at ends	NS EW	0.33 0.33		0.20 0.2*		0.10 0.10	300	Auditorium and exhibition hall. Large open spaces, balconies, etc.
184	Pacific Mutual, Los Angeles. Dec. 4, 1934.		RC. footing	St.	Bk.	Tile	RC	207	162	11	150	11 fl.	⊥6th 6th	1.27 1.15	1.08	1.0*, 1.2* 0.4*, 1.0*, 0.2*.		0.35 0.40	100	†.
185	Pellisier, Los Angeles. Feb. 13, 1935.		RC. piles	St. and RC.	Concrete, TC. F.	Tile	RC	86 [66]	51 [66]	12	150	12, 8, 6 fl.	NW NE	0.82 0.91	 0.50	0.10* 0.10*		0.30 0.75	1,700	High building on one corner of theatre.
186	Procter and Gamble, All Process Building, Long Beach. Oct. 20, 1934.	Fill	do	St.	RC	RC	Steel grill [67]	454	102	6	110	2, 5 fl.	⊥7th 7th	0.33 0.53, 0.36				0.07 0.07	Light	
187	Providence Hospital, Oakland. Sept. 8, 1934.	Firm clay	RC	RC	RC	Some RC	RC	150 (wing length).		6 and rotunda.	100	Roof wings and rotunda.	NS [68] EW	0.25 [69] 0.30 [69], 0.36				0.05 0.15	Gentle	X-shaped.
188	Ray, Oakland. Dec. 5, 1934	Clay	RC. footing	RC	RC., Bk. F. s.	ML. and P.	RC	120	96	10 and mezz.	140	10 fl.	Broadway 19th	0.67 0.55		0.24*, 0.07 0.07		0.25 0.25	1,350	T-shaped above second floor.
189	Rialto, San Francisco. Oct. 31, 1934.	Sand	do	St.	Bk.	ML. and P.	RC	160	106	8	105	8 fl. ends	New Montg. Mission	1.22 1.22	0.80	1.15, 0.4		0.50 2.50	Mod	H-shaped.
190	Royal Hotel, Oakland. Dec. 21, 1934.	Clay	do	RC	RC	ML. and P.	RC	60	42	7	90	7 fl.	San Pablo ⊥San Pablo	0.54 0.42		0.42 0.26		0.20 0.30	Light	6 sides, skew corner.
191	Stewart, Oakland. Dec. 17, 1934.	Sandy clay	do	St.	Bk.	W, L and P.	St. and W.	75	65	6	85	6 fl.	San Pablo ⊥San Pablo	0.60 0.34		0.25, 0.09 0.60, 0.08		0.10 0.10	Light	5 sides, skew corner.
192	Subway Terminal, Los Angeles. Aug. 16, 17, 1934.	Blue shale	do	St.	Bk.	Tile	RC	324	141	12	150	10, 11 fl.	Hill 4th	0.86, 0.69 0.72		0.55, 0.35, 0.28, 0.2*. 0.58, 0.35, 0.20.		0.40 0.30	Gentle	Building has 1 main part. 4 wings south, 1 north.†
193	Villa Riviera, Long Beach. Sept. 5, 1934.	Soft alluvium	RC. footing. 5,500	St.	RC	Tile	RC	230	50	15	184	15 fl.	NW-SE NE-SW	C. 1.25 L. 0.83				0.70 0.35	1,050	2 wings join at about 145°. Heavy surf running during test.
194	Rowan, Los Angeles. Mar. 13, 1935.		RC. footing	St.	Bk.	Tile	RC	235, 95	160	12	150	12 fl.	Spring 5th	1.85 1.3*†	1.56 1.50	0.50*, 0.29, 0.22. 0.5(?), 0.2		0.75 0.75	Light	Hollow square with back wing.†
195	Southwestern University, Los Angeles. Mar. 20, 1935.		do	RC	RC	Tile	RC	145	50	10	118	10 fl.	Hill 11th	0.84 0.49 [70]		0.6*, 0.25 0.62		0.50 0.25	Light	H-shaped.
196	Warner's Theater, Los Angeles. Mar. 15, 1935.		do	RC. and St.	Bk., TC. F.	Tile	RC	150 x 30-40.	113 x 35	9	100	9 fl.	Hill 7th	0.50 0.56, 0.51		0.23(?), 0.1*, 0.07. 0.45, 0.22, 0.11.		0.25 0.25	Light	Office part in front of theater.
197	Women's City Club, Berkeley. Dec. 18, 1934.	Alluvium	do	RC	RC	ML. and P.	RC	120	112	6	100	6 fl.	⊥Durant Durant	0.23 0.23		 0.08		0.05 0.05	Light	Irregular-shaped. Attached wings, auditorium, etc.
198	Y. M. C. A., San Francisco. Nov. 19, 1934.	Sand	do	St.	RC. and Bk.	ML. and P.	RC	137	137	9	101	8, 9 fl.	Leavenworth Golden Gate	0.72 0.60		0.43 [71] 0.27, 0.17		0.25 0.25	Gentle	Very irregular shape, with open courts, etc.
199	Building and city not named. Jan. 8, 1935.	Firm clay and sand.	Grillage footings	St.	Bk.	ML. and P.	RC	280	157	10		10 fl.	NS EW	1.25 0.72	1.04(?)	0.53, 0.40, 0.10 0.41, 0.3*, 0.2, 0.1.	 1.41 [72]	0.5 0.4	Light	Adjacent to no. 127.†
200	Y. M. C. A. Hotel, San Francisco. Nov. 20, 1934.	Sand	RC. footing	St.	Bk.	ML. and P.	RC	137	83	12	124	12 fl.	Leavenworth Turk	0.56 0.56		0.3* 0.27		0.10 0.25	Mod	H-shaped, narrow wings.

See footnotes at end of table.

TABLE 3.—*Summary of building vibration data, May 1, 1934, to Mar. 31, 1935*—Continued

HOLLOW RECTANGULAR BUILDINGS

No.	Building, city, date of observation	Soil	Foundation type and loading	Materials: Frame	Materials: Walls	Materials: Partitions	Materials: Floor	Dimensions: Length	Dimensions: Width	Dimensions: Height		Vibration data: Position	Vibration data: Component. Direction or street	Periods: Fundamental: Translation	Periods: Fundamental: Torsion	Periods: Other	Periods: Extraneous	Maximum building displacement	Wind velocity	Remarks
			Lbs. per sq. ft.					*Feet*	*Feet*	*Stories*	*Feet*			*Seconds*	*Seconds*	*Seconds*	*Seconds*	*0.001 inch*	*Beaufort scale or ft./min.‡*	
201	Citizen's Bank, Los Angeles. Mar. 13, 1935.		RC. footing	St	Bk. and TC.	Tile	RC	154	115	12	148	12 fl.	Spring 5th	1.38 1.40	0.77 0.76	0.48 0.48		1.25 0.50	Light	
202	Adam Grant, San Francisco. Dec. 27, 1934.	Hard clay under fill.	RC. caissons under steel grillage.	St	Bk	Tile	RC	137	137	14, 6	184	12, 6 fl.	Bush Sansome	0.82 1.07	0.65	0.4*, 0.13 0.41, 0.3*, 0.1*.	1.85 [73] 1.8*[73]	0.50 0.50	Gentle	
203	Federal Office, San Francisco. July 11, Aug. 27, and Nov. 11, 1934, and Apr. 24, 1935.	Sand	RC. footing. 6,000	St	Bk. and S.	(74)	RC	358, 243 [75]	210, 89 [75]	5	85	5 fl.	Leavenworth McAllister	(74) (74)						
204	Ferguson, Los Angeles. Feb. 25, 1935.		RC. footing	RC	Bk. and TC.	Tile	RC	90	60	8	89	8 fl.	2d Hill	0.66 0.89	0.6*	0.22, 0.10 0.28, 0.10		0.20 0.30	450	
205	H. W. Hellman, Printing Trades, Los Angeles. Mar. 26, 1934.		do	St	Bk	Tile	RC	198	124	8	120	8 fl.	Spring 4th	1.40 1.17		0.17 0.8*, 0.40		0.75 0.50	Light	
206	Merchants Exchange, San Francisco. Nov. 21, 1934.	Fill over clay	Grillage over piles	St	Bk	ML. and P.	RC	149 [76]	122	14	190	13 fl.	Montg Calif	1.35 1.45		0.8*, 0.26 0.8(?), 0.27		1.00 1.50	Gentle	Abuts Bank of America, No. 96.
207	Public Library, San Francisco. Oct. 29, 1934.	Sand	RC. Conn. footing	St	Bk., S. F.	Tile	RC	304	185	3	78	3 fl. west side and wing.	Larkin McAllister	0.38 0.36		0.31, 0.25 [77], 0.9(?). 0.25(?), 0.1*				High floors.
208	Sheldon, San Francisco. Jan. 11, 1935.	Fill over marine clay.	RC. footing on piles.	RC	RC	ML. and P.	RC	150	100	8	99	8 fl.	1st Market	0.55 0.70	0.4*	0.7*, 0.11 0.73, 0.55, 0.11.		0.25 0.25	Light	
209	Sutter Hotel, San Francisco. Nov. 2, 1934.	Sandy clay	RC. footing	St	Bk. and RC.	ML. and P.	RC	122	98	8	99	8 fl.	Kearny Sutter	0.55 0.87		 0.30		0.10 0.15	Gentle	
210	Wells Fargo, San Francisco. Nov. 21, 1934.	Sand over blue clay.	Spread grillage	St	Bk. and RC.	ML. and P.	RC	160	120	8	115	7, 8 fl.	2d Mission	0.76 0.85	0.56	0.25*		0.35 0.35	Gentle	

MISCELLANEOUS

No.	Building, city, date of observation	Soil	Foundation type and loading	Frame	Walls	Partitions	Floor	Length	Width	Stories	Feet	Position	Component. Direction or street	Translation	Torsion	Other	Extraneous	Maximum building displacement	Wind velocity	Remarks
211	Southern Sierras Terminal Station, El Centro, Feb. 19, 1935.	Silt	RC. footing	RC	RC	RC	RC	80	60			1 fl.	NS EW	0.36 0.37			0.07 [78]	0.04 0.10	Light	(78)
212	Aztec Brewery Tank House, San Diego. Feb. 21, 1935.			RC	RC	None	RC					2 fl. basement	S. Main ⊥S. Main	0.30 [79] 0.30 [79]				[79] 0.01 [79] 0.01	Light	Tank house. Very heavy RC. structure.

1 Above first floor.
2 Top of steel in tower. Second floor extension in back.
3 Same period in basement.
4 Wood studs in partitions.
5 Connected buildings cover large area. Much extraneous motion due to machinery.
6 Plant not running.
7 Very prominent period.
8 Floors about double normal spacing.
9 RC. to first floor.
10 Probably due to machinery.
11 Anchor bolts into stone.
12 Tower part.
13 May be ground period.
14 Probably from buildings nos. 158 and 30.
15 Partitions are concrete on expanded metal.
16 West wall is brick.
17 Above third floor.
18 Under wall.
19 Steel studs and bracing in cross-partitions.
20 Probably due to presses running.
21 Very prominent.
22 Irregular motion.
23 Brick filler in front.
24 Due to machine.
25 From Shell Building.
26 Most prominent translation period.
27 Hollow tile above third.
28 Terra cotta and granite face.
29 RC. wall in back.
30 Brick bearing walls.
31 Most prominent.
32 Height of tower.
33 Both prominent.
34 Tower.
35 Southeast wing.
36 Walls on court.
37 RC. piles under end walls.
38 Above eighth floor.
39 Tower dimensions above wings.
40 Dome. Large area building with central dome and 2 open courts.
41 Clock level in tower.
42 Probably due to tower.
43 Period in low wings. Foundation under tower heavier and deeper than under wings.
44 Probably transmitted from tower.
45 Dimensions above sixth floor.
46 Building has 3 street levels and 1 mezzanine.
47 Sixteenth to twenty-second floor.
48 Twenty-second to thirtieth floor.
49 Probably from no. 199.
50 Above tenth floor.
51 Printing press running.
52 Torsional period in wings only.
53 New parts.
54 Steel frame for water tank.
55 Periods of south wing 1.06 and 0.91; center 0.91; north wing 0.86.
56 Piles under part of building.
57 Wings only.
58 Brick with stone face on front and sides. RC. in back.
59 Probably due to abutting Central Bank Building, no. 101.
60 Old building, self-sustaining walls, hung floors.
61 Width in back.
62 Width in front.
63 New part of building is RC. Old part is brick bearing wall and wood.
64 Foundation of previous building.
65 Appears only adjacent to 6-floor building.
66 High part of building.
67 Floors generally omitted. Steel bracing added to give horizontal rigidity.
68 Parallel to wings in approximately this direction.
69 Both components.
70 Most prominent period. Component parallel to longitudinal axis of building.
71 Very prominent.
72 Probably from no. 127. Two courts, connecting cross-arm of building.
73 Probably due to Shell Building, no. 128.
74 Under construction. See table 7.
75 Dimensions of court.
76 Does not include back 1-floor extension.
77 Wing only. Vibration type unknown.
78 Synchronous condensers running. Very heavy foundation.
79 Same period and amplitude in second floor and basement.

(*b*) Measurements on each floor of 1 or 2 buildings and on several floors of a few other selected buildings.

(*c*) Simultaneous measurements of periods on several of the floors or at different parts of the same floor of a building to be made in a limited number of cases.

(*d*) Measurement at different heights of buildings having variable cross sections.

(*e*) Measurement of the periods of certain buildings undergoing construction at various stages of construction.

(*f*) Use of a building vibrator to put buildings into controlled vibration. Items (*c*) and (*f*) are covered in other chapters.

Prior to April 1, 1935, the periods of 212 buildings in California have been investigated. These include 48 buildings in Los Angeles, 74 in San Francisco, 30 in Oakland, 29 in Long Beach, 6 in Berkeley, 5 in San Jose, 4 in Pasadena, 3 in Palo Alto and Stanford University, 1 in Crockett, 1 in Huntington Park, 1 in Vernon, and 2 in El Centro. Two of the buildings are being covered in item (*f*).

Table 3, which was prepared by Mr. M. P. Taylor, lists certain structural and vibration data for these buildings, which are grouped

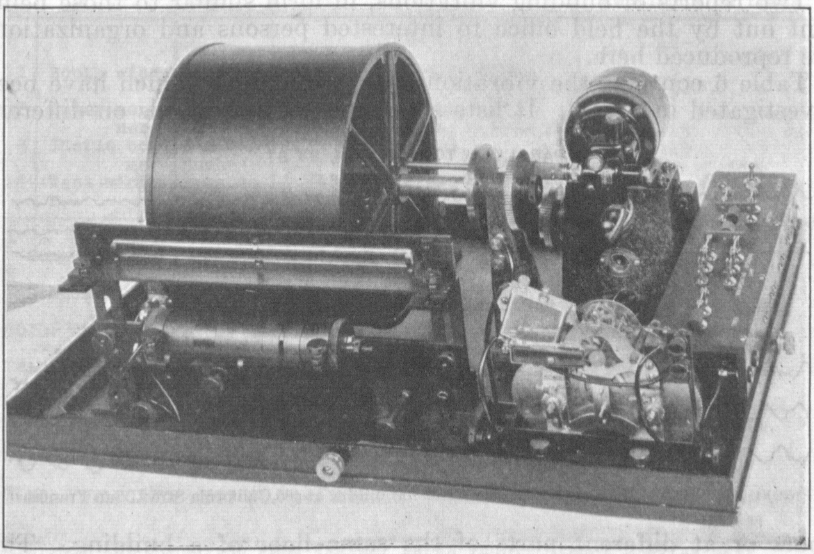


FIGURE 34.—Vibration meter recorder.

according to the shape of their cross sections. Three of the buildings are referred to by number, as their owners or managers object to the names being published. Two of these have interconnecting hallways, and are under the same management, although the frames of the buildings are independent. The building data in table 3 in many cases were obtained directly from the architects or designing engineers. In routine work this information was obtained when possible from the building managers and engineers, and the dimensions often were estimated from the street. An important source of information is a report of the subsoil committee of the San Francisco section, American Society of Civil Engineers, September 1932, entitled "Subsidence and Foundation Problem in San Francisco." Detailed maps of soil surveys likewise are important sources of subsoil data.

The periods are those actually read from the records, or the averages where several records have been taken. The item of judgment enters all the measurements. The periods listed are those which, to the persons making the measurements, are the most representative values of the records. The amplitudes listed represent the maximum displacement of the building from the rest position in thousandths of an inch at the time of the observation.

Figure 35 is a reproduction of a vibrogram of the building at 485 California Street. The time scale is marked by the dashed line. The distance between corresponding parts of consecutive dashes represents a half second. The following facts are illustrated in this record: (1) The fundamental periods of vibration are distinctly different in the two directions; (2) the fundamental mode of one component may underlie or be superposed upon the record of the other; (3) shorter waves, probably written by secondary modes of vibration, may be superposed upon the record of the fundamental mode; and (4) the measurement of a single wave on the record often gives an unreliable picture of the true motion.

Two reports of building vibrations, in form similar to those being sent out by the field office to interested persons and organizations are reproduced here.

Table 6 contains the vibration data of buildings which have been investigated in detail. It lists slightly different periods on different

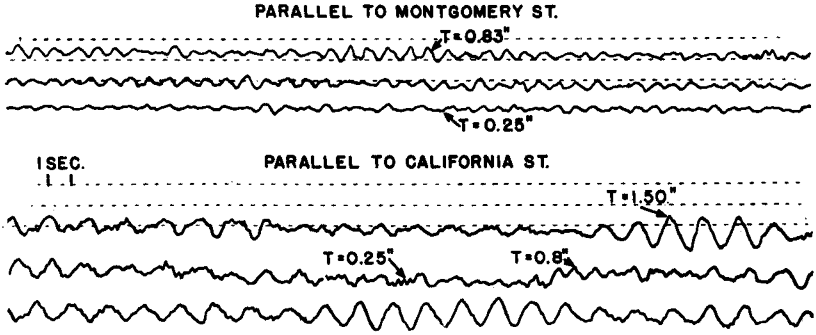


FIGURE 35.—Typical vibrogram. Observations in building at 485 California Street, San Francisco.

floors or at different parts of the same floor of a building. This condition is not to be interpreted as due to a real difference in period from part to part of a building, even though the difference may be slight, nor can it be attributed to instrumental errors. The fact that the observations at different parts of the building were taken at different times is sufficient to account for slight discrepancies in observed periods. The few simultaneous measurements that have been taken show that the period of the fundamental vibration of a building is the same (in any particular direction) at various parts of the building at the same instant, but that it may differ slightly from time to time, partially due, perhaps, to inequalities of structural conditions in the building; for instance, a stiff wall may be brought into play in one instant and not so much in another.

Preliminary Report

DEPARTMENT OF COMMERCE
U. S. COAST AND GEODETIC SURVEY
CALIFORNIA SEISMOLOGICAL PROGRAM

BUILDING VIBRATION TESTS

CITY: San Francisco. STREET AND NO.: Bush and Sansome Streets.

BUILDING: Standard Oil Co. HEIGHT: Stories, 21; Feet, 309.

OBSERVATIONS: DATE: 9/4/34. FLOOR: 20th. LOCATION: As shown on plan.

INSTRUMENTS: Spring recorder; Survey vibration meter. OBSERVERS: DSC, MT.

STATIC MAGNIFICATION: 190. DAMPING: 50. PENDULUM PERIOD: 2.5 sec.

TABLE 4. PERIODS AND AMPLITUDES IN STANDARD OIL COMPANY BUILDING, SAN FRANCISCO

No.	Location	Component	Periods	Maximum single amplitude on trace
			sec.	in.
1	South wing	Sansome	1.8±0.1; 1.2±0.1; 0.6ca	.2
		Bush	1.5±0.1; 0.5?	.2
2	Outer corner	Sansome	1.8±0.1; 1.5 ca; 0.6?	.3
		Bush	1.5±0.1; 1.3 ca; 0.5 ca	.15
3	Inside corner	Sansome[1]	1.75±.02;1.5?; 0.5?	.25
		Bush[2]	1.7±0.1; 1.4?; 0.5?	.2
4	West wing	Sansome	1.8±0.1; 1.2 ca; 0.4?	.4
		Bush	1.5±0.1; 1.2?; 0.4?	.25

1 Component 10° W. of Sansome St.

2 Component 10° N. of Bush St.

SOIL: Sand and sandy clay. FOUNDATION: Continuous, reinforced material. Concrete piles under each end wall.

FRAME: Steel.

WALLS: Brick and terra cotta. PARTITIONS: Tile. FLOORS: Reinforced concrete.

WEATHER CONDITIONS: Before tests: Fair. During tests: Fair.

WIND: VELOCITY: 600 ft. per minute. DIRECTION: West. CHARACTER: Very gusty.

DISTANCE AND DIRECTION FROM NEAREST KNOWN FAULT: About 4 miles east.

ALLOWABLE BEARING VALUE OF SOIL: 4000 lbs. design. Tests up to 15,000 lbs./sq. ft.

RELATION TO OTHER BUILDINGS: 2-story abutting on south, 5-story abutting on west.

REMARKS: Wind very irregular.

Set-ups 1, 2, 4,—components parallel to building sides.

PLAN OF SET-UPS:

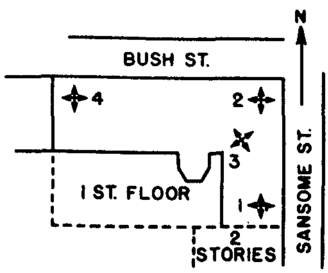


FIG.36

Preliminary Report

DEPARTMENT OF COMMERCE
U. S. COAST AND GEODETIC SURVEY
CALIFORNIA SEISMOLOGICAL PROGRAM

BUILDING VIBRATION TESTS

CITY: Los Angeles. STREET: Hill Street between Fourth and Fifth.

BUILDING: Subway Terminal. HEIGHT: 12 stories; 150 ca ft.

OBSERVATIONS, DATE: 8/16, 17/34. FLOORS: 10, 11. LOCATION: See plan of setups.

INSTRUMENTS: Wood-Anderson seismometer; Spring recorder.

OBSERVERS: RSMcL, WmWM. STATIC MAGNIFICATION: 1400. DAMPING: 6.

PENDULUM PERIOD: 2.5 sec.

TABLE 5. PERIODS AND AMPLITUDES IN SUBWAY TERMINAL BUILDING, LOS ANGELES

No.	Component	Periods	Maximum single amplitude on trace
		sec.	in.
A[1]	Hill	0.69±.02; 0.28±.02	0.1
	4th	0.56±.01; 0.35±.01	0.1
Room 1148[1]	Hill	0.68±.02; 0.57±.01; 0.35ca; 0.2?	0.2
	4th	0.67±.01; 0.58±.01; 0.2?	0.4
B[1]	Hill	0.70±.02; 0.54±.03	0.3
	4th	0.73±.02; 0.56±.02; 0.2ca	0.1
C[1]	Hill	0.87±.02; 0.5 ca; 0.2 ca	0.6
	4th	0.7ca; 0.54±.01; 0.20±.01	0.1
Room 1112[1]	Hill	0.86±.01; 0.55?	0.4
	4th	0.72±.01	0.5
D[2]	Hill	0.85±.01; 0.55?	0.2
	4th	0.57±.01; 0.3ca	0.1
Room 1127[1]	Hill	0.86±.01; 0.55±.02; ; 0.2ca	0.3
	4th	0.75ca; 0.58-.03	0.2
10th floor North wing[2]	Hill	0.66±.03: 0.50±.02; 0.2ca	0.2
	4th	0.60±.02; 0.2ca	0.3

[1] Observed on August 16, 1934. [2] Observed on August 17, 1934.

SOIL: Blue shale. FOUNDATION: Concrete. FRAME: Steel. WALLS: Brick.

PARTITIONS: Clay tile. FLOORS: Concrete.

WEATHER CONDITIONS: BEFORE TESTS: Clear. DURING TESTS: Clear.

WIND: VELOCITY: Low. DIRECTION: South. RELATION TO OTHER BUILDINGS: Independent.

PLAN OF SET-UPS:

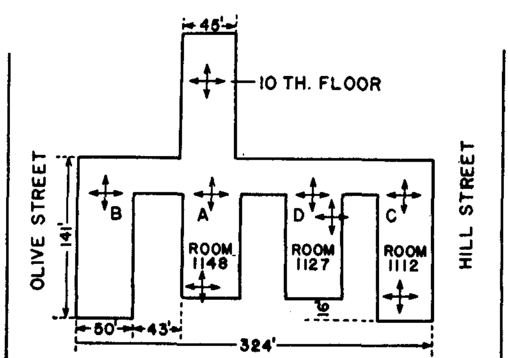


FIG. 37

TABLE 6.—*Vibration data for buildings in which observations were made on several floors*

Shell Building, Bush and Battery Streets, San Francisco. [Components [1]: (a) Battery Street, (b) Bush Street.]

Date 1934	Instrument location	Mean values of periods [2]	Building displacement [2]
		Seconds	*0.001 inch*
May 9	Roof	(a) 1.87; 0.62; 0.36 (b) 1.91; 0.67; 0.35	1.7 1.7
9	Twenty-ninth floor	(a) 1.91; 0.66; 0.35 (b) 1.87; 0.69; 0.35; 0.24	1.4 1.3
May 14	Twenty-eighth floor	(a) 1.91; 0.67; 0.38 [4] (b) 1.88; 0.67; 0.38	3.8 1.1
May 7	Twenty-seventh floor	(a) 1.94; 0.64; 0.4 [3] (b) 1.93; 0.64; 0.44	4.2 4.1
7	Twenty-sixth floor	(a) 1.96 (b) 1.96	4.6 4.7
9	Twenty-fifth floor	(a) 1.97; 1.90; 0.62 (b) 1.89; 0.69	1.3 1.1
9	Twenty-fourth floor	(a) 1.93; 0.25 (b) 1.92; 0.23	1.4 2.2
9	Twenty-third floor	(a) 1.88; 0.2 [3] (b) 1.92	1.8 2.2
9	Twenty-second floor	(a) 1.90; 0.35 (b) 1.94; 0.35 [4]	1.6 1.7
8	Twenty-first floor	(a) 1.88; 0.35 (b) 1.91	1.1 2.1
8	Twentieth floor	(a) 1.90; 0.35 (b) 1.92; 0.35	1.4 1.4
8	Ninteenth floor	(a) 1.90; 0.38 (b) 1.90; 0.39	1.7 2.6
8	Eighteenth floor	(a) 1.90; 0.40 (b) 1.90; 0.68; 0.40; 0.22	1.0 0.7
4	Seventeenth floor	(a) 1.90; 0.63; 0.42 (b) 1.88; 0.65; 0.42	0.6 0.9
3	Sixteenth floor	(a) 1.91; 0.63; 0.42 (b) 1.89; 0.63; 0.42; 0.21	0.6 0.6
3	Fifteenth floor	(a) 1.93; 0.63; 0.21 (b) 1.89; 0.41; 0.21	0.4 0.6
3	Fourteenth floor	(a) 1.94; 0.64; 0.46 (b) 1.90; 0.67; 0.45	0.6 0.3
3	Thirteenth floor	(a) 1.93; 0.64 (b) 1.94	1.4 1.1
3	Twelfth floor	(a) 1.94; 0.67 (b) 1.90; 0.69; 0.41	0.9 0.5
4	Eleventh floor	(a) 1.88; 0.64; 0.40 (b) 1.86; 0.70; 0.38	0.9 1.0
4	Tenth floor	(a) 1.91; 0.66 (b) 1.89; 0.69; 0.38	0.3 0.6
4	Ninth floor	(a) 1.90; 0.67; 0.42 (b) 1.93; 0.72; 0.42	0.5 0.4
4	Eighth floor	(a) 1.90; 0.67; 0.42 (b) 1.91; 0.68; 0.42	0.4 0.9
4	Seventh floor	(a) 1.93; 0.67; 0.37 (b) 1.94; 0.78; 0.39	0.3 0.6
14	Sixth floor	(a) 1.93; 0.67; 0.37 (b) 1.89; 0.70; 0.40	1.2 1.0
4	Fifth floor	(a) 1.91; 0.66; 0.35 (b) 1.92; 0.74; 0.47	0.1— 0.3
4	Fourth floor	(a) 1.95; 0.70; 0.40 (b) 1.91; 0.72	0.1— 0.1—
8	Third floor	(a) 1.85; 0.67; 0.37 (b) 1.95; 0.38	0.2 0.2
8	Second floor	(a) 1.88; 0.70; 0.50; 0.25 (b) 1.92; 0.74; 0.41	0.2 0.2
16	First floor	(a) 1.95; 0.70; 0.50 (b) 1.93; 0.70	0.1— 0.1—
8	Basement, center	(a) 0.8 [3]; 0.6 [3]; 6.2 [11] (b) ([4]); 6.2 [11]	0.1— 0.1—
7	Subbasement, center	(a) 0.76; 0.49 (b) 0.82; 0.53	0.1— 0.1—

See footnotes at end of table.

TABLE 6.—*Vibration data for buildings in which observations were made on several floors.*—Continued

Edison Building, 5th Street and Grand Avenue, Los Angeles. [Components: (a) 5th Street, (b) Grand Avenue.]

Date 1934	Instrument location	Mean values of periods [2]	Building displacement [2]
		Seconds	*0.001 inch*
Aug. 7	Twelfth floor, east end of hall	(a) 0.78; 0.26; 0.07 [3]	0.1
		(b) 0.65; 0.3 [3]	0.1
7	Tenth floor, south of center	(a) 0.77; 0.3 [3]	0.1
		(b) 0.66; 0.2 [4]; 0.06 [3]	0.1
7	Sixth floor, center of south side	(a) 0.26	0.1
		(b) 0.67; 0.25	0.1
7	Fourth floor, southwest corner	(a) 0.77 [4]; 0.26	0.1
		(b) 0.65; 0.26	0.1
7	Third floor, near northwest corner	(a) 0.26	0.1
		(b) 0.67 [4]; 0.26	0.1−
7	Second floor, near southwest corner	(a) 0.26	0.1−
		(b) 0.66; 0.26	0.1−
7	First floor, auditorium center	(a) 0.26	0.1−
		(b) 0.26	0.1−

Four Fifty Sutter Building, 450 Sutter Street, San Francisco. [Components: (a) Powell Street, (b) Sutter Street.]

Date 1934	Instrument location	Mean values of periods [2]	Building displacement [2]
May 25	Twenty-eighth floor, east stair landing	(a) 1.36; 0.44; 0.27	1.1
		(b) 1.55; 0.97; 0.51; 0.32 [4]; 0.21	1.2
25	Twenty-seventh floor, east stair landing	(a) 1.37; 0.46; 0.26	0.7
		(b) 1.55; 1.10; 0.49	1.2
25	Twenty-sixth floor, north wing	(a) 1.33; 0.42	0.6
		(b) 1.52; 0.95; 0.47	0.8
24	Twenty-sixth floor, southwest wing	(a) 1.34; 1.26; 1.00; 0.47 [4]; 0.32 [4]	1.3
		(b) 1.53; 0.97; 0.46 [3]	0.7
25	Twenty-fifth floor, north wing	(a) 1.34; 0.44	0.7
		(b) 1.55; 0.99; 0.48 [4]	1.1
25	Twenty-fourth floor, east end of south hall	(a) 1.36	1.8
		(b) 1.57; 0.49 [3]	1.3
25	Twenty-third floor, north wing	(a) 1.37; 0.46; 0.2 [3]	0.7
		(b) 1.56; 1.01	1.4
25	Twenty-second floor, west end of south hall	(a) 1.38; 1.33; 0.24 [3]	0.7
		(b) 1.55; 1.00	0.8
26	Twenty-first floor, north wing	(a) 1.36; 0.24	0.6
		(b) 1.51; 1.00; 0.23 [3]	0.6
26	Twentieth floor, east end of south hall	(a) 1.35; 0.25	0.8
		(b) 1.56; 1.00; 0.27 [4]	0.5
26	Nineteenth floor, north wing	(a) 1.44 [4]; 1.32; 0.45; 0.25	0.6
		(b) 1.59; 0.99; 0.54 [4]; 0.21 [4]	0.8
31	Eighteenth floor, west end of south hall	(a) 1.35; 1.00 [4]; 0.24	1.1
		(b) 1.53	1.7
31	Seventeenth floor, north wing	(a) 1.36; 1.30; 0.26	1.5
		(b) 1.59; 0.97; 0.4 [4]	2.5
31	Sixteenth floor, east end of south hall	(a) 1.35; 0.42	0.9
		(b) 1.58; 1.00 [4]; 0.48	1.4
31	Fifteenth floor, north wing	(a) 1.38; 0.45	1.7
		(b) 1.58; 0.42	2.4
June 1	Fourteenth floor, west end of south hall	(a) 1.26; 0.41; 0.19	0.3
		(b) 1.51; 0.98; 0.46; 0.19	0.5
1	Thirteenth floor, north wing	(a) 1.33; 0.44; 0.25 [3]	0.3
		(b) 1.51; 0.97; 0.47	0.5
1	Twelfth floor, east end of south hall	(a) 1.34; 0.43; 0.24	0.3
		(b) 1.54; 0.96; 0.48	0.3
1	Eleventh floor, north wing	(a) 1.35; 0.44; 0.25	0.4
		(b) 1.55; 0.96; 0.47; 0.28 [4]	0.4
1	Tenth floor, west end of south hall	(a) 1.39; 0.44; 0.25	0.3
		(b) 1.57; 0.98; 0.47	0.3
1	Ninth floor, north wing	(a) 1.34; 0.44; 0.25	0.1
		(b) 1.56; 0.47; 0.23	0.1
1	Eighth floor, west end of south hall	(a) 1.35; 0.44; 0.25	0.1
		(b) 1.57; 0.97; 0.48; 0.28	0.1
1	Seventh floor, center of south hall	(a) 1.35; 0.44; 0.24	0.1
		(b) 1.57; 0.97; 0.47; 0.27 [3]	0.1
1	Sixth floor, lobby south center	(a) 1.35; 0.43; 0.24	0.1
		(b) 1.56; 0.48; 0.27	0.1
2	Fifth floor, room 508 south side	(a) 1.31 [4]; 0.44; 0.24; 3.0 [4]	0.1−
		(b) 1.57; 0.49; 0.29; 0.19	0.1
2	Fourth floor, room 408 south center	(a) 1.33; 0.44 [4]; 0.25	0.1−
		(b) 1.54; 0.47	0.1
2	Third floor, room 314	(a) 1.3 [3]; 0.24 [4]	0.1−
		(b) 1.55; 0.48; 0.28	0.1
May 31	First floor, south hall near center	(a) 1.34	0.1−
		(b) 1.57	0.1
31	Basement, room B-4 south side	(a) 1.35; 2.8	0.1−
		(b) 1.50 [3]; 2.4 [3]	0.1−

See footnotes at end of table.

TABLE 6.—*Vibration data for buildings in which observations were made on several floors.*—Continued

Southern Pacific Building, 65 Market Street, San Francisco. [Components: (a) Spear Street, (b) Market Street]

Date 1934	Instrument location	Mean values of periods [2]	Building displacement [2]
		Seconds	*0.001 inch*
June 12	Fourteenth floor, tank room	(a) 1.15; 1.08; 0.4 [3]; 0.32; 0.27; 0.16	0.9
		(b) 1.20; 0.64; 0.21	0.8
11	Eleventh floor, west wing	(a) 1.14; 0.29; 0.20	0.9
		(b) 1.16; 0.3 [3]; 0.20	1.4
11	Eleventh floor, west corner	(a) 1.13; 0.30	1.5
		(b) 1.21; 0.33	0.7
11	Eleventh floor, center	(a) 1.14; 0.45 [3]; 0.21	1.0
		(b) 1.19; 0.30; 0.2 [3]	0.7
11	Eleventh floor, elevator	(a) 1.12; 0.46 [3]; 0.24	0.8
		(b) No record	----------
11	Eleventh floor, east corner	(a) 1.10; 0.93; 0.30	1.4
		(b) 1.24; 0.32; 0.19 [3]	0.6
11	Eleventh floor, east wing	(a) 1.12; 0.30	0.7
		(b) 1.19; 0.89; 0.30; 0.16 [3]	1.4
12	Tenth floor, west wing	(a) 1.11; 0.98; 0.30	1.0
		(b) 1.20; 0.94; 0.65; 0.21	1.2
12	Tenth floor, west corner	(a) 1.10; 0.57; 0.29	1.0
		(b) 1.20; 0.36 [3]; 0.27	1.2
12	Seventh floor, west wing	(a) 1.18; 1.00; 0.21 [3]	0.5
		(b) 1.23; 1.00; 0.27	1.1
12	Seventh floor, west corner	(a) 1.12; 0.98; 0.61	0.5
		(b) 1.22; 0.35	0.5
12	Seventh floor, center	(a) 1.13; 0.25	0.4
		(b) 1.22	0.7
12	Seventh floor, elevator	(a) 1.18; 0.21	0.4
		(b) 1.22; 0.21	0.8
12	Seventh floor, east corner	(a) 1.12; 0.23	0.7
		(b) 1.20; 0.2 [3]	0.6
12	Seventh floor, east wing	(a) 1.16; 1.00; 0.24	0.6
		(b) 1.12; 0.64; 0.24	0.9
13	Fourth floor, west wing	(a) 1.09; 0.27	0.4
		(b) 1.19; 0.97; 0.35; 0.23	0.5
13	Fourth floor, west corner	(a) 1.14; 1.07; 0.30	0.4
		(b) 1.19; 0.42; 0.21	0.4
13	Fourth floor, center	(a) 1.15; 0.31; 0.21	0.3
		(b) 1.21; 0.40; 0.28	0.3
13	Fourth floor, elevator	(a) 1.15; 0.29; 0.22	0.4
		(b) 1.25; 0.21	0.4
13	Fourth floor, east corner	(a) 1.00; 0.28	0.3
		(b) 1.25; 0.41; 0.22	0.5
13	Fourth floor, east wing	(a) 1.15; 0.99; 0.29	0.3
		(b) 1.08; 0.29; 0.22	0.5
13	Basement, west corner of seismograph room	(a) 1.10; 0.31; 0.20 [4]	0.1
		(b) 1.10; 0.47 [4]; 0.34 [4]; 0.27 [4]	0.1—

Oakland City Hall, Fourteenth and Washington Streets, Oakland. [Components: (a) Fourteenth Street (b) Washington Street]

Date 1934	Instrument location	Mean values of periods [2]	Building displacement [2]
Oct. 10	Sixteenth floor, center	(a) 1.20; 0.3 [3]	0.7
		(b) 1.10; 0.33 [3]	0.5
10	Fourteenth floor, center	(a) 1.20; 0.31 [3]	0.7
		(b) 1.10; 0.33 [3]	0.5
10	Twelfth floor, north of center	(a) 1.20; 0.4 [4]	0.3
		(b) 1.10; 0.45	0.3
10	Tenth floor, east of center	(a) 1.20	0.5
		(b) 1.10; 0.45	0.5
10	Eighth floor, south of center	(a) 1.20; 0.4 [3]	0.3
		(b) 1.10; 0.4 [4]	0.1
10	Sixth floor, northeast corner	(a) 1.20 [3]; 0.33 [3]	0.1
		(b) 1.10; 0.4 [3]; 0.33	0.3
10	Mezzanine, southwest corner	(a) 1.20; 0.33	0.1
		(b) 1.10; 0.33	0.1
10	Third floor, northeast corner	(a) 1.20; 0.33	0.1
		(b) 0.33	0.1
10	Basement, seismograph room	(a) 0.7 [3]	0.1

See footnotes at end of table.

TABLE 6.—*Vibration data for buildings in which observations were made on several floors.*—Continued

C and H Sugar Co., Crockett. [Components: (a) N–S, (b) E–W.]

Date 1934	Instrument location	Mean values of periods [1]	Building displacement [2]
	CHAR HOUSE [6]	*Seconds*	*0.001 inch*
Aug. 3	Tank platform, center	(a) 0.97; 0.20	0.2
		(b) 0.91; 0.28	0.2
20	Tank platform, near east side	(a) 0.94; 0.27	0.2
		(b) 0.90; 0.27	0.5
3	Roof of ninth floor, below tanks	(a) 0.94; 0.20	0.2
		(b) 0.94; 0.28	0.4
6	Roof of ninth floor, below tanks	(a) 0.93; 0.40 [4]; 0.25 [4]; 0.15 [4]	0.3
		(b) 0.90; 0.27	0.2
7	Roof of ninth floor, below tanks	(a) 0.30; 0.13	0.9
		(b) 1.00; 0.30; 0.1 [4]	0.7
3	Roof of ninth floor, west wing	(a) 0.94; 0.20	0.2
		(b) 0.92; 0.20	0.4
6	Ninth floor, west wing	(a) 0.96; 0.28	0.2
		(b) 0.90; 0.25	0.2
7	Ninth floor, west wing	(a) 0.30; 0.05; 2.5 to 3.0 [4]	2.5
		(b) 1.00; 0.27; 0.1 [4]; 0.06; 2.6	0.8
3	Roof of seventh floor, northwest section	(a) 0.94; 0.20	0.2
		(b) 0.92; 0.28	0.2
7	Roof of seventh floor, northwest section	(a) 1.5 to 2.0 [4]; 0.05 [4]	2.6
		(b) 0.30; 0.05 [4]	0.5
3	Roof of seventh floor, southwest section	(a) 0.97; 0.20	0.1
6	Roof of seventh floor, southwest section	(b) 0.94; 0.28	0.2
7	Roof of seventh floor, southwest section	(a) 1.00; 0.30; 0.05	0.2
		(b) 1.00; 0.30; 0.10; 0.05 [4]	0.4
6	Seventh floor, beneath tanks	(a) 0.94; 0.20	0.2
		(b) 0.90; 0.28	0.2
7	Seventh floor, beneath tanks	(a) 1.00; 0.30; 0.10; 0.05 [4]	0.5
		(b) 1.00; 0.28; 0.1 [3]; 0.05 [3]	0.6
6	Fifth floor, beneath tanks	(a) 0.93; 0.25 [3]	0.1
		(b) 0.94; 0.27	0.1
7	Fifth floor, beneath tanks	(a) 0.27; 0.10	0.4
		(b) 1.00; 0.53; 0.2 or 0.3 [4]; 0.10	0.4
	REFINERY		
6	Seventh floor, center	(a) 0.50; 0.26	0.1
		(b) 0.50; 0.26	0.1
7	Seventh floor, center	(a) 0.53; 0.15; 0.05	0.5
		(b) 0.50; 0.40; 0.35; 0.06	0.4
	PACKING HOUSE NO. 1		
6	Seventh floor, west of center	(a) 0.50; 0.05 [4]	0.1
		(b) 0.52; 0.25 [4]	0.1
7	Seventh floor, west of center	(a) 2.7; 0.54; 0.10	1.5
		(b) 2.7; 0.40; 0.05	1.1
	PACKING HOUSE NO. 2		
6	Fourth floor, between char house and refinery	(a) 2.5; 0.1 [3]	0.2
		(b) 0.28 [3]; 0.1 [3]	0.1
20	Fourth floor, west end	(a) 0.95; 0.28	0.2
		(b) 0.95; 0.28	0.2

Ferry Building, foot of Market Street, San Francisco. [Components: (a) Embarcadero Street, (b) Market Street]

Date 1934	Instrument location	Mean values of periods	Building displacement
Oct. 22	Twelfth floor, tower	(a) 0.69; 0.15	0.8
		(b) 0.82; 0.10	2.3
22	Ninth floor, tower	(a) 0.69; 0.3 [4]	0.8
		(b) 0.81; 0.3 [4]	1.2
22	Second floor, south end	(a) 1.14	0.2
		(b) 1.0; 0.3 [3]; 0.1 [3]	0.2
22	Second floor, center	(a) 1.1 [3]; 0.1 [3]	0.2
		(b) 1.0 [3]; 0.22	0.2
22	Second floor, north end	(a) 1.1 [3]; 0.3 [3]	0.1
		(b) 1.0 [3]; 0.25 [3]	0.2

See footnotes at end of table.

TABLE 6.—*Vibration data for buildings in which observations were made on several floors*—Continued

Claus Spreckles Building, 3d and Market Streets, San Francisco. [Components: (a) 3d Street, (b) Market Street]

Date 1934	Instrument location	Mean values of periods [2]	Building displacement [3]
		Seconds	*0.001 inch*
Oct. 18	Eighteenth floor	(a) 2.20; 0.40 (b) 1.95, 0.60; 0.36	1.1 1.5
18	Fifteenth floor	(a) 2.20; 0.40 (b) 1.93; 0.28	0.4 0.8
18	Fourteenth floor	(a) 2.20; 0.40; 0.20 (b) 1.94; 0.27; 0.20	1.1 1.5
18	Thirteenth floor	(a) 2.20; 0.41; 0.1 [4] (b) 1.94; 0.35; 0.2 [4]	1.1 1.1
18	Twelfth floor	(a) 2.10; 0.40; 0.20 (b) 1.90; 0.6 [4]; 0.30 0.2 [3]	0.8 0.8
18	Eleventh floor	(a) 2.15; 0.63; 0.19 (b) 1.90; 0.60; 0.35	0.8 0.8
18	Tenth floor	(a) 2.10; 0.66 (b) 2.00; 0.60; 0.29	0.8 0.4
18	Fifth floor	(a) 2.0 [3]; 0.66; 0.35 [3] (b) 1.9 [4]; 0.60; 0.31	0.4 0.4

Alexander Building, Bush and Montgomery Streets, San Francisco. [Components: (a) Montgomery Street, (b) Bush Street]

Date 1934	Instrument location	Mean values of periods [2]	Building displacement [3]
Oct. 17	Sixteenth floor	(a) 1.25; 0.40 (b) 1.33; 0.84	1.2 1.0
17	Fourteenth floor	(a) 1.25; 0.2 [4] (b) 1.35; 0.8 [4]; 0.2 [4]	0.7 0.7
17	Twelfth floor	(a) 1.25; 0.22 (b) 1.33; 0.7 [4]; 0.2 [4]	1.2 0.7
17	Eleventh floor	(a) 1.25; 0.4 [4]; 0.22 (b) 1.33; 0.7 [4]; 0.22	0.5 0.7
17	Tenth floor	(a) 1.24; 0.40; 0.2 [4] (b) 1.33; 0.40	0.5 0.7
17	Fifth floor	(a) 1.25; 0.40; 0.22 (b) 1.32; 0.40; 0.22	0.2 0.2

Bank of America Building, 1st Street and Santa Clara Avenue, San Jose. [Components: (a) 1st Street, (b) Santa Clara Avenue]

Date 1934	Instrument location	Mean values of periods [2]	Building displacement [3]
Sept. 26	Fifteenth floor, tower	(a) 1.20; 0.4 [3] (b) 1.31; 0.3 [3]	0.5 0.7
26	Thirteenth floor, seismograph room	(a) 1.21; 0.75 [3] (b) 1.30; 0.3 [4]	0.5 0.7
26	Eleventh floor, north end of hall	(a) 1.20; 0.32 [3] (b) 1.29; 0.80	0.2 0.2
26	Ninth floor, south end of hall	(a) 1.20 (b) 1.32; 0.3 [3]	0.2 0.5
26	Seventh floor, north end of hall	(a) 1.16; 0.42 (b) 1.31; 0.81; 0.23 [3]	0.5 0.2
26	Fifth floor, south end of hall	(a) 1.16; 0.35 [3]; 0.15 [3] (b) 1.31; 0.3 [3]; 0.12 [3]	0.2 0.2
27	Second floor, corner	(a) 1.2 [3]; 0.43 (b) 1.30 [3]; 0.37 [3]; 0.26	0.1 0.1
27	Second floor, northeast wing	(a) 0.25 [3]; 0.12 (b) 1.31; 0.35 [3]; 0.24	0.1 0.1
27	Basement, seismograph room	(a) 0.1 [3] (b) 0.2 [3]; 0.3 [3]	0.1— 0.1—

Sather Tower (Campanile),[7] University of California Campus, Berkeley. [Components: (a) N-S, (b) E-W]

Date 1934	Instrument location	Mean values of periods [2]	Building displacement [3]
July 19	Between seventh and eighth floors	(a) 1.19 (b) 1.19	0.3 0.6
19	Sixth floor	(a) 1.17; 0.26 (b) 1.18	0.3 0.3
19	Fourth floor	(a) 1.18; 0.27 [8] (b) 1.20; 0.26 [8]	0.3 0.3

See footnotes at end of table.

TABLE 6.—*Vibration data for buildings in which observations were made on several floors*—Continued

San Francisco City Hall, Polk and McAllister Streets, San Francisco. [Components: (a) Polk Street, (b) McAllister Street]

Date	Instrument location	Mean values of periods	Building displacement
		Seconds	*0.001 inch*
Oct. 29	Fourth floor, front lobby	(a) 0.71; 0.40; 0.11	0.1
		(b) 0.65; 0.27 [3]; 0.1 [3]	0.1
26	Fourth floor, south end	(a) 0.67 [3]; 0.25; 0.10 [3]	0.1
		(b) 0.7 [4]; 0.40	0.1
26	Fourth floor, north end	(a) 0.67 [3]; 0.40 [3]; 0.27	0.1
		(b) 0.69; 0.39	0.1
26	Fourth floor, at dome arch footings	(a) 0.70; 0.4 [3]; 0.10 [3]	0.1
		(b) 0.68; 0.27 [4]	0.1
26	Fourth floor, between dome arch footings	(a) 0.69; 0.17	0.1
		(b) 0.67; 0.35 [3]	0.1
July 27	Top of dome	(a) 0.71; 0.22	0.3'
		(b) 0.60; 0.32; 0.17	0.3
27	Seventh floor, gallery	(a) 0.65; 0.17; 0.11	0.1
		(b) 0.65; 0.18; 0.12	0.1

Los Angeles City Hall, 1st and Main Streets, Los Angeles. [Component: E-W]

Date	Instrument location	Mean values of periods	Building displacement
1935			
Apr. 3	{Twenty-fourth floor [9]	2.25; 0.8 [4]; 0.3 [4]	1.3
	{Twentieth floor	2.26; 0.50	1.1
3	{Twenty-fourth floor	2.25; 0.28 [3]	1.5
	{Sixteenth floor	2.25; 0.50; 0.25 [3]	0.9
3	{Twenty-fourth floor	2.25 [10]; 0.75	1.3
	{Twelfth floor	2.27 [10]; 0.82; 0.6 [3]; 0.41	0.6
3	{Twenty-fourth floor	2.23; 0.85	1.4
	{Ninth floor	2.25 [3]; 0.82	0.4

Pellissier Building, 3780 Wilshire Boulevard, Los Angeles. [Components: (a) NE-SW, (b) NW-SE]

Date	Instrument location	Mean values of periods	Building displacement
Feb. 13	{Twelfth floor [9]	(a) 0.91	0.5
		(b) 0.82	0.7
	{Eighth floor	(a) 0.91; 0.50	0.4
		(b) 0.82	0.5
13	{Twelfth floor	(a) 0.91	0.4
		(b) 0.82	0.5
	{Sixth floor	(a) 0.91; 0.50; 0.1	0.2
		(b) 0.82; 0.1	0.3

[1] Vibration periods are measured in directions parallel to the streets indicated by (a) and (b) in the heading of each tabular group.
[2] Measured from zero position.
[3] Approximate value.
[4] Questionable value
[5] Irregular motion.
[6] Height of Char House: 9 stories; 140 feet to roof, 164 feet to tank platform. During the observations on Aug. 3 there was one machine running on the ninth floor of Char House. The plant was in operation Aug. 7; not in operation Aug. 6.
[7] Each story corresponds in height to 2 ordinary stories.
[8] Periods of 0.27 and 0.26 sec. are very prominent on the fourth floor.
[9] Braces indicate simultaneous observations.
[10] There were a few waves of 2.5 sec. period at time of the third test; they appeared on both the twenty-fourth and twelfth floor records.

The data of table 6 indicate that the amplitude of vibration at a given level of a building is roughly proportional to the elevation of that level, even though records on different floors were usually made under varying wind conditions. Byerly [2] noted this in his measurements of building 127. This condition is further verified by measurements taken simultaneously at levels which are widely separated. See chapters 6 and 7.

[2] Byerly, Perry, Vibrations of Buildings. An abstract, Pacific Coast Applied Mechanics Meeting of the American Society of Mechanical Engineers, California Institute of Technology, Pasadena, Jan. 20–21, 1933.

Mr. Neil R. Sparks has plotted the periods of a number of buildings against their heights in figure 38. Although the points are scattered, the graph indicates roughly that the period is a linear function of the height. It must be understood, however, that the period of a build-

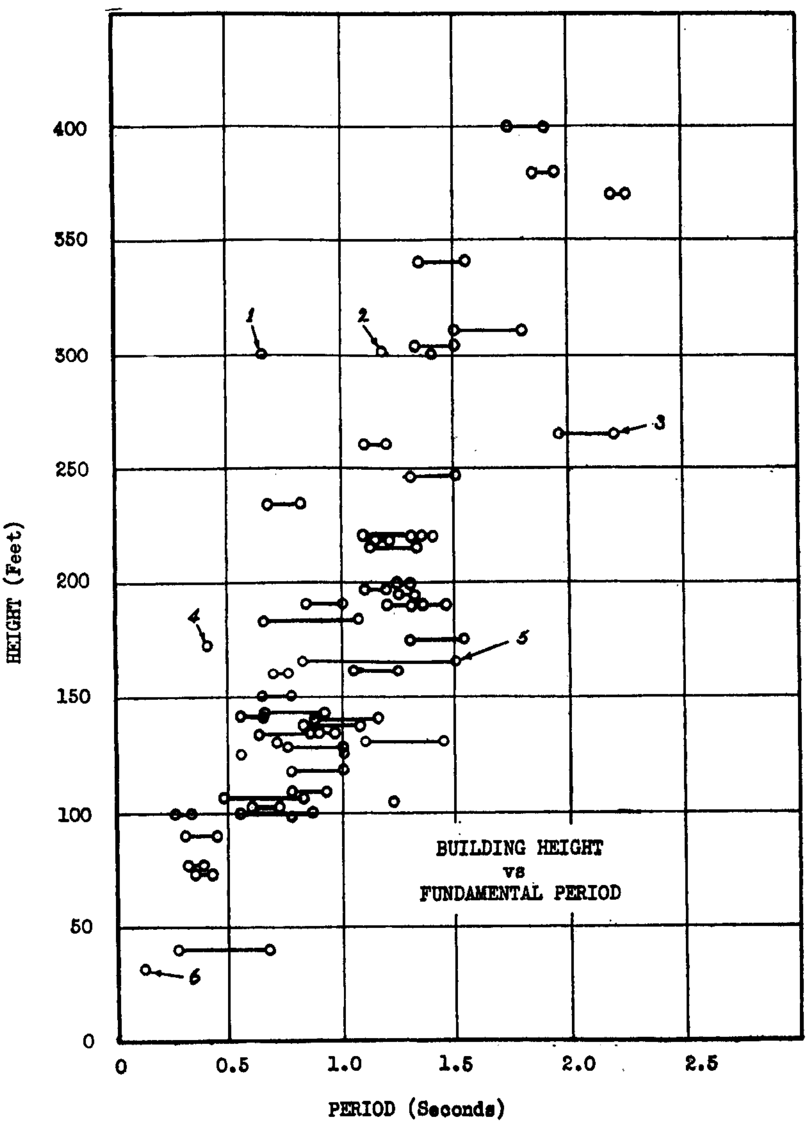


FIGURE 38.—Building height v. fundamental period.

ing is determined also by other factors such as elastic properties and mass.

The periods of vibration of the Shell Building (fig. 40) have been plotted against its height in figure 53. This graph shows clearly the nodal points of the second and third modes of vibration, but it does not show the actual deflection curve, as the observations were not taken simultaneously. The fundamental vibrations of high buildings have been observed on the ground floor during a high wind, and they are noticeable under moderate wind conditions on other floors although the secondary vibrations are predominant in that region.

Byerly has noted that vibrations of building 127 are transmitted to building 199, even though the two have no further connection than the lath and plaster of common hallways and 2 inches of terrazzo on the floors, and the foundations are separated by a ½-inch thickness of gypsum plaster board. Sparks, in his study of vibration records, has verified Byerly's conclusions in regard to buildings 127 and 199 and has also noted similar conditions existing elsewhere. A noteworthy example is the case of the Shell, 130 Bush Street, and Adam Grant Buildings of San Francisco, which are located in the same block, with the narrow 130 Bush Street Building sandwiched between the other two as shown in figure 40. The long 1.9-second period vibrations of the Shell Building are noted along both components of both the other buildings, and vibrations in the 130 Bush Street building are probably influenced by both of the other two. Each of these buildings is structurally independent. It is planned to investigate the vibrations of these buildings simultaneously.[3]

The tower of the Bank of America Building in Oakland is an addition to the older structure. The two buildings have interconnecting hallways, walls, and floors, but are otherwise structurally independent. A similar example is the Tribune Tower and the older Tribune Building in Oakland.

The Pacific Gas and Electric and the Matson Buildings in San Francisco form another example of abutting but structurally independent buildings. Byerly, Hester, and Marshall[4] have noted that these buildings have the same period in the direction parallel to Market Street. Their heights, however, are nearly the same.

The periods of five buildings have been measured to date at various stages of construction or repair. Vibrational data on these buildings are listed in table 7.

It is to be noted that increased stiffening from the addition of the curtain wall of the Federal office building seems to compensate the increase in load by the addition of the fifth floor. Byerly observed that the addition of the exterior walls to the framework of Building 128 reduced the period in one direction (north-south) by 0.1 second, but did not affect the east-west vibration. Other buildings under construction are the Jane Addams and the Louis Agassiz Buildings at the Pasadena Junior College. The measurements of these buildings are not yet complete.

[3] The foregoing conclusions were later verified by simultaneous observations in the three buildings.

[4] Perry Byerly, James Hester, and Kenneth Marshall, The Natural Periods of Vibration of Some Tall Buildings in San Francisco, Bulletin of the Seismological Society of America, vol. 21, no. 4, December, 1931.

FIGURE 39.—Telephone Building, San Francisco.

FIGURE 40.—Shell Building, San Francisco.

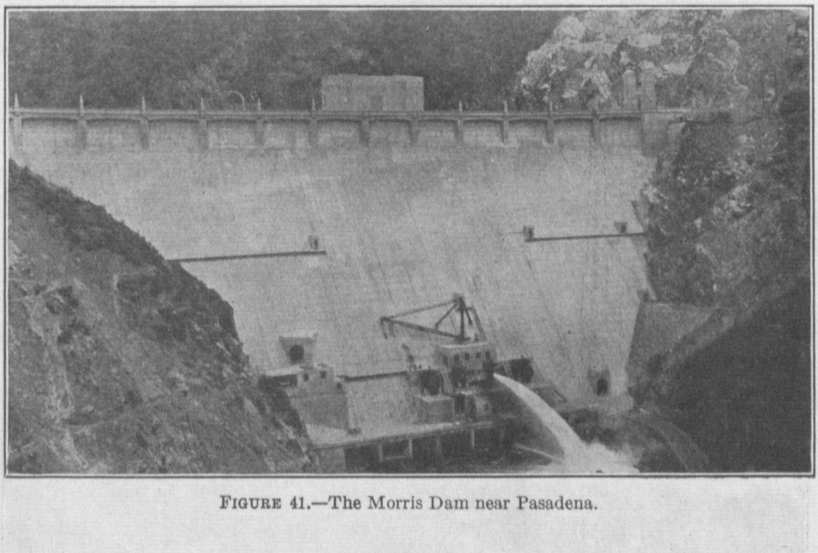


FIGURE 41.—The Morris Dam near Pasadena.

FIGURE 42.—Hollywood Storage Building.

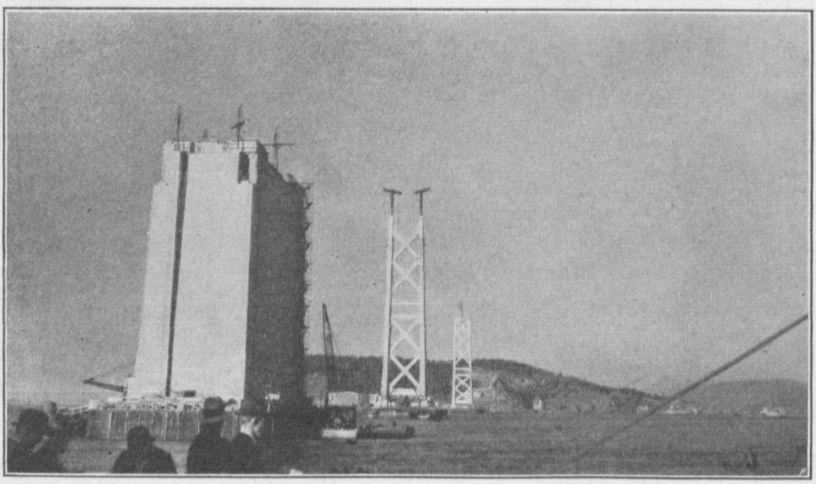


FIGURE 43.—Pier W-4 (and W-5 and W-6 with towers), Oakland Bay Bridge, February 15, 1935.

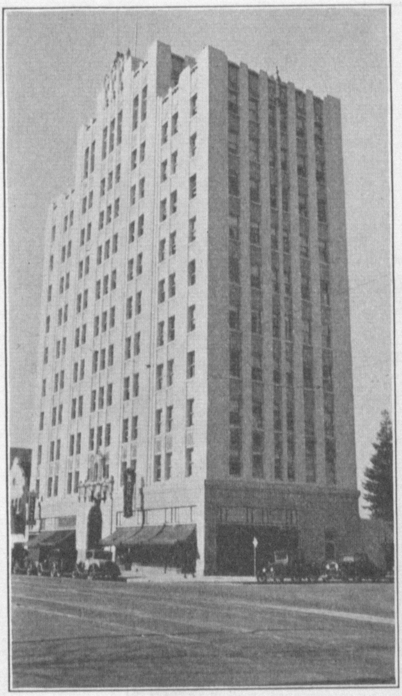


FIGURE 44.—Medico-Dental Building, San Jose.

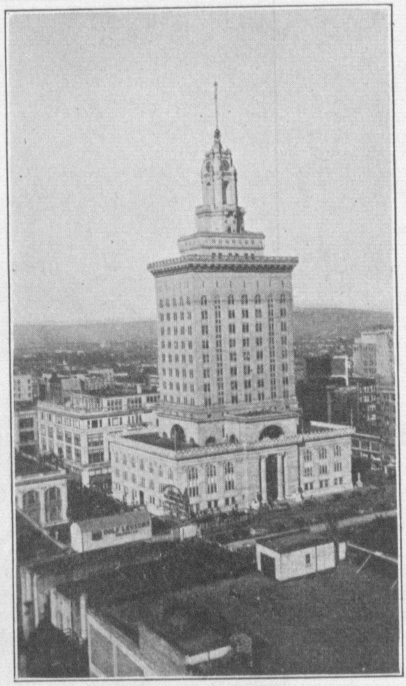


FIGURE 45.—Oakland City Hall.

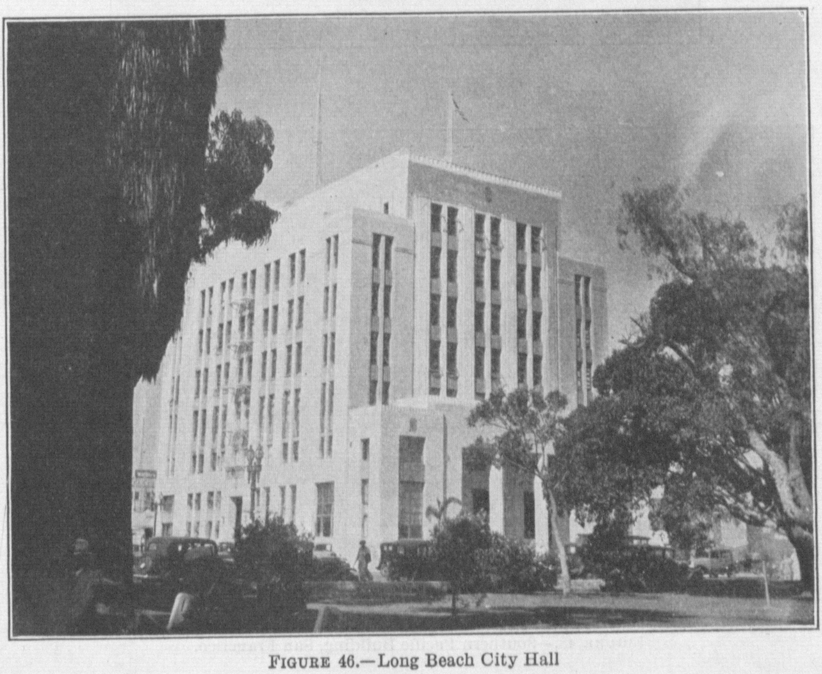


FIGURE 46.—Long Beach City Hall

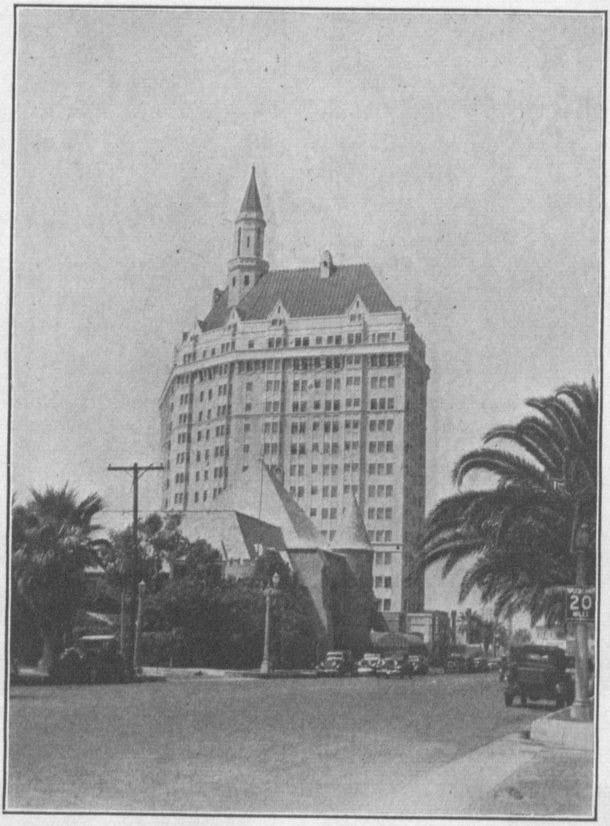


FIGURE 47.—Villa Riviera, Long Beach.

FIGURE 48.—Southern Pacific Building, San Francisco.

FIGURE 49.—Los Angeles Chamber of Commerce.

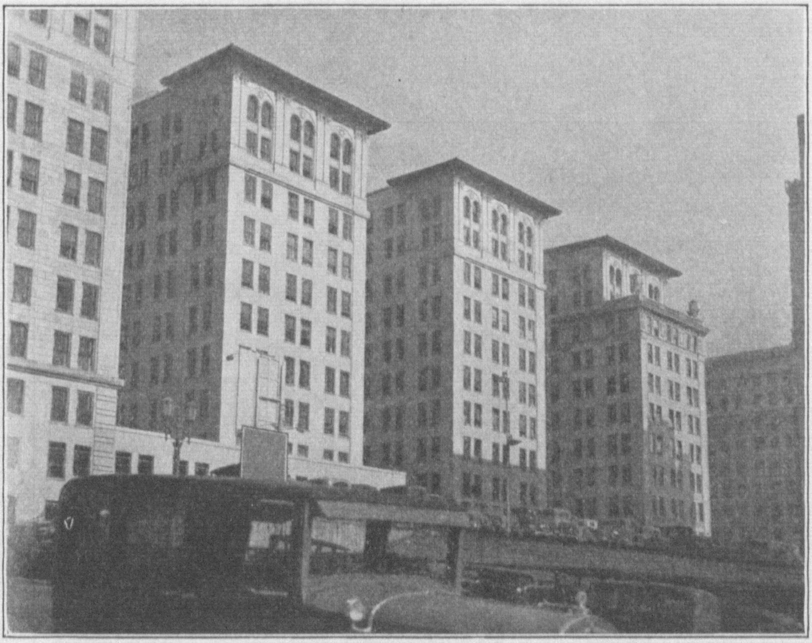


FIGURE 50.—Subway Terminal Building, Los Angeles.

FIGURE 51.—Los Angeles City Hall.

FIGURE 52.—Hartwell Building, Long Beach.

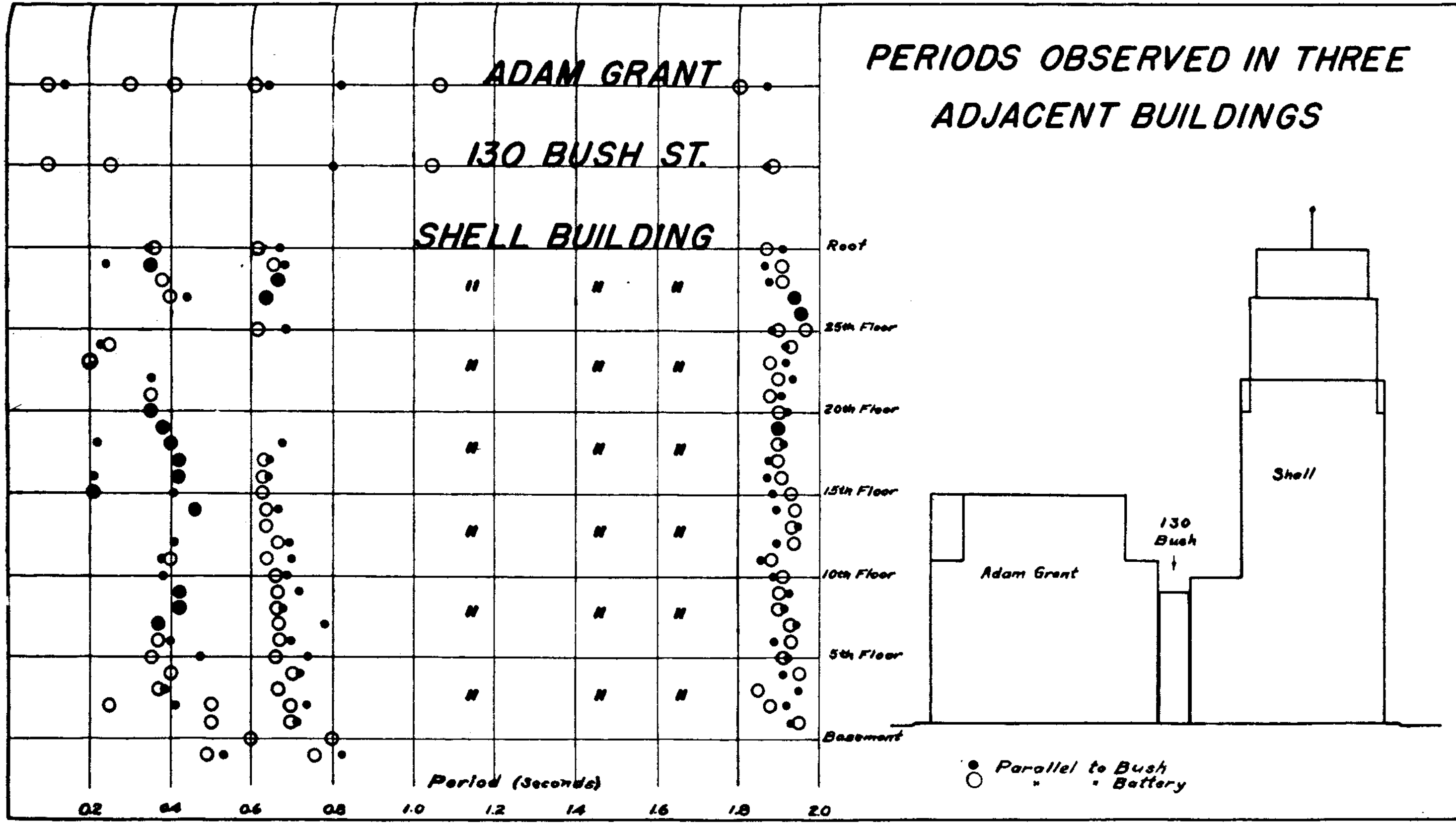


FIGURE 53.—Periods observed in three adjacent buildings.

TABLE 7.—*Vibration data for buildings undergoing construction or alteration at time of observation*

Federal Office Building, McAllister and Leavenworth Streets, San Francisco. [Components: (a) Leavenworth Street, (b) McAllister Street. July 11 test was made prior to pouring of concrete for fifth floor. Walls and partitions not in. Aug. 27 test made with attic floor and roof on east, south, and west sides complete. Fifth floor poured. North side of fifth story open. Walls and partitions not in. Nov. 20 test made with brick inner curtain walls up to above fifth floor. Outer curtain walls up to between first and second floors. Partitions and ceilings not in]

Date, 1934	Instrument location	Mean values of periods [1]	Building displacement [2]
		Seconds	*0.001 inch*
July 11	Fourth floor, east end	(a) 0.54; 0.40; 0.33 [3]; 0.1 [3]	0.4
		(b) 0.56; 0.26	0.2
Do.	Fourth floor, north side	(a) 0.54; 0.47; 0.31	0.1
		(b) 0.43; 0.28	0.1
Aug. 27	Fifth floor, east end	(a) 0.73; 0.2 [3]	0.9
		(b) 0.6; 0.2 [3]	0.5
Do.	Fifth floor, north side	(a) 0.65; 0.30 [3]	0.4
		(b) 0.57; 0.35 [3]	0.5
Nov. 20	Fifth floor, east end	(a) 0.49; 0.10	0.2
		(b) 0.38	0.1
Do.	Fifth floor, north side	(a) 0.52; 0.26 [5]; 0.10 [3]	0.1
		(b) 0.37; 0.26	0.1

Lagunita Court, Campus, Stanford University. [Components: (a) E-W, (b) N-S. June 15 test was made when metal lathing was partially on; planking on roof but no tile. Diagonal braces in walls and partitions are welded to steel frame. Sept. 25 test made when building was ready for occupancy. There are four separate sections, each 2 stories and attic, connected by flexible passages]

June 15	Attic of second floor in north end of southeast wing	(a) 0.22; 0.08 [4]; 0.05	0.1
		(b) 0.22; 0.08; 0.05 [3]	0.1—
Sept. 25	do	(a) 0.12; 0.031 [5]	0.1
		(b) 0.14; 0.06; 0.026	0.1—
Do.	Attic of second floor in north end of southwest wing	(a) 0.13; 0.08	0.1—
		(b) 0.14; 0.026	0.1—

Horace Mann Building, Pasadena Junior College, 1400 East Colorado Street, Pasadena. [Components: (a) N-S, (b) E-W. The walls were of brick and the partitions of hollow tile at the time of the first test. Removal of the walls had been started; much of the north parapet wall and all of the material around the dome had been taken down. In the second test all walls and partitions were down]

Aug. 24	Third floor near northwest inside corner	(a) 0.22	0.1—
		(b) 0.23	0.1—
1935 Mar. 1	Third floor near northwest inside corner	(a) 0.46; 0.35	0.1—
		(b) 0.46, 0.35	0.1—

Willmore Apartments, 315 West Third Street, Long Beach. [Components: (a) N-S, (b) E-W. Tests were made before and after building had been repaired and braced for earthquake resistance]

1934 Sept. 6	Eleventh floor, kitchen off solarium at northeast corner.	(a) 0.73; 0.2 [3]	0.1
		(b) 0.74; 0.2 [3]	0.3
Dec. 28	Eleventh floor, by stairway	(a) 0.62	0.2
		(b) 0.62	0.2

City Transfer Building, 1430 East Anaheim Street, Long Beach. [Components: (a) E-W, (b) N-S. Damaged by earthquake of March 10, 1933. Repair work was started about August 27 and completed by time of repeat test on November 15]

Aug. 27	Seventh floor, southeast corner	(a) 0.33; 0.2	0.1
		(b) 0.38; 0.2 [4]	0.1
Nov. 15	Seventh floor, southeast corner	(a) 0.340; 0.2 [4]	0.3
		(b) 0.315; 0.15 [4]	0.1

[1] Vibration periods are measured in directions parallel to the streets indicated by (a) and (b) in the heading of each tabular group.
[2] Measured from zero position.
[3] Approximate value.
[4] Questionable value.
[5] Very prominent period.

TANK TOWER VIBRATIONS

The primary purpose of the investigation of steel water towers was to determine, if possible, the influence of soil, foundation, degree of bracing, tension of tie rods, loading, wind velocity, direction and magnitude of side loading, and other factors, upon the character of vibration of these structures. An attempt was also made to compare the vibratory characteristics of a structure of this type in several directions and at all places on a tower where an instrumental set-up could be made.

The vibrations of 37 water towers have been investigated. Two of these have wooden tanks on steel frames. Two are on the tops of buildings, one is on the ground with the building built around it, and the remaining towers are independent structures. The capacities of the tanks usually range from 50,000 to 100,000 gallons, and the heights to balconies are usually from 63 to 133 feet. The few exceptions are noted in table 12. The recording instruments are usually placed on the platform of the tank between two consecutive columns, or if this is inconvenient, on the walk below the tank. It has been demonstrated that the fundamental period of the tank can be measured at both positions.

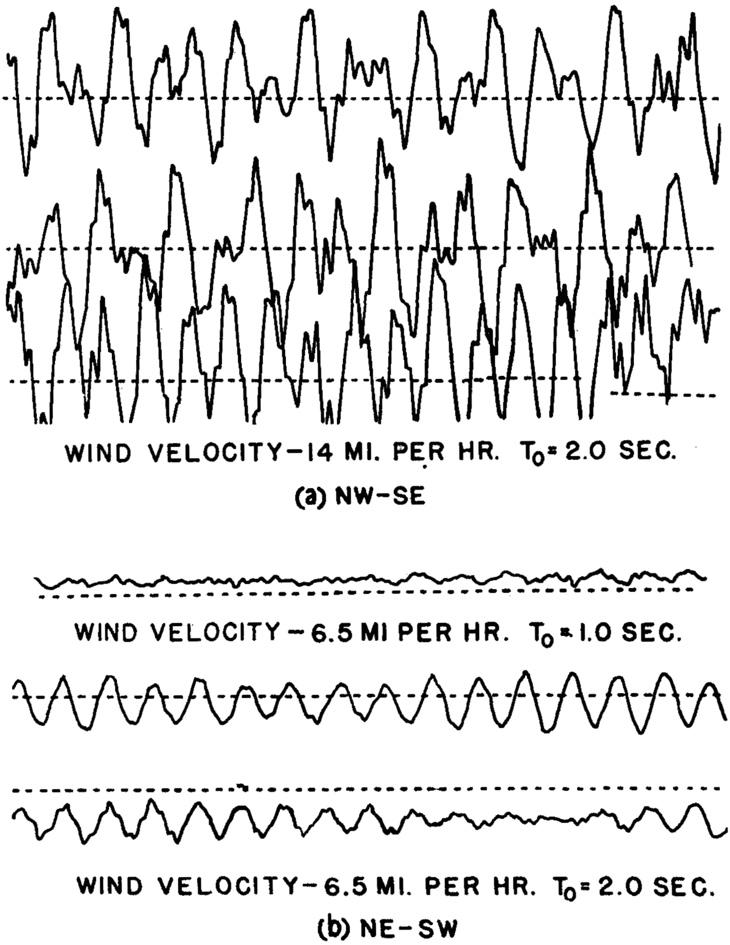
During the course of this investigation several tank towers have undergone strengthening for more effective earthquake resistance. The vibrations of some of these towers have been measured while the tanks were empty and full, and before and after reinforcing. On some, tests have been made from time to time as the tanks were being filled.

Figure 54 shows portions of records of vibrations of a tank tower taken on the same day but under different wind and instrumental conditions. The larger waves represent the fundamental translatory vibrations of the tower.

It is interesting that the maximum amplitude is roughly proportional to the square of the wind velocity. Note further that reduction of T_0 below the tank period materially reduces the trace amplitudes of the fundamental waves, but has little influence on the secondary vibrations provided the reduction of T_0 is not too great. This is the performance expected in view of the harmonic magnification curve of the instrument.

Figure 55 shows a record written by three types of vibration. The regular undulations (T_t=1.33 seconds) undoubtedly represent the fundamental vibrations of the tower. The long underlying waves have a period of about 2.6 seconds. Waves of this type appear on many, though not all, tank-tower vibrograms, especially if the wind at the time the records were being made was fairly strong. Mr. Arthur C. Ruge has observed a natural period of 2.65 seconds in a model of the Willard storage battery tank tower tested on a shaking table at the Massachusetts Institute of Technology, and ascribes it to the natural swing of water inside the tank.

The period of the short secondary waves shown in figure 55 is about 0.35 second. Waves of this type appear on nearly every tank tower vibration record. Most of these records were made with a single component vibration meter placed first on one side of the tank, and then at 90° around the tank to get the other component. The direction of registration was always tangent to the tank. Later, when it was possible to record two components simultaneously, it was observed that the secondary waves written by the instrument vibrating in the direction along the radius of the tank have much shorter periods (about 0.1 second, see fig. 57b). As the 0.35-second waves appear on very few, if any, records of the radial component, and as they have been observed to remain nearly constant during a



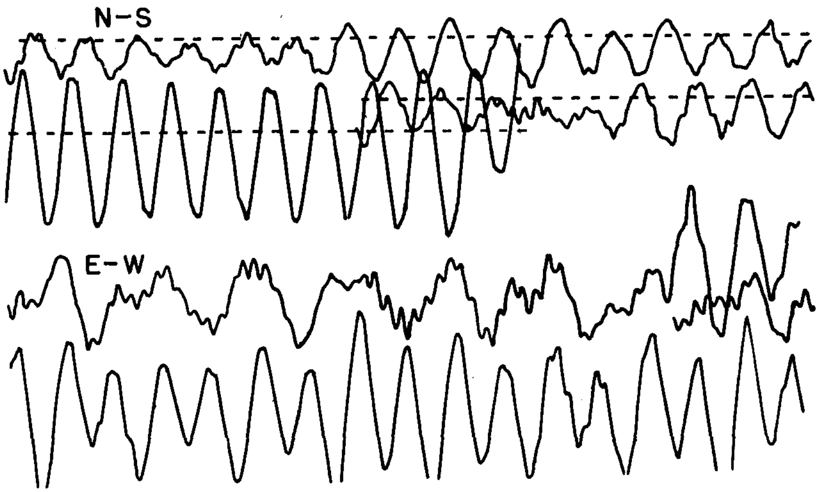
VIBROGRAMS OF THE CALIFORNIA CONSERVING COMPANY TANK TOWER, HAYWARD, CALIFORNIA

FIGURE 54.

change of load within the tank, it seems evident that they are produced by torsional vibrations of the tower.

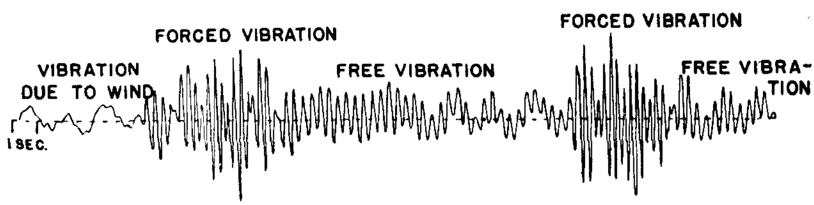
Figure 56 is a vibrogram taken during and after forced vibrations of a set of tierods in the upper panel. While the rods were being forced to vibrate, their period of 0.26 second was predominant in the record. After the force was no longer applied, it was the 0.34-second period of torsional vibration that was recorded.

Figure 57 is a record of a pull-back test (to be described later) on the same tank. The force was applied to one column and was directed tangent to the tank so that its release would produce strong torsional vibrations. The waves which are so dominant on the tangential component, but which are very faint on the radial component, have a period of 0.35 second, which is also the period of secondary waves



VIBROGRAM OF THE STANDARD SANITARY PRODUCTS CO. WATER TOWER

FIGURE 55.



SEPT. 17, 1934. SET-UP NO. 4
N–S COMPONENT
DIAGONAL TIE-RODS FORCED TO VIBRATE BY SHAKING ONE ROD FOR ABOUT 5 SEC.

VIBROGRAM OF THE PORT OF OAKLAND TANK TOWER

FIGURE 56.

on wind records (tangential component) of this tank. They clearly indicate rotary oscillations of the tank.

Secondary waves appearing on vibrograms of the radial component have shorter periods, usually from 0.10 to 0.15 second. They are probably produced by tilt resulting from vertical vibrations of the platform.

Figures 54 and 55 are typical tank tower vibration records where wind is the motivating force. The study of these records gives rise to several interesting problems, for instance, when the amplitudes of the vibrations become so large that a set of tierods go out of action as the tank approaches its maximum deflection, how is the character of vibration of the structure affected? Also, what part does the water in the tank play in these vibrations when all rods remain in action; and when some of them are slackened to zero tension? Mr. Ruge has suggested giving a tank tower a series of pull-back tests as a means of solving these problems. A pull-back test is usually conducted by attaching a rope to a member near the tank and connecting the other end to a block-and-tackle arrangement which is anchored to a rail, tree, or post. The release is effected by cutting or breaking a wire link. Usually, a no. 6 Calloy wire link is broken, the wire being previously tested to break at 2,000 pounds. Forces in earlier tests were measured by inserting in the line a dynamometer borrowed from Stanford University.

An attempt was made to so stress some of the towers that a set of tierods would go out of action. The two highest applied forces (3,200 and 3,700 pounds) failed to do this. Some of the tierods of the third tower so tested were very loose. If they were not out of

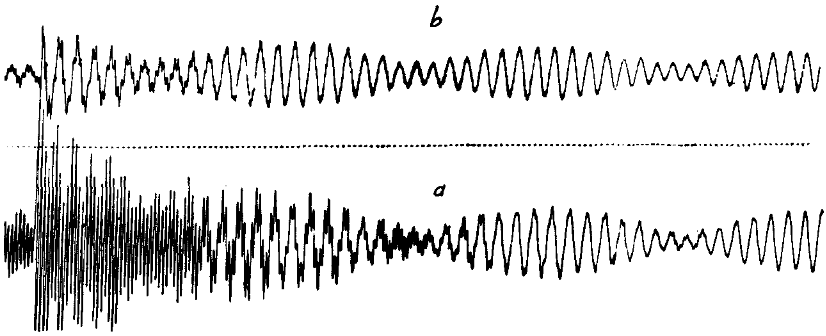


FIGURE 57.—Pull-back test on the Port of Oakland tank tower.

action at zero side load, they certainly went out before the applied force reached 2,000 pounds. We have not, therefore, arrived at an entirely satisfactory solution of our first problem but have, nevertheless, secured valuable information as to the role played by tierod tension in the natural vibration of a water tower. Furthermore, these tests have given rise to other problems which are not without interest.

Figure 62 shows typical records of pull-back tests. The sudden departure from the rest position represents the release of the applied force. A detailed analysis of vibrograms of pull tests reveals that a tank particle vibrates in an elongated ellipse. Upon the release of the pull, the major axis of the ellipse is in the direction of the pull. It then usually rotates to the transverse direction and continues on through the longitudinal again. It has been observed that the rate of rotation of the direction of greatest vibration is relatively slow on towers on which the tierods have recently been tightened. It appears, then, that inequality in the tension of tierods causes this rotation. The path of a particle through one cycle at different stages of vibration of some of the towers tested is shown in figure 63.

The extraneous vibrations on records of the radial component were caused by slight movements of the observer on the platform.

The records of the Port of Oakland tower, figure 62a, tank full, are of special interest. Twelve records of pull-back tests have been made of this tower, all of which show this characteristic beating effect, even though the exciting force has been applied in such a way as to set the tank into torsional vibration (see figs. 57 and 66). The period of this beating (recorded in fig. 62a) is invariably 22.5 to 23 seconds, and each beat contains 14 to 15 waves.

Such beating might be explained in various ways. The rotation previously mentioned of the direction of vibration at the rate of one cycle in 45 or 46 seconds would give results such as are observed here, except that the peaks on one component would occur simultaneously with the nodes on the other. But in the present observations the peaks occur almost together, so that this explanation is insufficient. It seems evident, then, that the cause is a union of two vibrations which are not far apart in frequency. The extremes in the periods of individual waves on all records of the Port of Oakland tower, tank full, have been observed to be 1.53 and 1.64 seconds. The union of two vibrations having these respective periods produces beats having a period of very nearly 23 seconds. Such action may be a combination of the tower vibrating alone and the tower and foundation vibrating together, especially if the structure is on piling driven through yielding material. The S. H. Frank & Co. tannery and the Paraffine Co. tank towers are two other structures of this type. As the characteristic beating did not appear on the pull-test records of these tank towers, it appears that we must seek further to find the cause of the beating on the Port of Oakland tank pull-test records.

Prof. L. S. Jacobsen offers the following explanation:

It is probable that the gravitational waves of the water in the tank, surging from one side to the other, reach maximum values with a slightly different period from that of the natural vibration of the tank tower. In other words, the body of water constitutes a separate vibrating system. If this be the explanation of the beating phenomenon, a change in the amount of water in the tank would most certainly change its natural surge period. It is true that the period of the tank tower would also be changed, but it is quite improbable that the period of beating would remain constant.

Figure 62b is a pull-back record taken on the same tower after the water level had been lowered 6 feet. This is conclusive evidence that the latter explanation is valid. Beating, although not nearly so evident, is still present. The period of beating is 11.5 seconds, and the period of the waves is uniformly 1.42 seconds. Compare this record with figure 62c taken of a similar tower tank full, on firm ground.

Professor Jacobsen states further:

For purposes of analysis we may think of the water-tank system and tower-tank system as constituting a coupled vibrating system. In this case the observed period of 1.53 seconds must be identified with the gravitational period of the water-tank system, although it, on account of the coupling, will be different from the period of a similar water-tank system isolated on a rigid foundation. In the same way, the period of 1.64 seconds must be identified with the tower-tank system, the tank containing an equivalent rigid mass of water. If the amount of water in the tank be decreased, the gravitational period identifiable with the water-tank system will increase; and the period identifiable with the tower-tank system, on account of the smaller equivalent rigid mass of water, will decrease. After the water level has been lowered 6 feet, the observed period of 1.42 seconds must be identified with the tower-tank system, and the period to be identified with the water-tank system can then be estimated from the 11.5-second period of beating. This gives the value of 1.62 seconds.

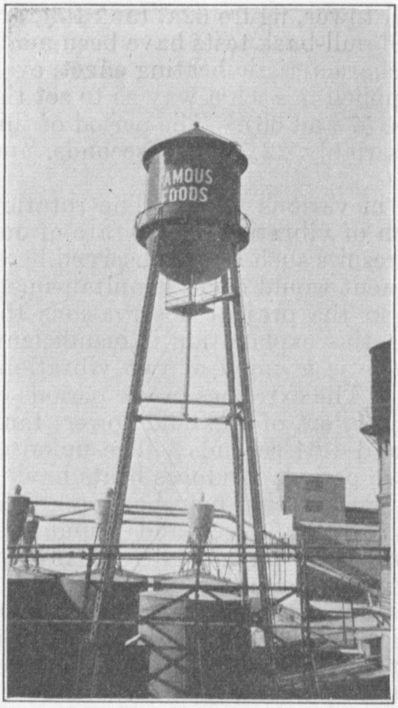


FIGURE 58.—Durkee Famous Foods tank tower.

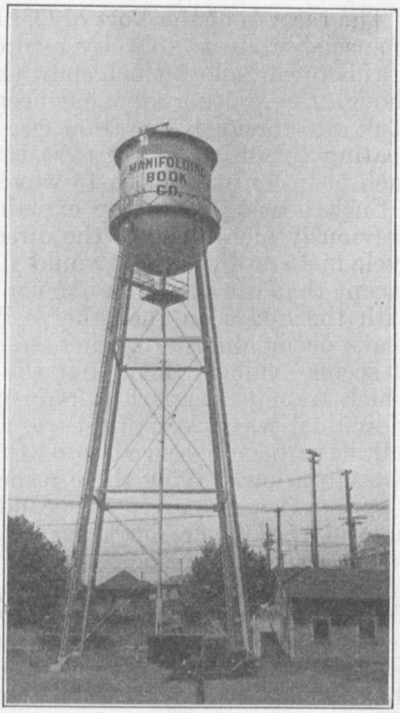


FIGURE 59.—Pacific Manifolding Book Co. tank tower.

FIGURE 60.—Port of Oakland tank tower.

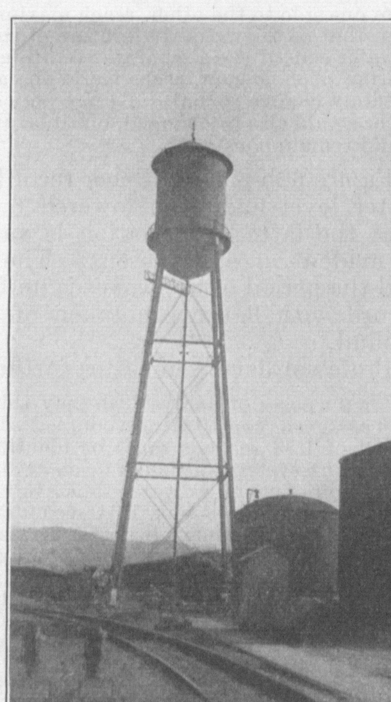


FIGURE 61.—Johns-Manville tank tower.

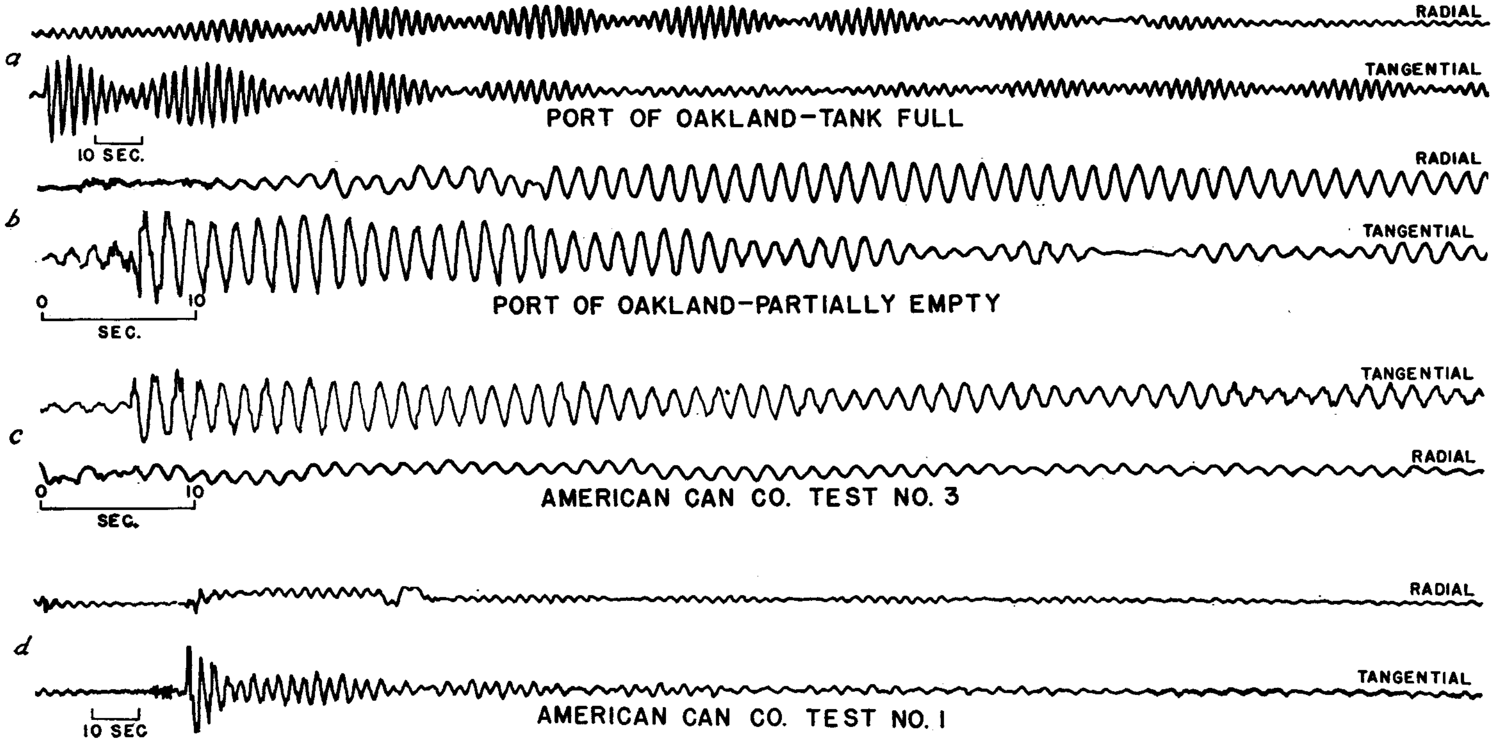


FIGURE 62.—Typical pull-back tests.

It is interesting to note that the first 4 or 5 waves after the release of the pull, tank full, usually have the shorter periods, 1.53 seconds or slightly greater. The waves in the fifth beat on the longitudinal component and in the eighth or ninth beats on the transverse component (about 90 and 180 seconds after the release, respectively) have periods near 1.64 seconds. In the fifth beat, the major axis of the

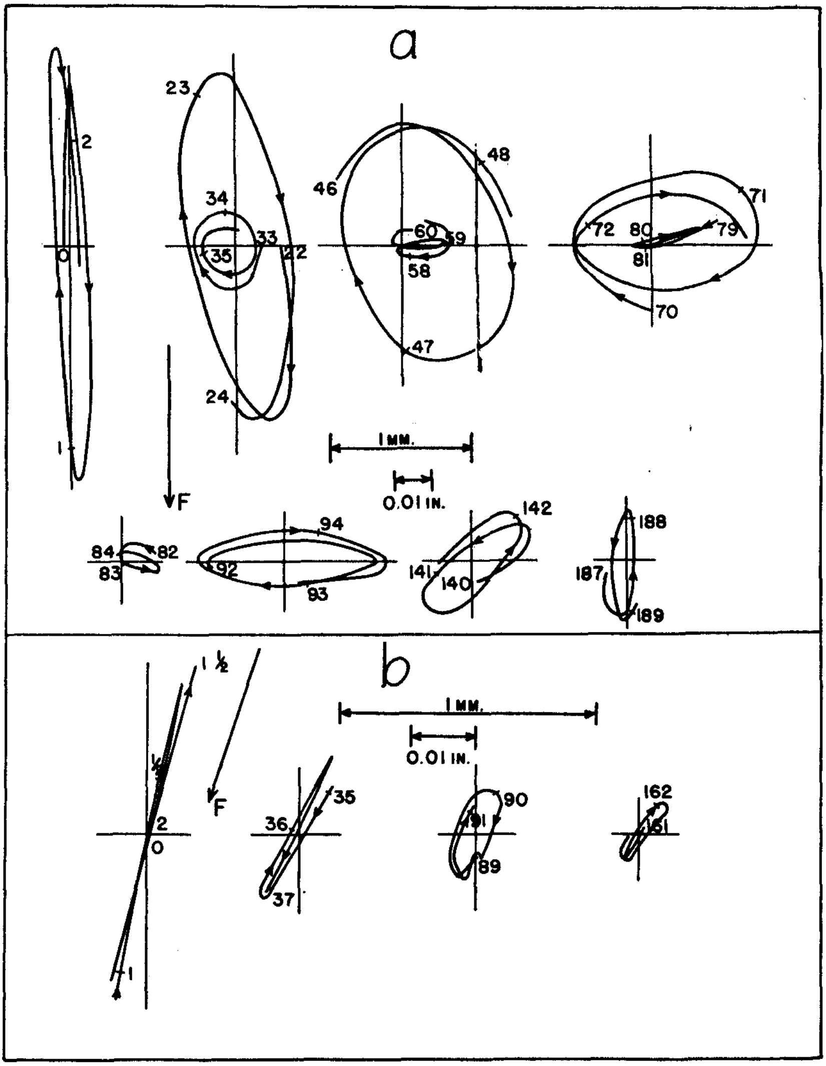


FIGURE 63.—Orbits of motion in tank vibration tests.

ellipse of vibration is transverse to the direction of the pull and in the ninth beat it is parallel to the pull (see fig. 63). Apparently, then, the water-tank system does not contribute materially to the vibration in the direction of the pull during the fifth beat, nor to the transverse vibration during the eighth or ninth beats. At these times the beating phenomenon temporarily disappears on the component concerned. The period of the waves elsewhere, not including the transitions between the beats, is 1.58 seconds, which is also the value obtained by combining the vibrations we are dealing with. The average period of all waves including the short transitional waves is 1.53 seconds.

When the record in figure 62d was taken, some of the tierods on the tower of the American Can Co., Oakland, were observed to be very loose. The natural tower vibrations seem to be damped out early on the records and a long period wave, 2.85 seconds, is dominant about a minute after the release of the pull, continuing for at least 7 or 8 minutes until the end of the record. Waves having this period appear also on wind-test records of other towers of this size including the Johns-Manville tank tower which has been reinforced and whose fundamental period is much shorter. That they also are caused by water action in the tank seems to be a reasonable explanation.

The early large-amplitude waves on the first two pull-test records of the American Can Co. tower (see table 10) have a longer period than similar waves appearing later, which in turn are slightly longer than vibrations produced by wind alone. Furthermore, the vibrations resulting from the release of the larger side load seem to have the greater periods. It has been noted that some of the tierods were very loose. It was necessary to give the turnbuckles as much as two complete turns to bring them back into proper tension. These rods probably had no initial tension and they certainly went out of action before the applied side load reached 2,000 pounds. Under these conditions, the period of vibration is not constant, and the vibrations of the larger amplitudes seem to have the longer periods.

It is well to emphasize that the concluding statement of the foregoing paragraph does not apply if the tierods all remain in tension throughout the test. The periods of all the other towers—except the Paraffine Co. tower, tank full—are independent of amplitude. This statement applies in equal measure to the American Can Co. tower after the tierods had been tightened. The period of this tower formerly was variable, 1.60 to 1.75 seconds. After tightening, it became very constant, 1.44 seconds, regardless of amplitude. The last test on the Paraffine Co. tower was made just after the tank had been filled after a reinforcing job, and the tierods had not yet been given their final cinching.

The damping ratio of a given structure may easily be determined from the records of pull-back tests. "Damping ratio" is defined as the ratio between the amplitude of a given wave crest and the amplitude of the crest (in the opposite direction) a half cycle later. It is usually represented by ϵ. The damping ratio of the tower, ϵ_t, is connected with the damping coefficient k in the differential equation of motion, viz,

$$\ddot{y}+2k\dot{y}+p^2y=0$$

by the relation

$$\epsilon=e^{\frac{k\,T_t}{2}}$$

where T_t is the damped (and also the observed) period of the tower. It is evident that cinching up loose tierods decreases the damping ratio as well as the period of the tower.

A report on a series of pull-back tests of a tank tower follows.

The vibration data for 19 selected water towers are listed, together with bracing, soil, and tierod conditions, in table 9.

Preliminary Report

DEPARTMENT OF COMMERCE
U. S. COAST AND GEODETIC SURVEY
CALIFORNIA SEISMOLOGICAL PROGRAM

TANK VIBRATION TESTS

CITY: Oakland. LOCATION OF TANK: Foot of 14th Street.

COMPANY: Port of Oakland. DATE: 12/10/34. 12/11/34. OBSERVERS: DSC, MWH, MT, CWC.

ELEVATION OF PLATFORM: 109 ft. 6 in. LOCATION ON PLATFORM: See description of tests.

SOIL: Fill overlying firm clay.

FOUNDATION: Concrete piers on piling. DOMINANT PERIOD OF GROUND: —.

WEATHER CONDITIONS: BEFORE TESTS: Fair. DURING TESTS: Fair.

WIND: VELOCITY: 300± ft. per min. DIRECTION: Variable.

DISTANCE AND DIRECTION FROM NEAREST KNOWN FAULT: About 5 miles southwest.

TYPE OF TANK: Standard (100 mi. per hr. wind design); 100,000 gal.

RELATION TO OTHER STRUCTURES: Independent.

REMARKS: Tank full. At time of tests tie rods had a relatively high initial tension. During test 6, one of the tie rods expected to go out of action was shaken by hand. The pull evidently loosened the rod, but the initial tension was by no means entirely relieved.

DESCRIPTION OF TESTS OF SERIES B: The tests in this report are classified under "Series B" to distinguish them from previous tests on the same structure which we shall class as "Series A". The purpose of pull tests under Series B is to determine the nature of motion in various directions and under different conditions of side loading. These may be described as follows:

a. The rope attached to the expension joint immediately adjacent to the bottom of the tank and the position of the instrument as shown in Fig. 64.

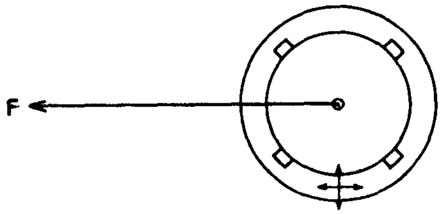


FIG.64

This arrangement was used in pull tests Nos. 1, 5, 6, and 8.

Test No. 1. The force was 2000 lbs. The instrumental record failed and no deflection was noted by the use of a transit set up on the roof of an adjacent building.

Test No. 5. The force was 2000 lbs. The drum speed was about 4 mm. per sec. The force was released by snapping a link of No. 6 calloy wire.

Test No. 6. Same as No. 5 except that the force was 3200 lbs. The release was effected by cutting one of the strands of a block and tackle arrangement. It was not very satisfactory.

Test No. 8. Same as Test No. 5 except that the drum speed was about 20 mm. per sec.

b. Rope passed around the columns just below the platform as shown in Fig. 65 used in Test No. 2. Force 2000 lbs. and drum speed 4 mm. per sec.

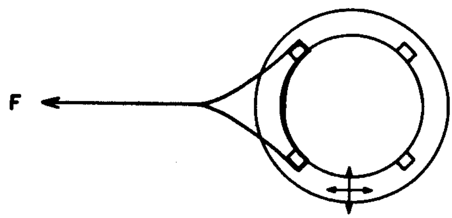


FIG.65

c. Rope attached to a single column and the pull in a direction tangent to the tank, used in tests Nos. 3 and 4.

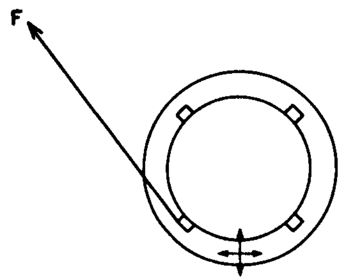


FIG. 66

Test No. 3. Instrumental record failed. The force was 2000 lbs. and was released by snapping a No. 6 calloy wire. A definite shock was noted by the observer on the platform. Drum speed 4 mm. per sec.

Test No. 4. Same as Test 3 except that an instrumental record was obtained.

d. Same as Test 5 except that instrument was set up over a column and components registered at 45° to the direction of pull, as shown in Fig. 67. Used in Test 7.

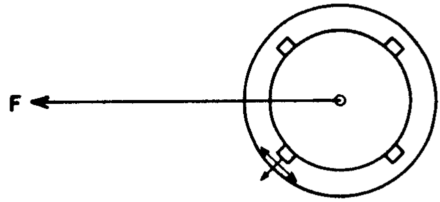


FIG. 67

e. Rope arrangement in Test 9 same as in Test 5. A single component on platform. A double component on a pier as indicated in Fig. 68. The recorders were provided with timers marking simultaneously. The force was 2000 lbs. and drum speed 10 mm. per sec. on upper recorder.

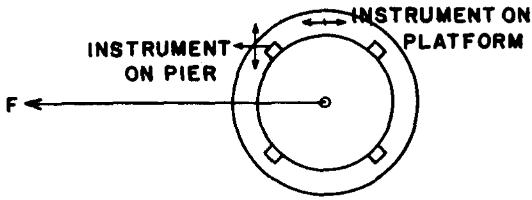


FIG. 68

INSTRUMENTS USED IN TESTS 1 TO 8 INCLUSIVE: Spring recorder; Survey vibration meters Nos. 6 and 7. Pendulum period, 0.80 sec.; static magnification, 105; damping, almost critical.

INSTRUMENTS USED IN TEST 9: Spring recorder, Survey vibration meter No. 1 on platform; A. C. motor recorder, meters 6 and 7 on pier; simultaneous timing. Pendulum period, 0.67 sec. on platform, 2.5 sec. on pier; static magnification, 110 on platform, 200 on pier; damping ratio, 2 ca. on platform, 8 on pier.

TABLE 8. PERIODS OF PORT OF OAKLAND TANK

Test No.	Force Applied	Component	Periods	A_{mt} [1]	A_{it} [2]	$2A_o$ [3]	A_o [4]	Remarks
	lbs.		sec.	mm	mm	mm	mm	
1	1800		Lost				?	See note 1.
2	Wind	Tang.	1.56±.02; 0.350±.005		2			
		Rad.	1.56 ca; 0.34±.01					Irregular.
	1800	Tang.	1.54 to 1.63; 0.35	31	30	3.0	2,3	See notes 2 and 3.
		Rad.	1.57 to 1.64	12		1.2		
3	2000		Lost				2½, 3.3	
4	Wind	Tang.	1.57 ca	6				
			0.35 ca	10				
		Rad.	1.60 ca	3				Torsional pull.
			0.35 ca	1				
	2000	Tang.	1.58±.01 0.350	65 15	65	3.8	2+ 3+	65 mm refers to 0.350 sec. period. 3.8 mm is compounded from T and R with secondary added. See notes 2, 4.
		Rad.	1.57±.01; 0.34 ca	19	19			
5	Wind	Tang.	1.55±.02; 0.33	3				
		Rad.	1.55±.02;0.1ca	1				
	2000	Tang.	1.55 to 1.60; 2.8 ca	20	20	2.8	1¾, 2½	2.8 sec. ca is near end of record.
		Rad.	1.54 to 1.63; 2.8 ca	11		1.1		See notes 2, 3, 5.

6	Wind	Tang.	1.55 ca 0.345 ca	4 1½				
		Rad.	0.1 ca	1				
	3100	Tang.	1.54 to 1.60	29	18	2.8	2.1, 3.5	Permanent set of 1 mm toward force.
		Rad.	1.53 to 1.63; 0.1 ca	12	2	1.2		2 mm amplitude built up slowly. See notes 2, 3, 6.
7	Wind	Tang.	1.58 ca; 0.34 ca					
		Rad.	1.60 ca; 0.1 ca					Diagonal registration.
	1700	Tang.	1.57 to 1.61	14	10	2.0	1½, 3	2.0 mm amplitude is compounded from T and R. See note 7.
		Rad.	1.55 to 1.58	15	9			
8	Wind	Tang.	1.20 ca 0.35	5 1				
		Rad.	0.105±.005	4				
	1800 to 2000	Tang.	1.54 to 1.57	25	17	2.4		
		Rad.	1.55±.01; 0.10 ca					
9	Wind	Long.[5]	1.60±.02; 0.346					
tower	2000	Long.	1.58±.02; 1.62; 0.35	21	21	2.0		See notes 2, 3, 8.
pier	Wind	Long.	1.57; 0.17					
		Transv.[6]	? ?					
	2000	Long.	1.60±.02; 0.174	1.7	1.5	0.02		See note 8.
		Transv.	1.57±.01; 0.27	0.6				

[1] Maximum single trace amplitude.

[2] Initial impulse on trace after the release.

[3] Computed maximum displacement of the tank from position of rest.

[4] Displacement of tank observed through a transit. The first figure refers to the displacement of tank due to pull before release of the force; the second refers to the extreme movement immediately after the release.

[5] Longitudinal-parallel to pull.

[6] Transverse-at right angles to direction of pull.

NOTES. 1. Under column headed "Force, lbs.", multiply by 0.87 to get the actual horizontal force. Under the column headed "Components", "Tangential" refers to the direction tangent to the tank and "Radial" to the direction perpendicular thereto. In most cases, the tangential component is in the direction of the pull.

2. All waves from records of pull tests of the Port of Oakland water tower show a definite beating in all directions. The maxima in the direction of pull occur shortly after the snap-back and recur at intervals of 23 sec. in gradually diminishing amplitudes. The snap-back is not recorded definitely on the component transverse to the pull but a maximum builds up

23 sec. later, and thereafter at intervals of 23 sec., simultaneously with the longitudinal component. The actual maxima of the beats gradually diminish, but on a single component it may seem to increase after the fifth or sixth beat. This is due to the plane of vibration shifting to the transverse direction then back to the longitudinal. It is interesting to note that this 23 sec. interval between the maxima of the beats is the same as recorded in earlier pull tests.

3. The tangential component on tests 2, 5, 6, and 9 begins with a period of 1.57 sec. This period increases to about 1.62 sec. in the fifth maxima and becomes 1.57, or thereabouts, in succeeding beats. The longer period on the radial component occurs in the 9th and 10th maxima.

   A 0.34 sec. wave appeared immediately before the snap-back, and was probably caused by the tackle slipping in the blocks.

4. The dominant wave on the record of the tangential component in test 4 has a period of 0.345 sec. An underlying wave having a period of 1.6 sec. is evident. The shorter wave is subordinate on the radial component, and after 20 sec. becomes subordinate on the tangential, and thereafter the motion behaves as in pull test 2, except that the 1.65 sec. period does not appear. As this pull produced rotation of the tank, and as the shorter period was dominant on the tangential component only, it seems definite that rotation is one of the causes of the secondary wave observed on records of vibrations due to wind.

5. The record of test 5 is very similar to that of test 2 except that the break was much cleaner. The secondary motion just before the break is not so conspicuous.

6. The record of test 6 is much the same as in tests 2 and 5. The amplitude is much too low for a supposed increase in force, but here the release was made by cutting a rope through a block and tackle arrangement and not a sudden snapping of a wire link directly in the line.

7. The beating effect on record of test 7 begins with equal intensity on both components. The components here are 45° from the direction of pull. Otherwise, the record differs but little from those of other tests.

8. The record on the platform was taken simultaneously with that of the movement on the pier. The movement on the pier shows beating, the same as that on the platform. There seems to be but little difference in the periods of vibration at the two places. The amplitudes at the platform are observed to be about 100 times those at the pier. It is interesting to note that the secondary vibrations of the pier have a different frequency than the secondary vibrations of the platform.

9. The damping ratio of the tank tower was computed from the records to be $1.015 \pm .005$.

TABLE 9.—*Tank tower vibration tests*

100,000-GALLON TANKS. 100 FEET TO BOTTOM

No.	Tank tower [8]	Date	Foundation	Tie rods	Mean values of periods [1]	Displacement	Wind velocity	Remarks
					Seconds	*0.001 inch*	*Ft./min.*[2]	
1	Port of Oakland [3]	June 15, 1934	Concrete piers on piling in hydraulic fill.	Tight	1.58; 0.35; (2.8); (0.12).[1]	5.0	Moderate	Water 1 ft. from top. Test made before rebracing tower.
2	American Can Co., Oakland [3]	Sept. 18, 1934	Concrete piers. Firm clay.	Very loose	1.61; 0.36; (0.11)	3.0	310	Water 0.8 ft. from top.
		Mar. 22, 1935	do	Very tight	1.44; 0.33; (0.11)	25.0	2,500	Practically full.
3	Fageol, Oakland [3]	Sept. 21, 1934	Concrete piers. Firm ground.	Tight	1.56; 1.52; 2.9; 0.40; 0.31.	1.5	790	Water 1 ft. from top. Large sign beneath tank.
4	Port of Stockton [3]	Sept. 20, 1934	Concrete piers on piling in river loam.	do	1.70; 0.42; 0.38	7.0	740	Water 0.6 ft. from top.
5	California Conserving Co., Hayward [3]	Sept. 21, 1934	Concrete piers. Clay	Moderate	3.0; 1.65; 0.42			Water 2.5 ft. from top.
		do	do	do	1.75; 1.60; 0.42; 2.9	9.0	1,110	Water 0.5 ft. from top.
		do	do	do	1.64; 0.39			Water 0.5 ft. from top. At right angles to preceding test.
6	Johns-Manville, Pittsburg [4]	July 23, 1934	Concrete piers. Firm ground.	Tight	1.03; 0.26; 2.85	15.0	2,000	East-west motion. Water 4 ft. from top at start.
		do	do	do	1.30; 0.25; 2.55 [5]		2,400	North-south motion. Tank full.
7	Pacific Manufacturing Co., Santa Clara.[3]	Sept. 24, 1934	Concrete piers. Firm clay	do	1.43; 0.30; 0.15	3.0	300	Tank full. Fairly uniform tension on rods.
8	Pratt-Lowe, Santa Clara [3]	do	do	Very loose to tight	1.59; 0.30	2.0	400	Tank overflowing. Some rods out of action; others tight.
	75,000-GALLON TANKS. 100 FEET TO BOTTOM							
9	National Motor Bearing Co., Oakland.[3]	Sept. 18, 1934	Concrete piers. Adobe	Loose and tight	1.43; 0.34	3.0	350	Water 1.5 ft. from top. Some rods out of action. 9 and 10 are identical towers.
10	Springfield Cedar Co., Oakland.[3]	Sept. 21, 1934	do	do	3.0; 1.45; 1.38; 0.35	5.2	630	Do.
11	Paraffine Co., Emeryville [3]	June 14, 1934	Concrete piers on piling in filled ground.	Probably loose	3.0; 1.60; 1.55; 0.47; 0.43.	5.6	Moderate	Tank full.
	Paraffine Co., Emeryville [6]	Mar. 20, 21, 1934.	do		1.10; 0.45; 0.25; 0.08	5.5	1,500	Tank full. After reinforcing.
	do	do	do		0.89		do	Water 8 ft. from top.

See footnotes at end of table.

TABLE 9.—*Tank tower vibration tests*—Continued

50,000-GALLON TANKS. 100 FEET TO BOTTOM

No.	Tank tower [8]	Date	Foundation	Tie rods	Mean values of periods [1]	Displacement	Wind velocity	Remarks
					Seconds	*0.001 inch*	*Ft./min.*[2]	
12	S. H. Frank Co., Redwood City.[7]	Aug. 10, 1934	Concrete piers on piling in marshy fill. Clay underneath.	Tight	1.12; 0.23	4.0	500	Tank probably full.
13	Standard Brands, Oakland,[7] highest tower.	Aug. 22, 1934	Concrete piers. Firm adobe.	Very tight	0.95; 0.19	3.1	650	Water 0.8 ft. from top. Tank on 3-foot riser.
14	Guggenheim and Co., San Jose.[4]	Sept. 27, 1934	Concrete piers. Adobe	Moderate	1.22; 0.29; 0.1ca	3.7	260	Tank full.
15	Norton Wool Co., San Francisco.[4]	Aug. 27, 1934	Cross-braced footings on piling in soft fill and clay.	Tight to very tight	1.23; 0.28	12.0	1,800	Water 1 ft. from top.
16	Standard Sanitary Products Co., San Pablo [5].	Sept. 19, 1934	Concrete piers. Adobe	Tight	2.63; 1.33; 0.32	6.5	1,030	Water 0.6 ft. from top.
17	Stauffer Chemical Co., Stege.[3]	do	do	Loose to moderate	1.32; 0.30	18.0	1,610	Water 0.8 ft. from top.
18	Western Plywood Co., Martinez.[3]	July 31, 1934	Concrete piers on piling in Bay silt.	Moderate	1.53; 1.47; 0.34	6.0	400	Water 3 ft. from top. Add 0.1 sec. to period for a full.
19	Durkee Famous Foods, Berkeley.[3]	Dec. 6, 1934	Concrete piers. Firm ground.	Tight	1.26; 0.30; 0.1ca	2.0	Light	Probably full.

[1] Periods in parentheses are from tests made at other times. All vibrations shown in this table are due to wind.
[2] Or Beaufort scale described in table 3.
[3] 100 m. p. h. wind design.
[4] Designed to resist 8 percent of gravity. Johns-Manville tank reinforced for 10 percent before pull back tests in February 1935.
[5] Period doubtful.
[6] Designed to resist 12 to 15 percent of gravity.
[7] Designed to resist 10 percent of gravity.
[8] Tanks 1, 2, 3, 4, 5, 6, 11, 14, 15, and 16, also tanks 21, 24, 25, 29, and 37, which are not shown in this table, were fabricated by the Chicago Bridge & Iron Works.
Tanks 7, 8, 9, 10, 12, 13, 17, 18, and 19, also tanks 20, 22, 23, 27, 28, 30, 32, 33, 34, 35, and 36, which are not shown in this table, were fabricated by the Pittsburgh-Des Moines Steel Co.

The most accessible tierods were tested on towers in table 9 on the ne day, and on two towers on the following day. The degree of htness in a rod was judged by the amount by which the center uld be moved by shaking the rod with a given estimated force plied at a given estimated distance from the lower end. If the rod emed to be out of tension, it was classed as "very loose."

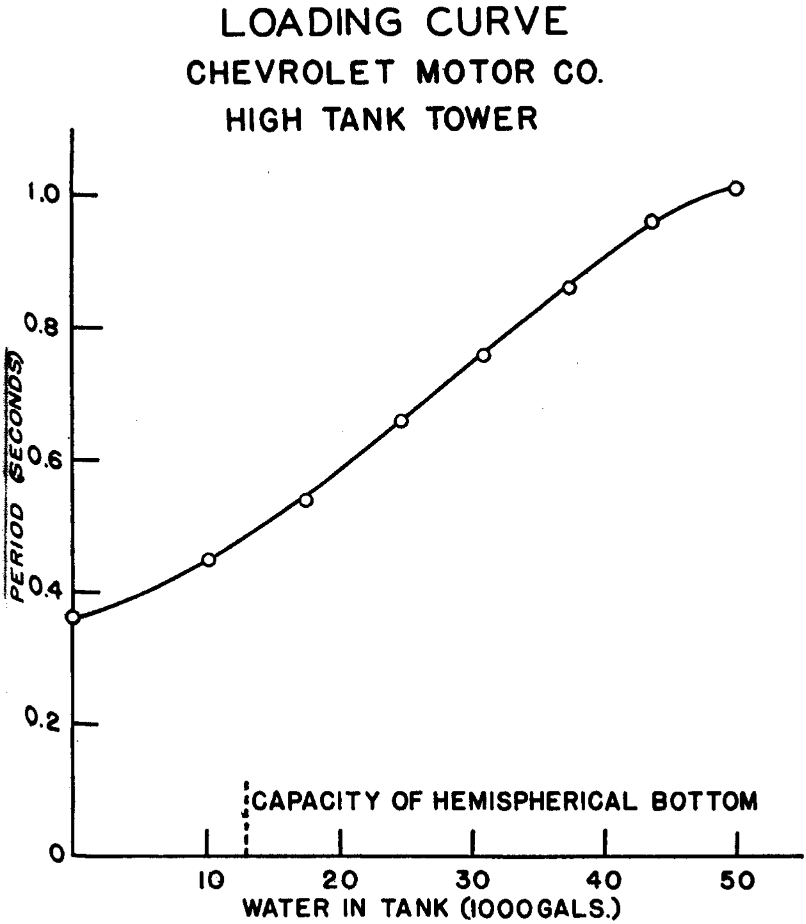


FIGURE 69.

The results of the pull-back tests are given in table 10. Table 11 ts the data for the tested towers that have been reinforced during is investigation. The vibration and structural data for the re-aining towers are listed in table 12. Figure 69 shows graphically e variation of period with water load of a tank.

TABLE 10.—*Tank tower pull-back tests*

100,000 GALLON TANKS, 100 FEET TO BOTTOM

No.	Tank tower [1]	Date	Force	Mean values of periods	Displacement	Damping ratio of tower	Remarks
			Lbs.	*Seconds*	*0.001 inch*		
1	Port of Oakland	Dec. 10–11, 1934	2,000	1.54 to 1.64; 0.35; 2.8.	[2] 112	1.007	Tank full; straight pulls; rods tight on all tests.
		do	2,000	1.58; 0.350	[2] 152		Tank full; torsional pulls; 0.35 sec. wave dominates.
		do	2,000	1.60; 0.174	1		Pier, parallel to pull.
		do	2,000	1.57; 0.27			Pier, transverse to pull.
		Mar. 26, 1935	2,000	1.42; 0.34; 0.11	76	1.010	Straight pulls; water 6 feet from top.
		Dec. 10–11, 1934	3,100	1.53 to 1.63; 0.34; 0.1.	[3] 112		Tank full, poor release.
		do	Wind	1.57; 0.33; 0.1	1		Tank full.
2	American Can Co., Oakland.	Dec. 12, 1934	2,000	1.65 to 1.73; 2.85; 0.38; 0.11.	112	1.06	Tank full; tierods loose.
		Mar. 22, 1935	2,000	1.44; 0.31; 0.11	56	1.016	Water 0.8 foot from top; rods tight.
		Dec. 12, 1934	3,000	1.66 to 1.75; 2.85; 0.38; 0.11.	152	1.09	Tank full; tierods loose.
		do	Wind	1.61; 1.55; 0.38; 0.11.	4		Do.
		Mar. 22, 1935	Wind	1.44; 0.33; 0.11	6		Water 0.8 foot from top; rods tight.
6	Johns - Manville, Pittsburg.	Feb. 26, 1935	2,000	1.30; 0.29; 0.11	40		Tank full.
		do	2,000	1.30; 0.3 to 0.4	1		Pier.
		do	Wind	1.30; 0.27; 0.11	1		Tank.

75,000 GALLON TANK, 100 FEET TO BOTTOM

No.	Tank tower [1]	Date	Force	Mean values of periods	Displacement	Damping ratio of tower	Remarks
11	Paraffine Co., Emeryville.	Mar. 20–21, 1935	2,000	0.89	[4] 96	1.010	Water 8 feet from top; tower previously reinforced.
		do	2,000	1.13; 0.45; 0.22	88	1.014	Tank full.
		do	Wind	1.10; 0.45; 0.23; 0.08.	12		Tank full; after reinforcing.
		June 14, 1934	Wind	1.60; 1.55; 0.47; 0.43.	17		Tank full; before reinforcing.

50,000 GALLON TANKS, 100 FEET TO BOTTOM

No.	Tank tower [1]	Date	Force	Mean values of periods	Displacement	Damping ratio of tower	Remarks
12	S. H. Frank and Co., Redwood City.	Feb. 19, 1935	2,000	1.09; 0.25	92	1.014	Tank full.
		do	2,000	0.4			Pier.
		do	Wind	1.10; 0.25; 0.12	4		Tank.
13	Standard Brands, Oakland; highest tower.	Dec. 19, 1934	2,000	0.65; 2.6	[5] 84		Water 12 feet from top.
		do	2,000	0.96; 2.55	84	1.016	Tank full.
		do	Wind	0.65			Water 12 feet from top.
		do	Wind	0.98	6		Tank full.
19	Durkees Famous Foods, Berkeley.	Dec. 6, 7, 1934	2,000	1.26	[3] 128	1.009	Tank full; rods tight.
		do	3,700	1.26; 0.30	[6] 240	1.011	Tank full.
		do	Wind	1.26; 0.30	20		Do.

[1] See footnotes of table 9 for side load designs.
[2] Displacement observed with transit, 0.125 inch.
[3] Displacement observed with transit, 0.140 inch.
[4] Record off both sides of sheet.
[5] Displacement observed with transit, 0.100 inch. Record off sheet.
[6] Displacement observed with transit, 0.260 inch.

TABLE 11.—Tank tower tests under different loading and bracing conditions

No.	Tank tower	Capacity	Platform height	Bracing design [1]	Date	Height of water	Condition of bracing	Average values of periods [2]	Displacement [3]	Wind velocity
		Gallons	*Feet*					*Seconds*	*0.001 inch*	*Ft./min.*[4]
11	Paraffine Co., Emeryville[5]	75,000	110	100 miles per hour	June 14, 1934	Full	Unbraced	(1.60, 1.55); (0.46, 0.42)	5.5	([6])
				15 percent	Mar. 20, 1935	do	Braced	1.10; 0.45	5.5	1,500
20	Chevrolet Motor Co., Oakland.[7]	50,000 and 25,000	119	100 miles per hour	Aug. 21, 1934	do	Unbraced	(1.44, 1.32); 0.30	3.0	1,470
				do	Oct. 15, 1934	Empty	do	0.42; 0.24	2.2	1,040
				10 percent	Nov. 9, 1934	do	Braced	0.36; 0.20	1.5	200
				do	Nov. 12, 1934	Full	do	1.01; 0.21	0.8	200
21	Fisher Body Plant, Chevrolet Motor Co., Oakland.[8]	50,000	64	100 miles per hour	Aug. 21, 1934	do	Unbraced	(1.22, 1.07); (0.26, 0.32)		600
				do	Sept. 13, 1934	Empty	do	0.24; 0.30	0.6	500
				20 percent	Oct. 3, 1934	do	Braced	0.42; 0.24; 0.14	2.5	980
				do	Oct. 10, 1934	Full	do	(0.74, 0.80); 0.16	1.7	1,100
22	Pacific Manifolding Book Co., Emeryville.	50,000	83	100 miles per hour	July 26, 1934	do	Unbraced	(1.04, 1.07); 0.26	2.0	530
				do	Oct. 3, 1934	Empty	do	(0.55, 0.35)	3.0	360
				10 percent	Oct. 10-11, 1934	do	Braced	0.32, (0.24; 0.19)	0.6	470
				do	Oct. 11, 1934	Full	do	(0.94, 0.91); (0.19, 0.24)	1.0	340
23	Pacific Coast Shredded Wheat Co., Oakland.	60,000	119	100 miles per hour	Aug. 1, 1934	Empty	Unbraced	(0.55, 0.39)	2.3	430
				8 percent	Aug. 21, 1934	do	Braced	0.40; 0.21	5.5	740
				do	Sept. 14, 1934	Full [9]	do	1.13; 0.25	6.0	940
24	Western Cooperage Co., San Francisco.	50,000	74	15 percent	Aug. 23, 1934	Empty	do	0.36	2.2	790
				do	Sept. 7, 1934	Full	do	0.88; 0.36	0.8	890

[1] Wind bracing is given in miles per hour; bracing against earthquake movements in percentage of gravity.
[2] When periods of the 2 components are different they are both given and placed in parentheses.
[3] Displacement of platform computed from position of rest.
[4] Also see Beaufort scale of wind force in table 3.
[5] Tower on piling.
[6] Moderate.
[7] Tower extends through a 2-story building and has independent foundation. Reconstruction work began about Oct. 15, 1934. Smaller tank which was removed during construction was built in tower directly beneath the larger tank.
[8] Tower rests on concrete columns built in and tied to 2-story reinforced concrete building. By Oct. 3 the tower had been rebraced.
[9] Water 2.8 ft. from top.

TABLE 12.—*Periods of miscellaneous tank towers*

No.	Tank tower	(a) Capacity (b) Platform height	Foundation and soil	Date	Mean values of periods [1]	Displacement [2]	Wind velocity	Remarks
		(a) Gallons, (b) feet			*Seconds*	*0.001 inch*	*Ft./min.*[3]	
25	American Can Co., Sacramento	(a) 100,000 (b) 85.	Concrete piles in alluvial loam.	Sept. 20, 1934	1.22; 0.28	4.6	680	Water 0.2 foot from top.
26	California and Hawaiian Sugar Refining Co., Crockett.[4]	(a) [4] (b) 22.	([4])	Aug. 20, 1934	(0.94, 0.90); 0.27	0.4	420	Two 25,000-gallon tanks full; others empty; 9-story building.
27	De Vaux-Durant, Oakland	(a) 75,000 (b) 101.	Concrete piers in firm clay	Aug. 2, 1934	1.45; 0.40	7.0	950	Water 2.5 feet from top.
28	City of El Centro, El Centro	(a) 100,000 (b) 75.	Concrete piers in silt	Feb. 19, 1935	0.94	1.0	([12])	Water 11 feet from top.
29	Do	(a) 250,000 (b) 80.	do	do	(1.15, 1.09)	0.8	([12])	6-column tower; elliptical bottom on riser; tank full.
30	Ford Motor Co., Richmond	(a) 100,000 (b) 133.	Concrete piers on piling in fill.	July 24, 1934	1.48; 0.37	8.5	1,030	0.37-second period has larger amplitude than fundamental; tank full.
31	W. P. Fuller Co., South San Francisco.	(a) 50,000 (b) 50.	Concrete piers in firm ground.	Aug. 9, 1934	0.70; 0.22	2.0	1,750	Flat bottom wood tank on 50-foot steel tower; tank full.
32	Hercules Powder Co., Hercules [5]	(a) 100,000 (b) 125.	do	July 25, 1934	(1.16, 1.20); 0.26	3.0	330	Water 1.2 feet from top.
33	Standard Brands, Oakland [6]	(a) 100,000 (b) 65.	Concrete piers in adobe	Aug. 22, 1934	0.24; 0.10	0.6	1,320	Tank empty.
34	Standard Brands, Oakland [7]	(a) 100,000 (b) 65.	do	do	(0.52, 0.30); 0.30	0.6	1,060	Do.
35	U. S. Navy, Sunnyvale [8]	(a) 200,000 (b) 100.	do	Sept. 28, 1934	1.06; 0.10 [9]	0.8	800	Water 1.0 foot from top; level lowered somewhat during test.
				do	1.06; 0.19 [10]	2.0	800	
				do	1.07; 0.2? [9]	1.2	800	
				do	1.13; 0.19 [10]	1.2	800	
				do	1.12; 0.12 [9]	2.0	800	
				do	1.09; 0.2? [10]	2.4	800	
36	U. S. Veterans' Administration Bureau, Palo Alto.[11]	(a) 100,000 (b) 107.	Concrete piers in firm soil	Aug. 10, 1934	(0.78, 0.83); 0.33	2.5	870	Tank about half full (water 10 feet above stand pipe).
37	Willard Storage Battery, Los Angeles.	(a) 60,000 (b) 92.	Concrete piers in alluvium	Aug. 15, 1934	(1.51, 1.41); 0.19	7.5	1,500	Tank full.

[1] Parentheses are used when periods on 2 components are different.
[2] Measured from position of rest.
[3] See Beaufort scale of wind force in table 3.
[4] Two 25,000-gallon tanks and 2 smaller tanks are set on platform supported by steel framework on roof of 9-story steel and concrete charhouse. Continuous concrete arches rest on 16- to 20-foot piling driven to shale. 22-foot platform braced for 10 percent side loading.
[5] Braced for 8 percent side loading.
[6] Tank no. 1 (raw water), 4-foot riser, 2-panel, elliptical bottom tank with foundation crosstied. Braced for 10 percent side loading.
[7] Tank no. 2 ([illegible] water), 4-foot riser, 2-panel, elliptical bottom tank with foundation [illegible]. Braced for 10 percent side loading.

Certain conclusions may be derived from these data. From table it is indicated that, other things being equal—

1. Additional bracing effectively reduces the period of the tower, and the eater the amount of bracing the greater the reduction of the period. Compare e periods of towers 6, 11, and 12 to 15 with the periods of other towers in the spective groups. Compare also the periods of towers 12 and 13 with those of and 15.

2. The period of a tower having loose tierods is reduced if the tierods are htened. If the tension in the tierods of a tower is uniform throughout, the riod is probably the same in directions at right angles to each other. A differ- ce in period in the two directions has been observed where some of the rods ve been slackened prior to reinforcing. See table 11.

3. Towers on yielding ground where piling is required usually have a greater riod than towers on firm ground. However, there is strong evidence that me of the piling we are dealing with has been driven through compact material. r example, the Port of Oakland tower rests on piling driven through hydraulic l. If comparative periods of water towers can be relied upon, it seems likely at this fill is relatively compact.

As the motion of water in the tank tends to complicate the records, comparison of periods of empty tanks would be more reliable. rom table 10 it is seen that—

1. The period of vibration is independent of the amplitude for towers having preciable initial tension in all of their tierods. If, however, some of the tierods e very loose, and especially if some are out of action or go out of action as the wer swings to one side, the vibration of the structure is irregular and the period greater, the large-amplitude vibrations having the greatest periods.

2. The method of arriving at the order of actual displacement by reducing ace amplitudes through the use of instrumental constants is reliable. A com- rison of the figures in the displacement column with the corresponding "transit" splacements given in the footnotes verifies this. The tabulated displacements re calculated from the trace, while those in the footnotes represent displace- ents observed simultaneously with a transit.

3. The amplitude of vibration resulting from the release of a side force is oportional to the force, provided each release has the same degree of suddenness.

4. Reinforced or well-braced towers show less displacement than similar wers not so reinforced.

5. Larger tanks are displaced less under a given side load than smaller tanks, ubtless because their towers have been more heavily braced.

6. Towers on yielding ground, where piling is required, seem to be displaced rther under a given side load than similar towers on firm ground.

7. The ratio of the displacement of the tank to that of the pier is less if the ound is yielding (where piling is required). These ratios have been observed be 150 :1 or less for the Port of Oakland tower which is on hydraulic fill, and 0 :1 or more for the Johns-Manville tower, which is on firm ground.

8. The fundamental translatory vibration of the tower is transmitted to the er, but secondary vibrations of the tank and pier have been observed to have fferent periods.

BRIDGE AND BRIDGE PIER VIBRATIONS

The Colorado Street Bridge in Pasadena has been set into forced bration and tested in detail. This investigation is fully discussed another chapter. Messrs. McLean and Moore have tested longi- dinal vibrations at the center of the long span while the span was ing loaded by driving a heavy Buick sedan across it. They found at loading of the span caused a definite change in slope of the road- ay. They also found indications that the loading of an adjacent ch caused bending in the arch being tested. A report of the easurements of wind and traffic vibrations of the Colorado Street ridge, of the type which has been sent out in the preliminary form interested persons and organizations follows.

Preliminary Report

DEPARTMENT OF COMMERCE
U. S. COAST AND GEODETIC SURVEY
CALIFORNIA SEISMOLOGICAL PROGRAM

BRIDGE VIBRATION TESTS

CITY: Pasadena. LOCATION: West Colorado Street crossing Arroyo Seco.

STRUCTURE: Colorado Street Bridge. DATE: 12/7, 10/34. OBSERVERS: McL. M.

POSITION ON BRIDGE: Roadway; center and piers of 220 ft. span; center of second span east.

INSTRUMENT: Wood-Anderson Seismometer. STATIC MAGNIFICATION: 1400.

DAMPING RATIO: 8. PENDULUM PERIOD: 2.5 sec.

TABLE 13.--PERIODS OF PASADENA BRIDGE VIBRATION TESTS

Location & Date	Component	Periods	Maximum single amplitude on trace	Remarks
		sec.	in.	
Center of long span 12/7/34	T*	0.86±.02;	1.5	
12/10/34	L**	0.85±.01; 0.45±.01	0.5	Special loading tests
Over pier 2 from W., 12/10/34	T	0.84±.02; 0.7? 0.56?	0.5	
	L	0.44±.01;	0.4	
Center of span 4 from W., 12/10	T	0.85±.02; 0.7?	0.3	
	L	0.44±.01;	0.3	
Over pier 3 from W., 12/10/34	T	0.85±.01;	0.8	
	L	0.43±.01;	0.2	
Center of long span 2, 12/10/34	T	0.85±.01; 0.45±.01; 0.38±.03	0.5	
	L	0.86±.03;	0.2	

* T = component transverse to roadway.
**L = component parallel to roadway.
On all records there are short period vibrations (0.07 to 0.10 sec.) which are probably due to traffic.

SOIL: Rock under west pier of 220 ft. span. Poorly consolidated alluvium under east pier and under piers of second span east.

STRUCTURE: Reinforced concrete.

WEATHER CONDITIONS: BEFORE TESTS: Clear before 12/7. Rain before 12/10.

DURING TESTS: Clear.

WIND: Velocity, very low. Character, varying.

DISTANCE AND DIRECTION FROM NEAREST KNOWN FAULT: Very close to Eagle Rock fault.

ALLOWABLE BEARING VALUE OF SOIL: --. DOMINANT PERIOD OF SITE: --.

RELATION TO OTHER STRUCTURES: Independent.

Vibrations of the Southern Pacific bridge across the Suisun Bay caused by wind alone and by a passing train were measured with the following results:

TABLE 14.—*Observations on Suisun Bay Bridge, July 31, 1934.*

[Components: Transverse (T); longitudinal (L)]

Location of instrument	Mean values of periods	Displacement [1]	Remarks
	Seconds	*0.001 inch*	
Accelerograph room on Pier 14, 2,222 feet from south approach.	(T) 0.60; 0.33	1.6	Wind alone.
	(L) 0.56	1.6	Do.
	(T) 0.7 [2]; 0.33; 0.05 [2]	(?)	Train crossing bridge.
	(L) 0.64; 0.24; 0.05 [2]	14	Do.

[1] Maximum displacement measured from position of rest.
[2] Approximate values.

The greatest amplitudes occur just as the locomotive reaches the pier. The transverse record of the train passing over is incomplete. The pier is constructed of reinforced concrete. It is 20 by 55 feet at the base and 20 by 49 feet at the top. It is 174 feet high above its base on bedrock, which is shale; 64 feet of it is above water, 50 feet is in water, and 60 feet is in mud. The Southampton Fault parallels the bridge and lies 2 miles to the west.

Mr. Henry Dewell suggested that periods of the reinforced concrete piers of the San Francisco-Oakland Bay Bridge and the Golden Gate Bridge, which were under construction, be measured (1) before the steel of the superstructure was placed upon them, and that subsequent measurement of the periods be made; (2) when the steel towers are completely in place; (3) when the cables are completely in place; and (4) when the bridges are finally completed. Acting upon this suggestion, the periods of 14 piers of the San Francisco-Oakland Bay Bridge and 3 piers, including San Francisco pylon no. 1 of the Golden Gate Bridge, have been measured. Marin Pier, of the Golden Gate Bridge, and Piers W-2 and W-3 of the San Francisco-Oakland Bay Bridge had the towers in place or under construction at the time these measurements were started, and Pier E-1 was so rigid that little or no motion was recorded. The results of the measurements are given in table 15.

TABLE 15.—*Vibration observations on bridges*

[Components: Transverse (T): longitudinal (L)]

OAKLAND BAY BRIDGE

Location	Date	Average values of periods	Displacement[1]	Remarks
		Seconds	*0.001 inch*	
Pier W-1	Apr. 9, 1935	(L) 0.54; 0.28; 0.10; 0.05	0.1	Motion apparently nc fected by walk cables.
	do	(T) 0.29; 0.07	0.1	Do.
Pier W-2	Sept. 6, 1934	(L) Nothing measurable.		Tower on pier. See t
	do	(T) 3.0; 0.30	0.3	
Pier W-3	Nov. 14, 1934	(L) 3.5; 0.77; 0.50	0.6	Tower on pier.
	do	(T) 1.02; 0.5[2]	0.2	
Top of tower, Pier W-3	do	(L) 3.52; 1.0[2]	210; 1000[3]	Wind 30 miles per across tower.
	do	(T) 3.55[4]; 1.03	10; 90[5]	Do.
Pier W-4	Feb. 15, 1935	(L) 0.62	0.1	
	do	(T) 4.0[7] or 2.5[7]; 0.99	0.7	
Pier W-5	Oct. 19, 1934	(L) 2.6; 0.44	0.1	
	do	(T) Nothing measurable.		
Pier W-6	Sept. 6, 1934	(L) 3.2; 0.57	0.8	
	do	(T) 3.5[2]		
Pier E-1	do	(L) 0.16 to 0.17	0.02	
	do	(T) Nothing measurable.		
Pier E-3	do	(L) 2.5 to 3.5; 0.6; 0.9[2]; 0.45[2].	0.1	0.6 second appears to be dominant period. ord is irregular.
	do	(T) 3.0 to 4.0; 0.6; 0.45	0.05	Do.
Pier E-4	do	(L) 3.6[2]; 0.47	0.2	
	do	(T) 3.6[2]; 0.58	0.2	
Pier E-5	do	(L) 3 to 5; 0.7[2]; 0.45	0.1	
	do	(T) 0.45; 1.0[2]	0.05	
Top of Pier E-7	Dec. 15, 1934	(L) 2.7; 0.35	0.3	
	do	(T) 2.7[7]; 0.65; 0.35; 0.11	0.1	
Top of Pier E-9	do	(L) 3[2]; 0.5	0.3	
	do	(T) 2.0; 0.9[7]; 0.55	0.2	
Top of Pier E-10	do	(L) 2.3; 0.9[7]; 0.4[7]	0.7	
	do	(T) 1.9[2]; 0.8[7]	0.1	
Bottom of Pier E-17	Sept. 14, 1934	(L) 2.1; 0.4 to 0.5	0.2	
	do	(T)		
Top of Pier E-17	do	(L) 2.5; 0.50; 0.22	0.7	
	do	(T) 3.0[2]; 0.4?; 0.22	0.5	

GOLDEN GATE BRIDGE

Location	Date	Average values of periods	Displacement[1]	Remarks
North (Marin) Tower, first strut above roadway.	Sept. 11, 1934	(L) 4.3	35	Wind about 30 miles hour almost parallel tower. Top strut saddles not on.
	do	(T) 4.0[2]; 1.79; 0.30	8	Do.
North Tower, second strut from top.	Jan. 15, 1935	(L) 4.40; 1.8	35	Top strut and saddles place. Wind about miles per hour.
	do	(T) 1.81	4	Do.
North Tower Pier	do	(L) 4.4	0.1	
	do	(T) 4.4[2]	(6)	
South (San Francisco) Pier	Jan. 14, 1935	(L) 0.22 to 0.30; 0.05[2]	0.01	Top pier being polished tower placement.
	do	(T) 0.25	0.005	Do.
San Francisco Pylon No. 1	do	(L) 0.63	0.3	
	do	(T) 0.35	0.1	

[1] Maximum displacement computed from the position of rest.
[2] An approximate value of period.
[3] 1-inch displacement estimated from visual test. See text.
[4] Probably a record of the longitudinal component.
[5] Belongs to 3.55-second period. Probably corresponds to larger amplitude of L.
[6] Less than 0.1.
[7] Existence of period questionable.

Measurements of the bridges in stage (1) of the foregoing plan of procedure have practically been completed. Three of the piers measured are on land. The instruments used on Pier E–1 were not sensitive enough to pick up much motion, although low amplitude, 0.16-second period waves may be observed on one of the components. Two walkway cables in a slack condition were attached from Pier W–1 to the San Francisco anchorage and the rocker arms for the cables were in place at the time measurements were made. It is doubtful whether the attached cables seriously affected the vibrations of the pier, as the vibrations were too regular and the periods were too short. Besides, the expected difference in period across and lengthwise to the bridge was observed. The vibrations of the San Francisco Pylon No. 1 are likewise regular.

Records obtained from all piers in water, except the San Francisco Pier of the Golden Gate Bridge and perhaps Pier W–4 of the San Francisco-Oakland Bay Bridge, shown in figure 43, show a very irregular motion. There is no doubt that this is caused by wave action of the water. An underlying irregular wave having a period from 2 to 4 or even 5 seconds appears on all these records including that of Pier W–4. Oncoming waves have been observed to have this range in period. Amplitudes of vibration due to this motion are usually much greater than those of the natural vibrations of the piers alone. It is natural to expect that waves striking against the pier will cause it to rock on its foundation, but the magnitude of this rocking is ordinarily not much more than a thousandth of an inch. The fact that the East Bay Piers 7, 9, 10, and 17 are on piling probably serves to further complicate their motion.

Owing to the great mass and height of Pier W–4, its motion is not influenced much by ordinary wave action. It has a definite vibration in what is probably its natural period, even though underlying motion is produced by waves.

The San Francisco Pier of the Golden Gate Bridge is surrounded by dead water enclosed by a fender. It is a massive structure and the amplitude of its vibrations before the tower was erected was very small.

A visual test of the period was made near the top of the tower on Pier W–2. The period in each direction was observed to be 3.00 seconds. In the direction across the bridge a shorter vibration, period unknown, was superposed on the longer. Vibration of the tower is evidently transmitted to the pier, as a definite 3.0-second period has been observed there also.

A maximum single trace amplitude of about 12 inches was observed visually but not photographed at the top of the tower on Pier W–3. The static magnification of the instrument was 120 and its period was 1.2 seconds. Since the ratio of the period of vibration of the tower and the period of the instrument was about 3, the actual magnification was about 12. Therefore, the single-trace amplitude as observed visually represented a movement of the tower of about 1 inch from the position of rest, or 2 inches for the total movement. The wind velocity was observed to be 25 to 30 miles per hour.

Vibrations of the Marin Tower in both directions were recorded simultaneously on the strut about 100 feet from the top of the tower. Investigations of the vibrations of the piers and towers are not yet complete.

DAM VIBRATIONS

Forced vibrations of the Searsville Storage Dam near Palo Alto and the Morris Dam near Pasadena are discussed in another chapter.

A measurement of the natural vibrations of blocks 4 to 10 of the Morris Dam (fig. 41) was made with a Wood-Anderson seismometer in June 1934. With a magnification of 1,400 the trace amplitude was less than 0.1 mm in a moderate westerly wind. The level of the impounded water was about 110 feet below the roadway at the gatehouse. The period of blocks 4, 5, 7, 9, and 10 was very near 0.16 second; on blocks 6 and 8 it was about 0.17 second. Even though the amplitudes were very small, the waves on most records were quite distinct and their period uniform.

In August 1934 a record of a blast about 3 miles upstream was made on the top of block 5 (see fig. 70). The coda of this blast record contains waves having periods ranging from 0.14 to 0.18 second, the

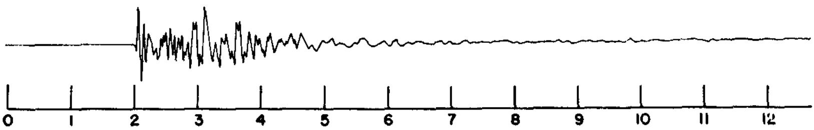


FIGURE 70.—Record of blast at the Morris Dam.

dominant periods being 0.16 to 0.17 second, with shorter-period waves superposed on them.

According to Mr. S. B. Morris, the computed period of the dam, assuming the blocks to be vibrating cantilevers on an infinitely rigid foundation, is about 0.14 second. It appears, then, that we are justified if we assign the 0.16- to 0.17-second period waves to the natural vibrations of the dam, bearing in mind that the yielding of the underlying rock, which in actuality is not infinitely rigid, tends to increase the effective heights and hence the periods of the blocks.

The dam has been built over a fault which has been inactive since the oldest gravel of the stream bed overlying the fault was deposited. Special provision for this fault has been made in the dam, including an open joint about 3 feet across, along which motion can take place without serious injury to the structure.

GROUND VIBRATIONS

The following reproductions of reports of pavement and ground vibration tests in San Francisco are self-explanatory.

The results of the explosion tests alone were not very conclusive, as the motion resulting from the blasts was damped out rather rapidly. However, after the force of the first impact had died down, a few waves having periods of about a tenth of a second or slightly less could be detected. Waves having this period appear also on records of vibrations caused by traffic on the streets below. Furthermore, forced vibrations induced by the microvibrator seemed to have a tendency to approach a resonance point at 0.1 second, which was the lower limit of the vibrator period. These combined tests point to a ground period of slightly less than 0.1 second. The dominant period of traffic vibrations at the mint site recorded by the Wood-Anderson seismometer, however, is about 0.055 second. The frequencies in the range of ground vibrations hitherto observed in San Francisco are

Preliminary Report

DEPARTMENT OF COMMERCE
U. S. COAST AND GEODETIC SURVEY
CALIFORNIA SEISMOLOGICAL PROGRAM

PAVEMENT VIBRATION TESTS

At the request of the Telephone Company, pavement vibration measurements were made at the following places in San Francisco:

Stockton & Geary Sts.,
Post & Grant Sts.,
Post & Stockton Sts.,
Post & Powell Sts.,
Sutter & Stockton Sts.,
Stockton St. Tunnel, south end.

The purpose of these tests was to determine, if possible, the vibration period and amplitude caused by street traffic at these points. The Telephone Company had had several telephone cable breaks along Stockton St. in the vicinity of Post St. It was hoped that the observations would show either a different period or a relatively larger amplitude near Post and Stockton Streets to account for these breaks. With the present instrumental equipment there would always be some question as to whether true absolute periods are obtained or whether the recorded periods are affected by some resonance from the violin string effect of the suspension. It is fairly safe to state that there are two different frequencies present, the slower of the order of about 12 cycles and the faster of the order of about 40 cycles per second. Because of the very small amplitude in all cases, a comparison of amplitudes would be of little or no value. An instrument similar to the Benioff type of electromagnetic seismometer using very high magnification and eliminating the violin string effect should show slightly different periods at the same location and this might be accounted for by different water content of the ground on these two dates.

Table 16. SUMMARY OF PERIODS OBSERVED IN PAVEMENT VIBRATION TESTS.

Test No.	Location and Date	Component	Frequencies (cycles/sec.)
8	Corner of Stockton & Sutter 2/14/35	Stockton Sutter	12 to 14; 37 to 40 12 to 14; 37 to 40
9	Corner of Stockton & Post 2/14/35	Stockton Post	11 to 12; 37 to 40 13 to 15;
10	Corner of Stockton & Geary 3/19/35	Stockton Geary	14 to 15; 40 to 50 14 to 15; 40 to 50
11	Corner of Powell & Post 3/19/35	Powell Post	15 to 18; 40 to 50 15 ± ; 35 ±
12	Corner of Grant & Post 3/19/35	Grant Post	12 to 14; 13 to 15;
13	S. end of Stockton St. Tunnel 4/4/35	Stockton Sutter	10? 10 to 11;
14	Corner of Stockton & Sutter 4/4/35	Stockton Sutter	10 to 11; 10 to 11;

In the first two tests of the earlier observations, a single-component Survey vibration meter was used. Unfortunately, violin string vibration induced in the filament by traffic was so bad as to render the records useless. A three-component accelerometer, $T_0 = 0.1$ sec., was used in tests 3 to 7. Regular trains of waves having a frequency of 60 cycles per second appeared on these records in such extent as to cover up many waves having other frequencies. It was later found that vibrations of this frequency could be induced in the accelerometer springs. It appears, therefore, that these waves are instrumental in origin and their measurements are not included in this report.

In tests Nos. 9 to 14 inclusive, the two-component Survey vibration meters were used. The violin string vibration of the filament was carefully eliminated so that the records give a picture of the actual ground motions. The frequencies measured on these records are therefore more reliable than those of the previous tests and should be substituted for those previously reported.

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GROUND VIBRATION TESTS

At the request of the Procurement Division of the Treasury Department, observations were made March 18, 1935, to determine the period of ground vibration at the site of the new Mint in San Francisco. This site is the block bounded by Hermann, Webster, Buchanan, and Duboce Sts. This site consists of a rocky hill with steep cliffs on Duboce, Buchanan and the easterly end of the Hermann Street side. The hill rises to a height of nearly 100 feet above the street level at the corner of Buchanan and Duboce Streets. The top of the hill is rather flat. The rock of the hill consists of serpentine and cherts of the Franciscan series. Near the surface, the rock-especially the serpentine-is badly weathered. The steeper cliffs are of chert, and serpentine seems to underly the gentler slopes. According to the map prepared by Mr. H. O. Wood, the intensity of the 1906 earthquake at the site was relatively weak. Because of the solid character of the formation, it was thought advisable to set off some small blasts to produce vibrations in the rock. The hill is partially covered by sand ranging in depth from less than an inch to a few feet which acted as a blanket for the explosions. The explosions were under the direction of Mr. Wm. F. McCandlish of the Hercules Powder Company. Fig. 71 shows the distribution of shots and instruments.

Hole 1 was 40 in. deep. The charge consisted of one and one half 1 in. by 8 in. sticks of 40 percent gelatin which was detonated by a No. 6 6 ft. electric blasting cap. Hole 2 was 7½ ft. deep. 10 sticks of dynamite and a No. 6 12 ft. blasting cap were used. Hole 3 was 9 ft. deep. 17 sticks of dynamite and a No. 6 12 ft. cap were used. Hole 4 was 10 ft. deep. 13 sticks of dynamite and a No. 6 12 ft. cap were used.

Position A is at the point of the hill just above the steep cliff overlooking the corner of Buchanan and Duboce Streets. The instruments were placed directly upon the chert. It is about 10 feet lower than the shot points. Position B is a level part of the masonry foundation of an old reservoir. This foundation presumably lies on rock, although this is not definitely known. Position B has about the same elevation as the shot points. Position C is on a sandy slope about twenty feet lower than the shot points. The instrument was set up on sand which probably overlies serpentine. Position D is on the side walk about 60 feet lower than the shot points.

TABLE 17. PERIODS OBSERVED IN GROUND VIBRATION TESTS AT MINT SITE

Blast no.	Dist. from blast to inst.	Inst.	Periods	Amp. of first ground motion	No.	Dist. from blast to inst.	Inst.	Periods	Amp. of first ground motion
	ft.		sec.	mm.		ft.		sec.	mm.
		Position A					Position B		
1.	160	SVM-2*	Record failed		1.	165	WA	Record failed	
2.	185	" "	0.06-0.07	.011E*	2.	160	"	"	"
3.	185	WA*	0.06-0.07 0.10-0.12	 .016E	3.	146	SVM-2	0.08-0.09	.012N* .007E
4.	183	"	0.05-0.07 0.09-0.11	 .010E	4.	155	" "	0.08-0.09	.010N .006E
		Position C					Position D		
1.	157	SVM-1*	0.07?	?	1.				
2.	125	" "	0.08?	?	2.				
3.					3.	180	SVM-1	0.04-0.05 0.08-0.10	 .004N
4.					4.	192	" "	0.04-0.05 0.08-0.09	 .006N

*Abbreviations used above: SVM-2, Survey vibration meter (2-component). SVM-1, Survey vibration meter (1-component). WA, Wood-Anderson seismometer. E, east. N, north.

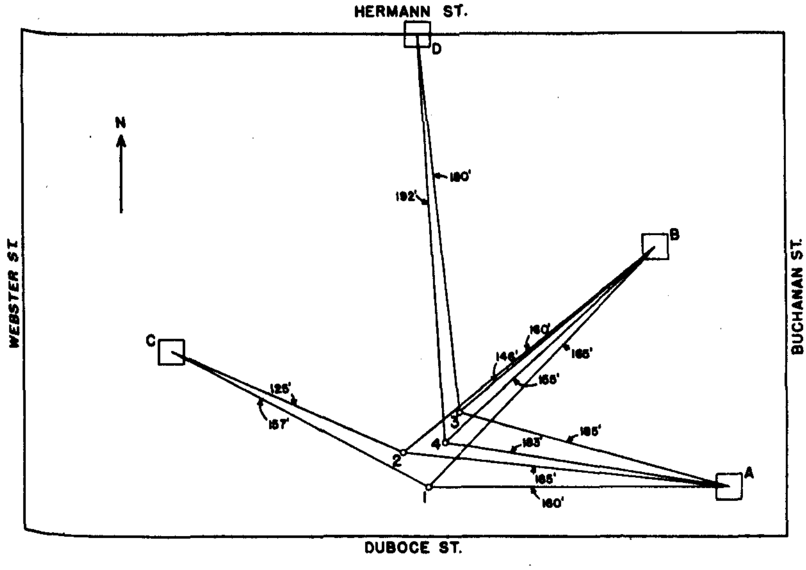


FIG. 71

NOTE:
1, 2, 3 and 4 denote the positions of the blasts (all about the same level at the top of the hill.
A, B, C and D show the positions of the instruments.

also in the range of bowstring frequencies of the recording instruments, consequently our readings remain in doubt until we can check these measurements with instruments which operate on another principle.

The recordings of ground vibrations in the South have been more successful. These are summarized in the following table:

TABLE 18.—*Ground-vibration observations*

[Components: (a) East-west; (b) north-south]

Location	Date	Average values of periods	Displacement [1]	Remarks
		Seconds	*0.001 inch*	
Long Beach, near Sovereign Apartments.	Sept. 5, 1934	(a) 0.32; 0.2 [2]; 0.10; 0.03	0.01	Heavy surf striking on sea wall.
	do	(b) 0.32; 0.20; 0.1; 0.03	0.01	Do.
Long Beach, rear of Ocean Center Building.	do	(a)		Do.
	do	(b) 0.32; 0.18; 0.1	0.04	Do.
Long Beach, near Villa Riviera.	do	(a) 0.30; 0.20; 0.13	0.04	Do.
	do	(b) 0.30; 0.18; 0.13; 0.09	0.04	Do.
Long Beach, sidewalk, Procter & Gamble.	Oct. 20, 1934	(a) 0.32	0.04	Normal vibrations.
	do	(b) 0.33	0.04	
Long Beach Junior College	Sept. 26, 1934	(a) 0.32	0.01	50 feet from building.
	do	(b) 0.32	0.01	Ground floor. Normal vibrations.
Hollywood, near Hollywood Storage Co.	Sept. 28, 1934	(a) 1.9 [2]; 0.3 [3]; 0.2	0.01	150 feet from building.
	do	(b) 0.30	0.01	Normal vibrations.
Long Beach, near Belmont fire station.	Oct. 10, 1934	(a) 0.33; 0.1 [3]	0.07	Do.
	do	(b) 0.32; 0.05	0.07	
Long Beach, sea wall near Villa Riviera.	Nov. 15, 1934	(a) 0.32; 0.2!; 0.03	0.02	Surf very light.
	do	(b) 0.32; 0.15 [3]; 0.02	0.01	Do.
Long Beach, 2926 Cedar Street.	Nov. 22, 1934	(a) 0.32 [3]; 0.16	0.04	Normal vibrations.
	do	(b) 0.32; 0.16	0.04	
El Centro, Southern Sierras Power Co., terminal station meter house.	Feb. 19, 1935	(a) 0.25; 0.07	0.02	0.07 is the period of a 5,000-kva synchronous condenser in main building 200 feet away.[4]
	do	(b) 0.37; 0.07 [3]	0.07	

[1] Maximum displacement measured from position of rest.
[2] Existence of period questionable.
[3] Approximate value of period.
[4] Observations were made in small meter house with concrete floor resting on ground.

It appears that 0.32 second is the dominant period of the ground underlying Long Beach, although it is possible that these vibrations are microseisms induced from some external source. Shorter periods of 0.2 and 0.1 second are not so definite. The ground near Ocean Boulevard consists of 30 or 40 feet of adobe overlying white sand. Alluvial silt underlies the region near 2926 Cedar Street. Incidentally, according to Köhler,[1] the natural period of the upper subsurface layers under Göttingen, Germany, as determined by independent methods, is about 0.32 second.

The recorded periods of nearly all of the lower buildings of Long Beach listed in table 3 are observed to be in the neighborhood of 0.32

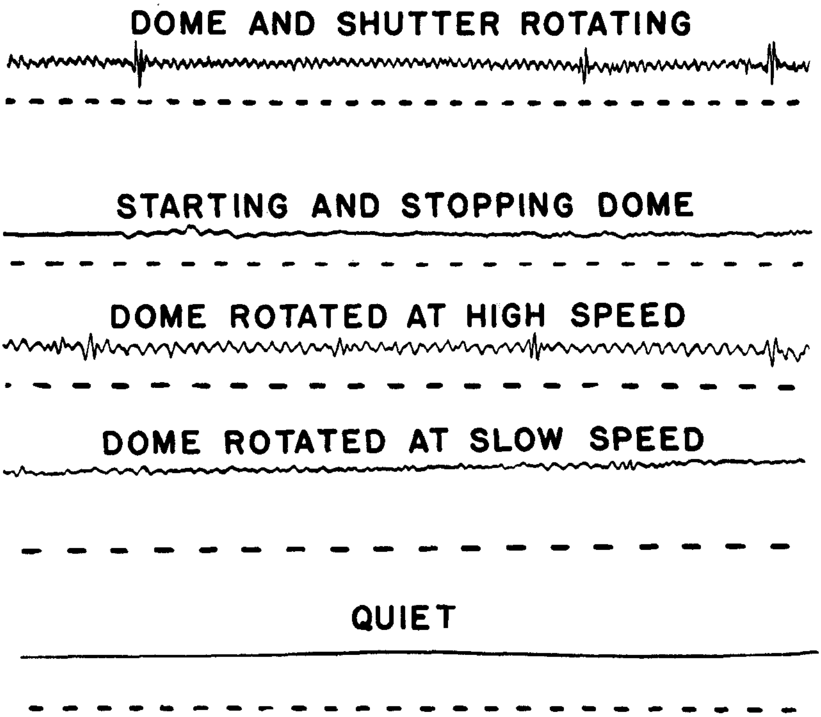


FIGURE 72.—Mount Wilson telescope pier vibration tests.

second. This period may also be assigned to secondary waves on the vibrograms of some of the higher buildings. Furthermore, it has been observed that the amplitudes of the vibrations on the ground of a few of these buildings, for instance the Belmont fire station, are not appreciably different from the amplitudes recorded on an upper floor. The reader may form his own conclusions as to whether these periods belong to the building concerned or whether the building is merely moving along with the vibrating ground.

[1] Dr. Reinhard Köhler, Eigenschwingungen des Untergrundes, ihre Anregung und ihre Seismische Bedeutung, Nachrichten von der Gesellschaft der Wissenschaften zu Göttingen, Mathematisch-Physicalische Klasse, Fachgruppe II, Neue Folge, Band 1, Nr. 2, 1934.

SPECIAL VIBRATION MEASUREMENTS

Vibrations of the concrete slab roof of the Chevy Chase Reservoir, Glendale, caused by the movement of a tractor and grader which were at work covering the slab with 2 feet of earth, were measured with a strong-motion accelerograph in June 1934.

The dominant period of the waves appearing on all three components of the record, which may also be the fundamental period of the slab, is 0.08 second. Other waves, which probably represent secondary vibrations of the slab, have periods of 0.06 and 0.04 second.

The maximum single amplitude of the vertical vibrations is computed to be very nearly 0.001 inch. The amplitudes in the horizontal directions are about half this. It is to be noted that the instrumental period, 0.1 second, is greater than the periods of the forcing motion, so that the term "accelerograph" in this case is a misnomer. The observer noted that these vibrations felt like a very slight earthquake, even though their amplitudes were small. This is quite characteristic of high frequency vibrations. The maximum acceleration was computed from the period and amplitude to be 1.5 percent of gravity.

Special vibration tests at the Mount Wilson Observatory, near Pasadena, were made in January 1935. The purpose of the measurements was to determine how the vibrations produced by the dome of the 100-inch telescope rolling on its track were transmitted from the building footings to the telescope pier, and to determine the order of movement and tilt of the telescope pier while the dome was being operated. Observations were first made on the concrete immediately beside the casting at the north end of the telescope, and then on the telescope yoke at the south end and on the concrete pier immediately underneath. Tests were also made at ground level on the telescope pier and on the building foundations. The structure is built upon rock. The results are shown in figure 72 and table 19.

TABLE 19.—*Observations on Mount Wilson Observatory 100-inch telescope pier*

[Tests made Jan. 11, 1935]

Test no.	Instrument position	Mean values of periods	Displacement [1]	Remarks
		Seconds	*0.001 inch*	
A 1	On concrete beside casting on north end of telescope.	0.28	0.02	Person running down stairs.
	do	0.33		Person running up stairs.
A 2	do	0.17	0.01	Tilt test with dome rotating at high speed. Evidently no tilt.
A 3	do	0.6 [2]; 0.27 [3]; 0.10 [3]	0.01	Starting and stopping dome.
A 4	do	0.17; 0.1 [3]	0.01	Dome rotating at high speed.
A 5	do	0.17; 0.10 [3]	0.01	Dome rotating at low speed.
A 6	do	0.18 [3]; 7 [2]	0.01	Everything quiet.
B 1	do	0.29	0.07	Person running down stairs.
	do	0.28		Person walking up stairs.
B 2	do	0.15; 0.07	0.03	Dome slowly rotating ⅛ turn.
B 3	do	0.29; 0.15; 0.1	0.04	Dome rapidly rotating ¼ turn.
B 4	South end of yoke	0.28; 0.15; 0.1	0.04	Dome slowly rotating ⅛ turn.
B 5	do	0.28; 0.17; 0.1	0.06	Dome rapidly rotating ¼ turn.
C 1	Column at north window	0.18; 0.09	0.01	East-west motion. Dome rotating rapidly. Considerable tilt shown.
	do	0.17	0.01	North-south motion. Dome rotating rapidly. Considerable tilt shown.
C 2	Telescope pier at ground level.	0.17; 0.09	0.01	East-west motion. Dome rotating rapidly.
	do	0.17; 0.10	0.01	North-south motion. Dome rotating rapidly.

[1] Maximum displacement measured from position of rest.
[2] Existence of period questionable.
[3] Approximate period.

ACKNOWLEDGMENTS

Prof. Lydik S. Jacobsen has acted in an advisory capacity throug out the preparation of this report. The writer wishes to further a knowledge his indebtedness to the seismologists, engineers and arch tects who have given helpful advice and criticism during this invest gation; to the managers, owners, or engineers of the buildings or e tablishments he has visited, for allowing him access to their premise to the engineers and architects who have given him information nece sary to complete his reports; and to certain manufacturing concer and all others who have cooperated with him throughout this invest gation.

Chapter 6.—VIBRATION STUDIES

R. S. McLean and W. W. Moore

INTRODUCTION

Vibration measurements may be made quite simply in very little time in ordinary structures, but considerable study is sometimes necessary to be certain that the vibrograms are correctly analyzed. It is necessary to consider those characteristics of the structure which would be expected to affect its vibrations, in addition to the features which appear on the gram. Motions recorded on the gram may be

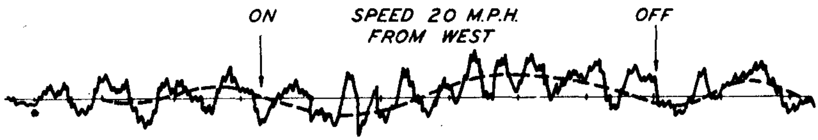


Figure 73.—Vibrogram obtained on Colorado Street Bridge in Pasadena.

misinterpreted unless the physical features causing such action are taken into account. Three illustrative cases will be discussed.

Portions of tests made on the Colorado Street Bridge in Pasadena (fig. 73) are cited as the first example. The instrument used is quite sensitive to tilt, and the long waves indicated by the dotted lines are due to moving loads. Their period is dependent on the speed of the load, and independent of the natural period of vibration of the bridge.

Actual vibrograms of buildings or other structures are seldom smooth curves with constant amplitude and period. In addition to

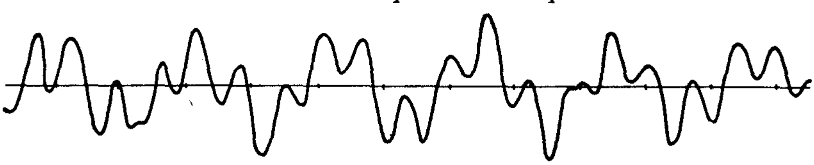


Figure 74.—Vibrogram. Los Angeles City Hall.

the slower regular movement of the building in its fundamental period, harmonics are frequently present. External forces also cause variations in period and amplitude that greatly complicate the record. It then becomes the task of the observer to identify the various frequencies of vibration and to allow for irregularities caused by external forces. Figure 74 is an example of such a record.

A number of graphs of the resultant motion obtained by combining two simple harmonic vibrations are shown in figure 75. Except for one case, these are for ideal cases in which the amplitude of each component of the motion is constant.

Two curves of constant amplitude have been combined in figure 76 to form a graph similar to figure 74, which is a portion of a vibrogram made in the east-west direction on the ninth floor of the Los Angeles City Hall.

Vibrograms must also be examined with great care because of the errors introduced by improper orientation of the vane of the vibration meter. The seismometer used for much of the work done by subparty no. 5 was originally equipped for double reflection to gain greater magnification, and the mirror was set at a small angle to the plane of the vane. In adapting this instrument to building vibration

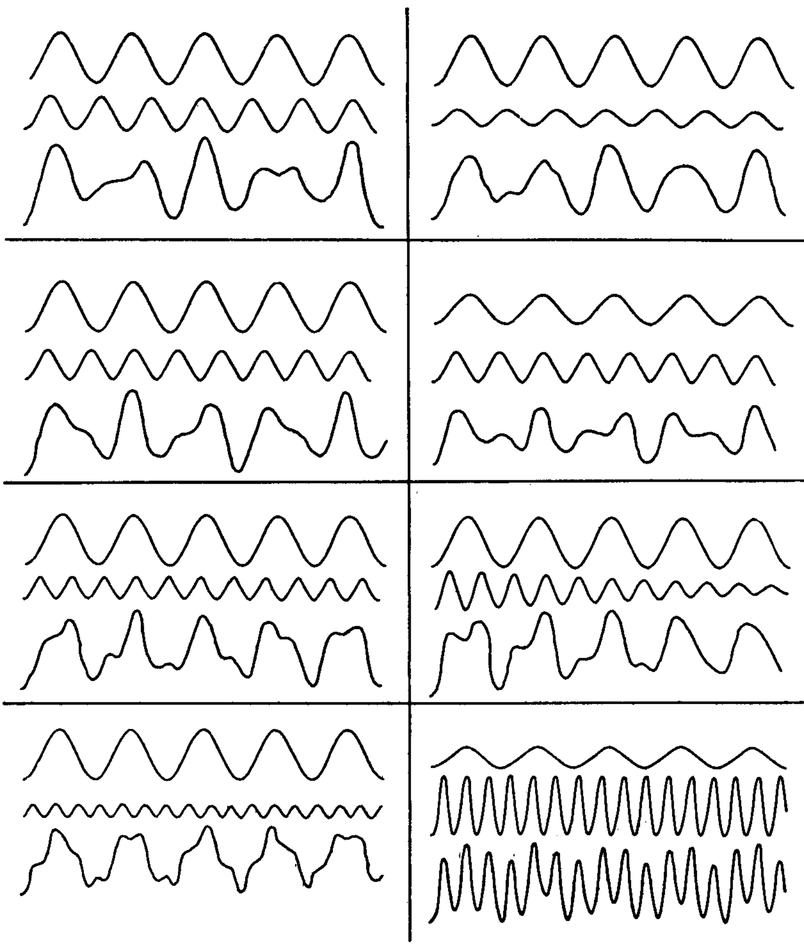


FIGURE 75.—Combinations of simple harmonic motions. The bottom curve is in each case the resultant of the two above it, illustrating the complexity of wave types that can be obtained from as few as two sine curves.

work using single reflection, the vane was turned out of correct orientation by an angle which varied up to perhaps 15°.

The vibrograms of building A, which is rectangular in plan and four times as long as it is wide, illustrate this point (see fig. 77). Assuming that the angular error of orientation is 10° the longitudinal east-west record will include a component of the transverse north-south motion whose amplitude will be two-tenths of the latter. It was assumed from the vibrogram that the transverse period was 1.23

seconds and the longitudinal period was 0.49 second. Waves of the shorter period and of three different amplitudes were combined with the component of the north-south motion. The resulting curves are quite similar to the east-west vibrogram, and indicate that

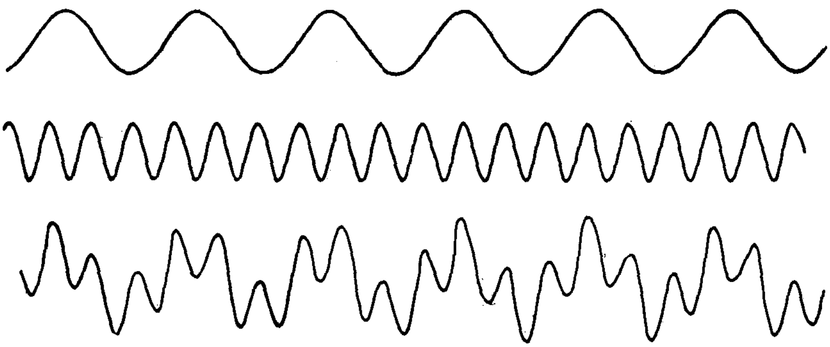


FIGURE 76.—Vibrogram analysis. The resultant of these two sinusoidal waves is similar to the vibrogram shown in figure 74.

although periods of both 1.23 seconds and 0.49 second appeared on the gram, the 0.49-second period is correct for the east-west direction. When considerations of this kind are not available the analysis of the recorded motion may in simple cases be accomplished by inspection and in more difficult cases, if the motion is periodic, by Fourier analysis; otherwise more complicated methods such as that of periodogram analysis must be resorted to. Machines exist which are capable of facilitating the work in extremely complicated cases.

It was found in the case of building B that vibrations of approximately 1-second period appeared on the vibrograms made in both

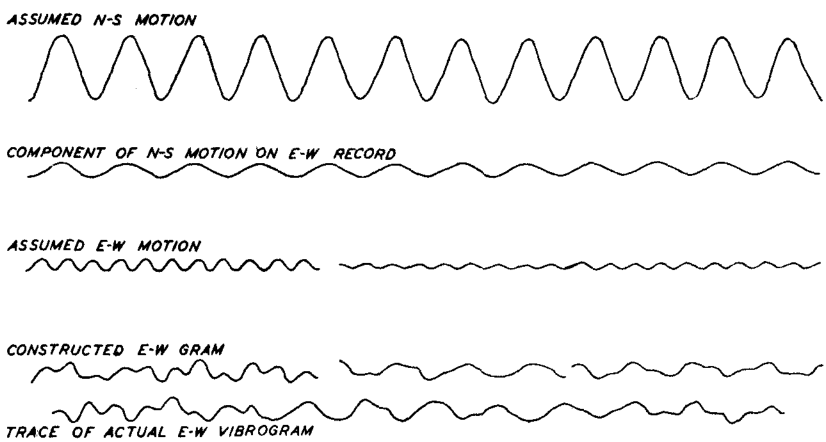


FIGURE 77.—Effect of error in orientation.

directions. As the building is more than three times as long as it is wide, it seemed improbable, just as with building A, that the period should be the same in both directions. Records were made simultaneously with instruments at both ends of the building, portions of

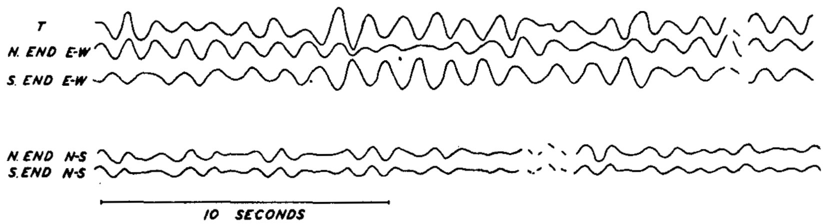


FIGURE 78.—Torsion study in a rectangular-shaped building. The north-south motion is very similar for simultaneous observations made at the two ends of the building. The east-west motions are dissimilar, and their difference, shown in the top curve, measures the torsion of the building.

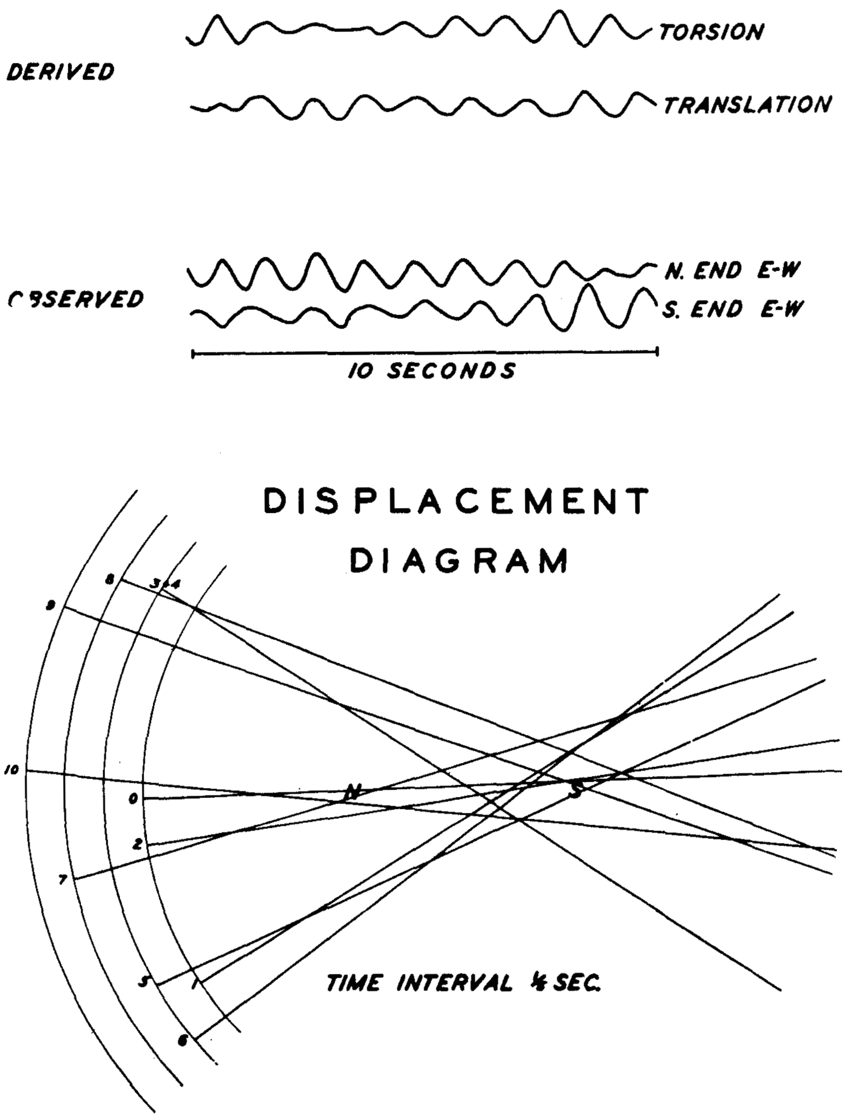


FIGURE 79.—Torsion study, Pickett method.

which are reproduced in figure 78. The curve marked "T" was obtained by plotting the differences of the ordinates recorded simultaneously at the ends of the building. It gives the relative transverse movement of one end with respect to the other and indicates the presence of torsional vibrations.

A somewhat different method of determining torsional vibrations is that used by Mr. G. H. Pickett, illustrated in figure 79. Successive

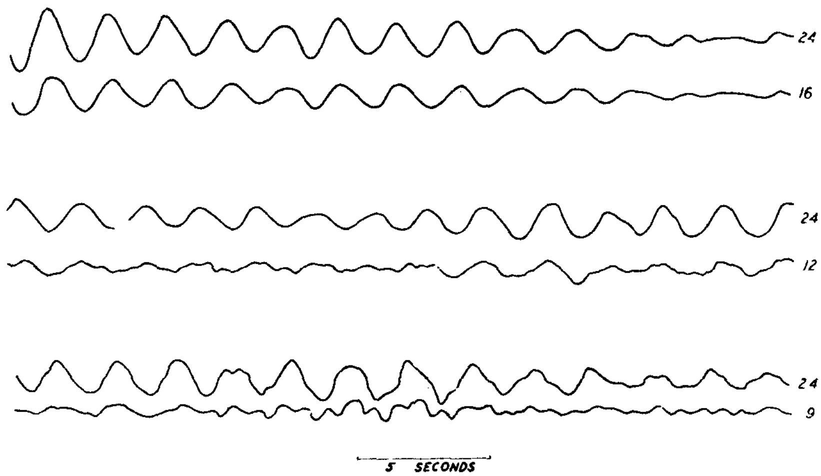


FIGURE 80.—Los Angeles City Hall, simultaneous measurements. The numbers refer to the floors on which the vibrograms were obtained.

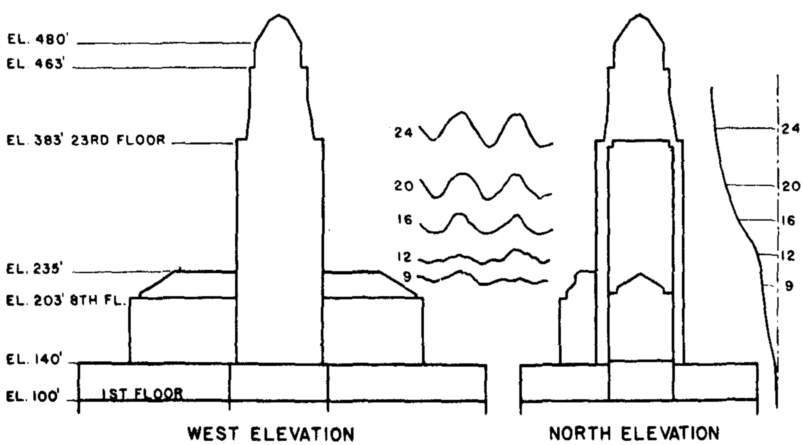


FIGURE 81.—Los Angeles City Hall. Elevations, vibrograms, and deflection curve.

positions of each end from an assumed longitudinal base line were plotted to a distorted scale from a 10-second portion of the transverse records. Longitudinal movements were not plotted, since they were the same for each end. From these diagrams it is possible to distinguish torsional and translational movements.

Simultaneous recording has been used in other buildings with valuable results. Portions of records made in the east-west direction in pairs on the twenty-fourth and sixteenth, twenty-fourth and twelfth, and twenty-fourth and ninth floors of the Los Angeles City Hal are reproduced in figure 80. The complete records show conclusively that the lower floors move in the same fundamental period as the twenty-fourth, a fact which is to be expected, but which could not be determined from individual records at different times on the various floors, because of changes of external disturbing forces.

Figure 81 shows sketch elevations of the building and a comparison of the motion of five floors. The portions of the several runs plotted were selected because the motion on the twenty-fourth floor was the same in period and amplitude. From this group the deflection diagram has been constructed. While it is based on the displacement of floors at various times, it is believed to be nearly the same as the maximum deflection at any one time.

TANK PERIOD COMPUTATIONS

The method proposed for computing the period of vibration of a tower is based upon the relation of period to the deflection of the tower under a horizontal load equal to its own weight. The mathematical formula necessary for computing the period of vibration can be derived quite simply. It is assumed that the deflection of the tower during oscillation is equivalent to that of a heavy mass supported on a weightless spring having one degree of freedom. As the weight of the tower is usually about one-twentieth of the water load, the latter assumption is believed to be sufficiently precise.

The period of oscillation is then given by the equation:

$$T=2\pi\sqrt{\frac{\delta_{st}}{g}}=1.1\sqrt{\delta_{st}}$$

The term δ_{st} is the deflection in feet of the top of the tower under a static horizontal force equal to its own weight and load. The following computations are based on this equation.

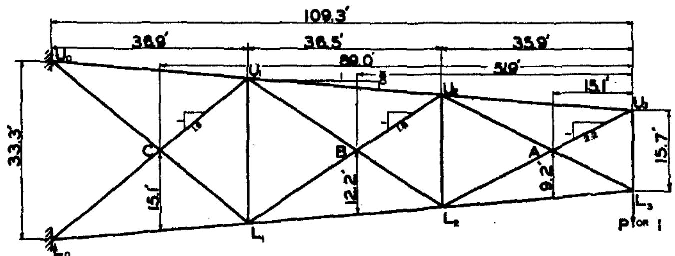
Since it is evident that tierods will buckle under compression, only those which are subjected to tension may be considered in designing a tower of this type to resist lateral forces. The measured period of the Willard Storage Battery Co. tower in Los Angeles was 1.45±.05 seconds, while the computed period was 1.93 seconds, considering one-half of the tierods as acting. The truss joints were assumed pin connected, and a more elaborate study considering the joints as rigid should lead to a somewhat shorter computed period. Probably the assumption that the foundation was rigid should result in an opposing and larger error. It is reasonable to believe that there will be some yielding of the foundation, the effect of which would be to lengthen the period of the tower. The computations were, therefore, thought to be in error as it was believed that the measured period should, if anything, be longer than the computed period.

Further consideration suggested that, since the tierods in a tower usually have some initial tension, they might be treated as acting in both tension and compression under the conditions of vibration measurements. That is, the deflection of the tower is very small under test conditions, and the resulting compression in a tierod might be so low as to fail to exceed the initial tension in the member.

In computing the deflection of a truss under load, the factor considered is the change in stress which a member undergoes as the load is applied. Until the compression caused by the applied load exceeds the initial tension in a tierod, the member must be considered as contributing to the rigidity of the structure. Some other factors dependent on tierod initial tension will be treated later.

Assuming all tierods to be acting, the computed period of the Willard tower is 1.5 seconds, which compares very well with the measured period of $1.45 \pm .05$ seconds. This tank is a standard 60,000-gallon tank, on a 3-panel steel tower, 92 feet to the platform.

A typical computation of period is shown in figure 82. Deflection is computed in the usual manner, using the Maxwell-Mohr theorem of dummy unit load. The period may then be found by use of the



$\Sigma M_A \quad -L_2L_3 = U_2U_3 = \frac{15.1}{18.4} = .82$

$\Sigma M_B \quad -L_1L_2 = U_1U_2 = \frac{51.9}{24.4} = 2.14$

$\Sigma M_C \quad -L_0L_1 = U_0U_1 = \frac{88.0}{30.2} = 2.94$

$\Sigma V_A \quad -L_2U_3 = U_2L_3 = (.5 - .081 \times .82) \times 2.2 = .95$

$\Sigma V_B \quad -L_1U_2 = U_1L_2 = (.5 - .081 \times 2.14) \times 1.8 = .59$

$\Sigma V_C \quad -L_0U_1 = U_0L_1 = (.5 - .081 \times 2.94) \times 1.6 = .41$

$T = 2\pi\sqrt{\frac{\delta_{st}}{g}} \qquad \delta_{st} = \Sigma \frac{SUL}{AE} \qquad S = U \times P$

$P = \frac{1}{2}(\text{WATER LOAD} + \text{WT. TANK} + \frac{33}{140}\text{WT. TOWER}) = 443$*

$\delta_{st} = \frac{140.7 \times 443}{30{,}000{,}000} \times 1000 = 2.08' \quad \text{OR} \quad T = 1.1\sqrt{2.08}$

= 1.58 SEC. (COMPUTED)

AND 1.56 TO 1.58 SEC. (MEASURED)

*TIMOSHENKO 'VIBRATION PROBLEMS IN ENGINEERING' PAGE 57

BAR	L	A	U	U^2L/A
L_2L_3	35.9	19.33	− .82	1.25
U_2U_3	"	"	+ .82	1.25
L_1L_2	36.5	"	−2.14	8.65
U_1U_2	"	"	+2.14	8.65
L_0L_1	36.9	20.27	−2.94	15.70
U_0U_1	"	"	+2.94	15.70
L_2U_3	38.2	1.27	− .95	27.15
U_2L_3	"	"	+ .95	27.15
L_1U_2	42.3	"	− .59	11.60
U_1L_2	"	"	+ .59	11.60
L_0U_1	45.4	"	− .41	6.00
U_0L_1	"	"	+ .41	6.00
L_2U_2			0	
L_1U_1			0	
			TOTAL	140.70

FIGURE 82.—Computation of tower period.

equation previously given. These figures are for a standard 100,000-gallon tank on a 3-panel, 100-foot steel tower owned by the port of Oakland.

The subparty in northern California has made a series of period observations corresponding to various amounts of water in the tank of the Pacific Manifolding Book Co. at Emeryville. This is a 50,000-gallon tank on a 3-panel steel tower 83 feet high, which has been reinforced for earthquake loads.

A curve showing the relation of weight of water to depth is shown in figure 83. There are also both computed and measured curves of period versus height of water. There is good agreement between these curves in view of the approximations used in the computation. In considering these curves a few facts should be remembered. For the computation, foundations were assumed rigid. A yielding of the foundation causes the measured period to be longer than the computed period by about 30 percent when the tank is empty. When water is in the hemispherical bottom of the tank, there is probably more movement of the water relative to the tank than when the tank is more nearly full. As the mass of the water does not all act, the computed period, which considers all the water as effective, becomes the longer at low depths of water. When the tank is full, a larger part of the mass of water is apparently effective,[1]

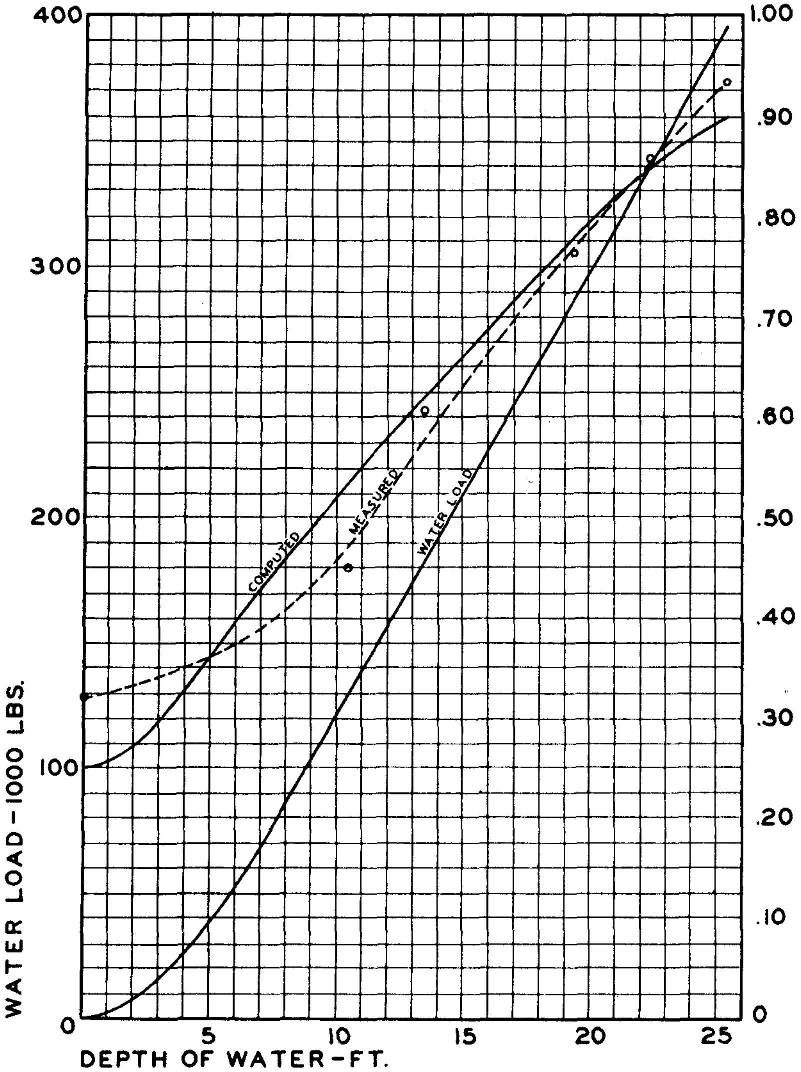


FIGURE 83.—Period and water load of 50,000-gallon steel tower tank.

and close agreement between the measured and computed periods results. This agreement has been verified by other, independent computations of period with tanks full. When the percentage of water acting becomes quite large, the measured period again becomes the longer of the two because of yielding of the foundation.

[1] From experiments at Stanford University, Professor Jacobsen estimates that about 70 percent of the water will be effective when the depth of the water is equal to the diameter of the tank. While the effects of movement of water and yielding of foundation are always opposed, it is interesting that in this case they are nearly equal.

As previously stated, in case the stress due to initial tension in a ierod is not exceeded by the stress due to a given lateral load, the ierod is not in actual compression. Therefore, the change in stress f that member must be considered in computing deflection. That s, the member is treated as a two-force member until its total tensional stress becomes negative. The same reasoning applies to the orce u due to the unit load as has been explained for the force S due o total load. It may now be seen that the initial tension in the tierod ust be known, or assumed, in order to compute the period of oscillation for any desired amplitude.

If the amplitude is such as to require a force that will throw a ierod into actual compression, that tierod is neglected in computing

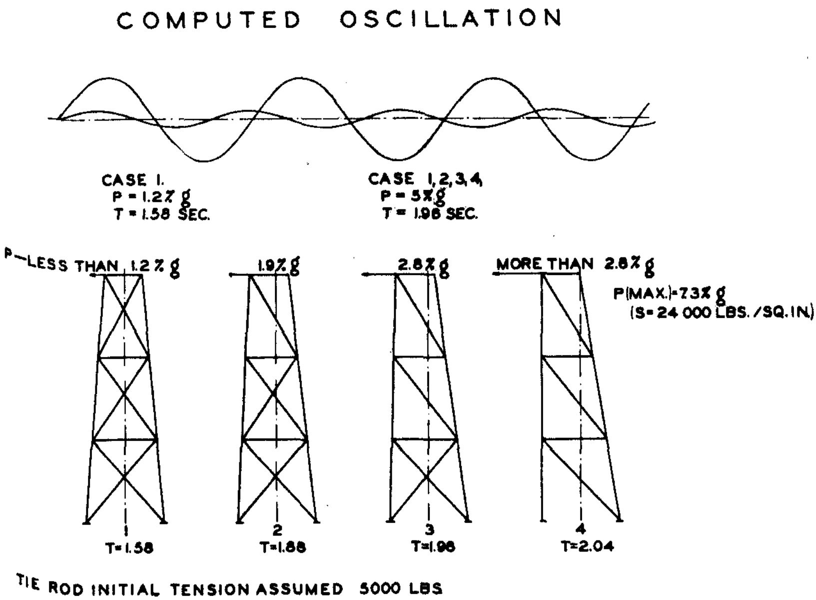


FIGURE 84.—Tierod compression and tower oscillation.

the deflection and period. If initial tension is equal in all tierods, it becomes evident that the tierods in different panels do not buckle at the same amplitude. The upper panel tierod goes out of action first, then the center, and last the lower one as the amplitude of motion increases progressively. The period for each of these separate cases is computed and shown in figure 84 along with the force necessary to cause the required deflection. During an actual cycle of oscillation the tower may go through all four of these phases, and as a result the period may be a composite of the four computed periods. This type of movement is represented for an assumed amplitude by the computed curve at the top of the figure. It should be noted that a changing rigidity, as in this case, results in a skew resonance curve. However, in view of the uncertainty as to action of water in the tank, and the rather small changes in period, a detailed consideration of this phenomenon was not undertaken.

It may be interesting to note that an acceleration of about 15 percent of gravity is necessary to cause the total load of the tower to rest on one pair of footings. This acceleration corresponds to an amplitude of oscillation of slightly less than 4 inches. Extrapolation of results of pull-back tests give an amplitude of 3.6 inches for a similar force.

The method presented appears to be reliable for the conditions of vibration measurements. The computations have shown that there is a possibility of movement of the water relative to the tank and of yielding of the foundation. The extent of these effects under earthquake conditions is not known, but they may be of importance. Seismograph records during actual earthquakes should aid in evaluating the effect of movement of water in the tank, lack of foundation rigidity, and computed variations of period with amplitude.

In conclusion the authors wish to acknowledge their indebtedness to Prof. R. R. Martel for his helpful assistance and criticism in the preparation of this paper.

Chapter 7.—THE BUILDING AND GROUND VIBRATOR

J. A. Blume

INTRODUCTION

L. S. Jacobsen

An experimental study of periods of vibration of buildings may be made by observing the free, transitory vibrations set up in the structures by impulsive external agencies, as, for instance, wind gusts. Under these conditions a building will vibrate in a number of its natural modes, the largest number being equal to three times the number of stories in the building. The time displacement curve obtained by the seismograph set up in the building will be far from simple in regard to the amplitudes of the motion, but it will contain only a finite number of periods. Sometimes it is possible to identify these with definite periods as, for instance, the fundamental torsional period, the third-mode transverse period, etc. As a rule, however, we may say that the wind-gust excitation of the building is quite indeterminate and will not be under the control of the observer.

In the field of mechanical engineering, a method of studying vibration properties of machine parts by the centrifugal force agitator has been employed successfully for years. During the last 10 years or so, the centrifugal force agitator has also been applied to structures such as bridges and buildings, and a great deal of information about their dynamic properties has been obtained by inferences drawn from the records of the resulting vibrations.

If such a centrifugal force agitator be at the disposal of the observer, he can excite the mode of vibration of the building that he wishes to study by placing the agitator in a definite relation to the nodal points of the distortion curve associated with the mode of vibration. He can repeat the test if necessary, and convince himself of the accuracy of the observations. The records are obtained with seismographs as in the case of the wind-gust experiments, but they are quite different from the former in appearance. In this case, the records show the forced vibration of the building and exhibit very prominently the period of the agitator or shaking machine. If the speed of rotation of the agitator be varied, the response of the building to this variation is recorded as an increased or as a decreased amplitude of motion, and it is easy to determine at what speed of rotation a maximum amplitude or response occurs. In other words, by letting the rotative speed of the machine decrease slowly, as in coasting from a given top speed, all the modes of vibration of the building excitable from the particular location of the vibrator will appear in the seismogram. If the higher modes are to be studied, the rotative speed must be high, and the unbalanced element producing the centrifugal force need not be very large. If the fundamental modes of the building must be excited, the rotative speed will be relatively slow and the unbalanced element will have to be correspondingly large in ord that sufficient periodic disturbance may be created. It so happe that the fundamental modes of a building are easily excited by wi gusts, but the higher modes are more difficult to obtain; with t vibrator the opposite is true, so that the two methods compleme each other well.

Not only is it possible to draw response curves from the recor obtained by excitation with the vibrator, but it is also possible fro these curves to identify and measure the periods of the building a be certain that no periods are overlooked. Moreover, the curv indicate, qualitatively at least, how much friction or damping present in the building and how the damping forces vary with t amplitude and period of the vibration. Another way of estimati the magnitude of the damping of the building is to compare deceler tion curves of the vibrator located in the building with those obtain when the vibrator is located on "solid" ground. If the vibrator driven at constant speed, the electrical power supplied to the machi may be compared with that for the "solid" ground location.

Considering that the time during which the vibrator has bee available for tests was only 6 or 7 months up to the date of this repor one is impressed with the number and diversity of problems that hav been dealt with. A cursory as well as a thorough study of the accu mulated data convinces one of the great usefulness and importanc of continuing with this work. From the beginning of the tests, th question of the size of the vibrator has been discussed, and it can b stated now that the results indicate that the vibrator is sufficient large for exciting the second and higher modes of vibration of mos Pacific coast buildings. If the work with the vibrator is to be ex tended to ground tests and if such massive structures as concre and earth-fill dams are to be tested, a larger machine will be neede This larger machine should be able to exert 3 to 4 times the dis turbing forces of the existing one, and it would be very desirab to have it designed for rotative speeds up to 900 revolutions pe minute. With such apparatus it should be possible to learn a grea deal about ground constants.

PRELIMINARY CONSIDERATIONS

During August, September, and October of 1934, a vibrator wa built at Stanford University. It had been decided that the mos logical, as well as economical, procedure would be to construct small trial machine in order to learn whether or not such vibrator would be of value in the California Seismological Program, and with this in mind, a preliminary design was made by the authors. The design was approved, and construction began on August 1. Professor Jacobsen was appointed adviser, and Mr. Blume observer, the latter to carry out the actual construction of the machine in the Stanford University machine shops, to use it in the field later, and to prepare reports on the tests made. This organization comprised subparty no. 8, the Building and Ground Shaker Party.

Previous work has been carried on with vibrating machines in Ger many, where they have been used extensively to test railway bridges and to compact filled ground. Dr. Reinhard Köhler, of the Geo physical Institute in Göttingen, has done considerable research by the forced-vibration method to obtain natural periods of vibration of the ground at Göttingen and to compute its damping coefficients. A few frame houses have been vibrated in Germany by means of a bicycle wheel to which was attached an unbalanced mass. Vibrating machines involving eccentric wheels are manufactured commercially by Lösenhausenwerk A. G., Düsseldorf-Grafenberg. It was impossible to buy one of these machines because under the N. I. R. A. appropriation all materials and articles purchased had to be produced and manufactured in the United States. Moreover, the cost and the length of time required for delivery of one of these machines were considerably in excess of our estimates.

DESIGN AND CONSTRUCTION OF THE VIBRATOR

In the design of the vibrator two factors have been kept in mind as being of great importance, namely, portability and adaptability. Inasmuch as it was not known what forces would be necessary to vibrate a large building or ground sufficiently to yield workable records, the machine was made as large as could be readily handled in a building by a few men. In order to simplify the evaluation of the test results, it is desirable that the centrifugal force reactions of the vibrator shall have effective components in one direction only. For some tests the forces exerted by the vibrator must be horizontal, and for other tests, as, for instance, ground or soil vibrations, the machine must produce variable forces in the vertical direction. It is, therefore, necessary to have two unbalanced masses or flywheels, rotating in opposite directions on two shafts geared together. If only one unbalanced flywheel were used, the centrifugal forces would be effective in all directions of the plane of the wheel, as is the case with the unbalanced bicycle wheel apparatus. Employing two shafts naturally increases the over-all dimensions of the machine considerably. However, it is felt that the apparatus as built is really portable. Its dimensions are: Height, 48 inches; length, 36 inches; and width, 20 inches. It weighs about 340 pounds, depending upon the amount of unbalance attached to the wheels. The ratio of the maximum reactive force that may be produced to the total weight of the machine is 10. It is believed that this ratio is much greater than that of the Lösenhausenwerk machines. The weight of the frame has been kept a minimum by using duralumin instead of steel. Experience has shown that four men can easily remove the machine from the truck, place it upon a dolly, and wheel it into the elevator of a building and thence into its test position within a few minutes.

Under the question of adaptability the range of periods and the corresponding forces have been considered. The apparatus was made flexible by providing a means of changing the amount of unbalance to suit local conditions. Provision has been made to vary the amount of unbalanced moment between 0 and 1,540 inch-pounds by changing the number of lead plates on the wheels of the vibrator. The fastest speed that has been attained is about 750 revolutions per minute, a period of 0.08 second. However, it is felt that for safety, a maximum speed of 600 revolutions per minute should be assigned. The slowest period that has been definitely excited in a building by the tests made thus far is 1.33 seconds; an unbalanced moment of 768 inch-pounds was used at the time. Building periods considerably longer than 1.33 seconds could probably be excited by using the maximum amount of unbalance.

Most of the actual construction of the vibrator was done in the Stanford University machine shops. Acknowledgment is due the mechanical engineering department of that university for gratuitous permission to use its equipment in this work.

DESCRIPTION OF THE VIBRATOR

The vibrator consists essentially of three ¼-inch steel-plate wheels, each 30 inches in diameter, to which are bolted lead plates in order to produce "unbalance." The forces developed by the machine are directly proportional to the amount of this unbalance as represented by the familiar equation for centrifugal force, $F=\frac{Wr}{g}\omega^2$, in which Wr equals the total unbalanced moment in inch-pounds; ω, the angular

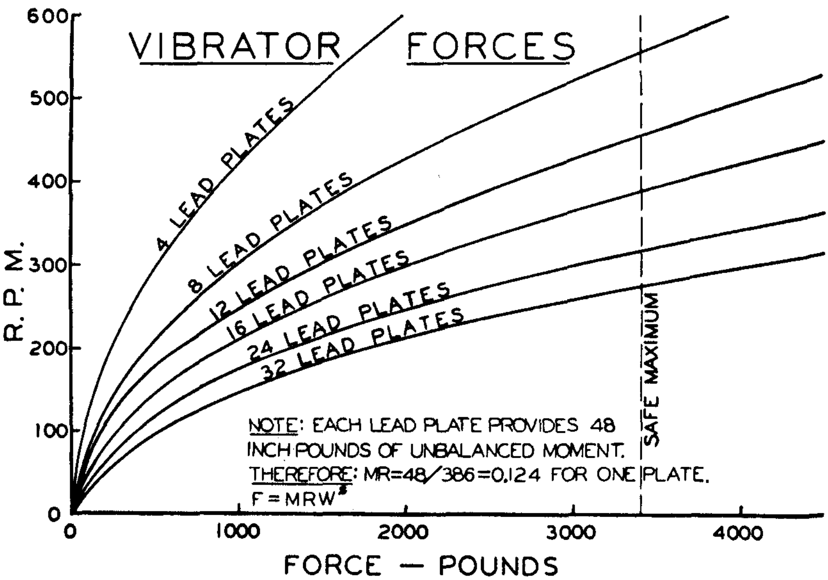


FIGURE 85.

velocity in radians per second; g, the acceleration of gravity in inches per second per second; and F, the centrifugal force in pounds. Figure 85 shows the forces developed at various speeds with the amount of unbalance plotted as the parameter.

Two of the wheels are attached to one shaft and the third wheel is attached to the other shaft. The two shafts are geared together by 16-inch cast-iron spur gears and consequently rotate in opposite directions. The same amount of unbalance is applied to each shaft. Figure 86 illustrates how the force is limited to one direction in the plane.

The machine may be driven either directly or through a 6:1 speed-reduction gear. In all of the tests made thus far the procedure has been to drive the machine directly by means of a direct-current motor and V-belt until the highest speed desired has been attained, and then to remove the belt, the record of the response of the structure being taken as the machine slows down. This procedure allows the speed of the machine to decrease very slowly since it eliminates the friction and windage losses of the motor and belt. Ball bearings are used throughout the machine. Ordinarily the vibrator takes about 5 minutes to come to a stop from a speed of about 500 revolutions per minute. Of course, this length of time varies with many factors such as the type and size of structure being vibrated, the amount of unbalance, and the method of anchoring the vibrator to the structure. This question will be considered in future tests in an attempt to obtain the energy input into the structures from deceleration curves, in order to evaluate the amount of damping.

It was decided to use a direct-current motor to drive the vibrator in order to obtain a flexible as well as delicate speed control. As

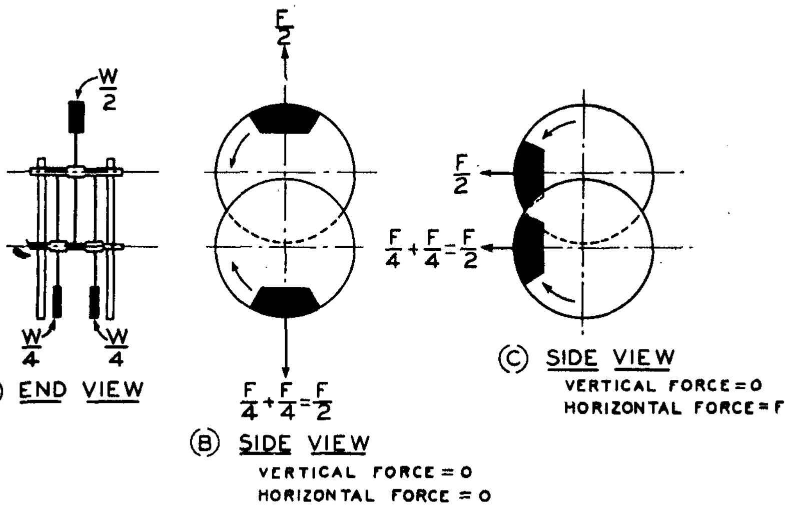


DIAGRAM OF WHEELS SHOWING UNBALANCE

FIGURE 86.

direct current is used very little in buildings today, it was necessary to provide a motor-generator set to convert alternating current to direct current. A 3-horsepower single-phase alternating-current motor and two direct-current motors were bought. These three units are wired so that rheostats control the fields of both the direct-current generator and the direct-current motor, thus providing a wide range of speed. The alternating-current motor can be driven from either a 110- or 220-volt circuit. As mentioned in the preceding paragraph, the test procedure to date has been to accelerate the vibrator to the speed desired and then remove the belt, taking the records of vibration as the machine decelerates. Obviously, this method does not demand a delicate speed control. However, at the time of the design, it was not known whether this method would provide time for establishing a sufficiently steady state to give distinct resonance phenomena. The direct-current apparatus will be necessary for tests involving sustained vibrations of constant periods.

METHOD OF MAKING TESTS

In buildings the vibrator is securely braced between two columns. Ordinarily a piece of 4- by 4-inch lumber is placed horizontally between the end of the vibrator and the adjoining column, and is screwed tightly into place by means of a large bolt attached to the frame of the machine (fig. 87). Ballast is usually placed on skids bolted to the base of the machine in order to steady the apparatus.

On structures that afford no columns or walls that can be braced against, it is necessary to resort to other methods of transmitting the disturbing forces. On the top of Searsville Dam, a specially built wooden frame was used which was held to the dam by bolts set into the concrete. On the Colorado Street Bridge, the vibrator was held in place mainly by piling a great many sacks of cement on the skids

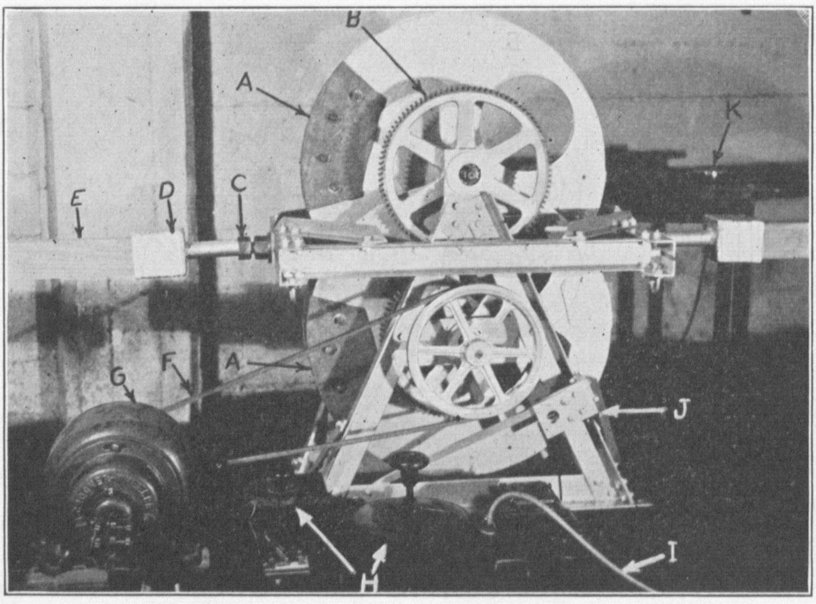


FIGURE 87.—The building and ground vibrator. (A) Lead plates, (B) 16-inch spur gear, (C) nut for tightening timber struts, (D) cup for holding strut, (E) the 4- by 4-inch strut, (F) V-belt, (G) D. C. motor, (H) rheostat, (I) armored cable bringing direct current from the motor-generator set, (J) bracket for the 6 to 1 reduction gear shaft, (K) tachometer.

attached to the base of the frame. For ground-shaking tests at Mare Island, a platform was constructed of 2- by 12-inch lumber and the vibrator bolted to this platform. Ballast and sloping braces were also used. The machine was also placed on its side to exert vertical forces for a few tests. This set-up is illustrated in figure 88. The Survay vibration meters and the spring recorder are placed in the foreground of the picture for the purpose of illustration only. Ordinarily, the instruments are placed at a considerable distance from the vibrator when records are taken, so that local disturbances will not affect the records. The method of using the instruments and recorders is described in another chapter of this report. In a forced vibration test, the positions of the vibrator and of the instruments that record the response of the structure to the disturbing forces depend upon the information sought. By means of judicious locations various modes of vibration of buildings can be excited and the motion recorded. The tests made in the Bank of America Bldg., which will be described later in this report, illustrate this feature of controlled vibration studies. In buildings, the Survey vibration meters (used with a magnification of about 200) yield excellent records of the responses to the forces of the vibrator. On rigid structures such as dams and on solid rock, it has been found necessary to use Wood-Anderson instruments with magnifications of about 1,500 in order to obtain legible records.

RESULTS OF EXPERIMENTS

Several experiments have been conducted with the vibrator for the purpose of testing it under various conditions and of ascertaining just what can be done in dynamic and seismological research with such a machine. Incidentally, these tests have furnished considerable information regarding the dynamic behavior of the structures vibrated.

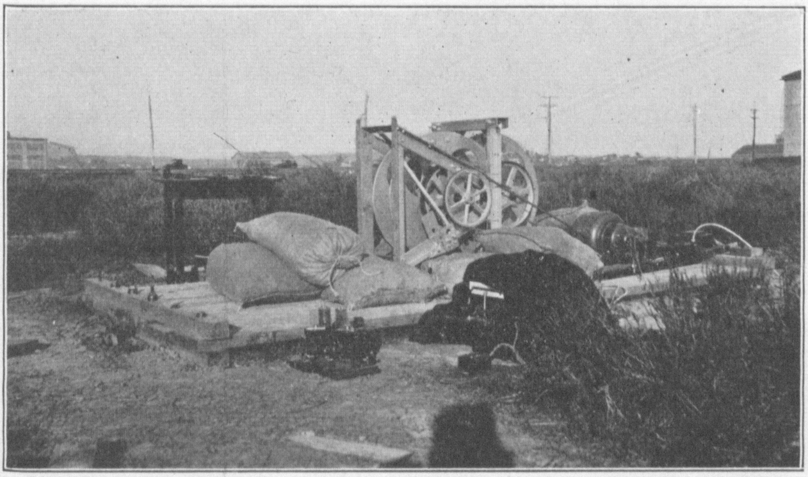


FIGURE 88.—Vibrator at Mare Island set to exert vertical forces.

The tests have included buildings, dams, bridges, rock, and soft ground; in each case records showing forced vibration have been obtained. The test results indicate that valuable information regarding the vibration characteristics of structures can be obtained by the use of such a machine that otherwise would be difficult to determine. This is especially true when simultaneous timing is used on several records. The new instrument being developed by Dr. Hugo Benioff to record vibrations from four different locations simultaneously on one record with possible magnifications up to 25,000 will be of great value in studies by the controlled, forced-vibration method.

Preliminary reports on most of the structures vibrated have been issued in mimeographed form from the San Francisco office. Although these reports are preliminary, they include practically all of the observational information obtained. Interpretation and mathematical treatment of the results have been omitted.

The following tests were made with the vibrator to the date of this report, May 1, 1935. No attempt is made here to describe every run made in these experiments nor to go into much detail. The tests are presented in chronological order.

Palo Alto Transfer & Storage Co. Building, Palo Alto, Calif., November 13 and 15, 1934.—This is a four-story "storage-type" building with reinforced concrete frame and floors, and hollow tile walls, Fig. 89. It is rectangular in plan, 75 by 40 feet. The vibrator was placed on the third floor parallel to the 75-foot side of the building, and the vibration meters were located on the fourth floor. Five runs of the vibrator were made using various amounts of unbalance. All of the records showed definite forced vibration and resonance effects. Resonance curves for four different degrees of unbalance are plotted in Fig. 90. The fundamental période of the building appears to be 0.18 second. Although this seems short for the fundamental period of a four-story building, it may be explained by the fact that the street wall in the direction under consideration contains no openings above

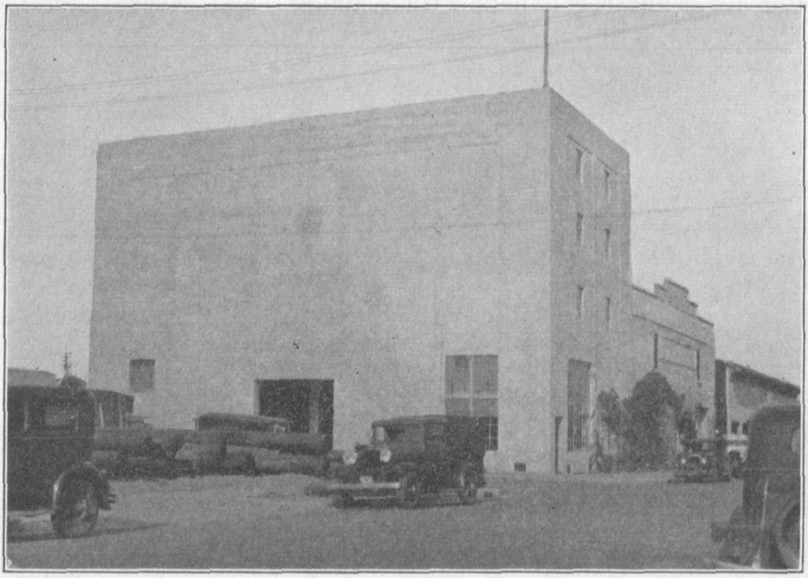


FIGURE 89.—Palo Alto Transfer and Storage Co. Building.

the first story, and is consequently very rigid, while the opposite wall contains no openings at all and is, therefore, still more rigid. The second period which is probably near 0.06 second could not be reached by the vibrator.

In four of the runs, the belt to the vibrator was removed at the highest speed and the records were taken as the machine slowed down. In one run, the belt was not removed. A comparison of the records indicates that the machine comes to a stop in about one-third of the time when the belt is not removed. However, there was no appreciable difference in amplitude for the two conditions.

This building offers opportunity for a computational study now that its response to the vibrator is known. Flexural deflections could be neglected in this structure without appreciable error. Since the amplitudes on the peaks of the resonance curves are approximately proportional to the forces of the vibrator, a purely viscous friction could be assumed in damping computations. A more detailed consideration of damping computations is presented elsewhere in this report.

Searsville Dam, near Woodside, Calif., November 27 and 28, 1934.—Searsville Dam is a gravity-arch dam of interlocking concrete blocks. It is 290 feet wide and stands 80.5 feet above its foundation. It is 51 feet thick at the bottom and 3 feet thick at the top. At the time of the tests, the water surface of the lake was approximately 20 feet below the top of the dam. The first runs yielded records that showed forced vibration, but no resonance peak. Accordingly, for the following runs, the amount of unbalance was reduced and the pulley ratio of the driving apparatus was altered so that a faster speed could be attained. A maximum speed of about 750 revolutions per minute was reached during the tests. Although the records obtained show the amplitude increasing out of proportion to the forces, there is no "peak" or period below which the vibration shows a reduced ampli-

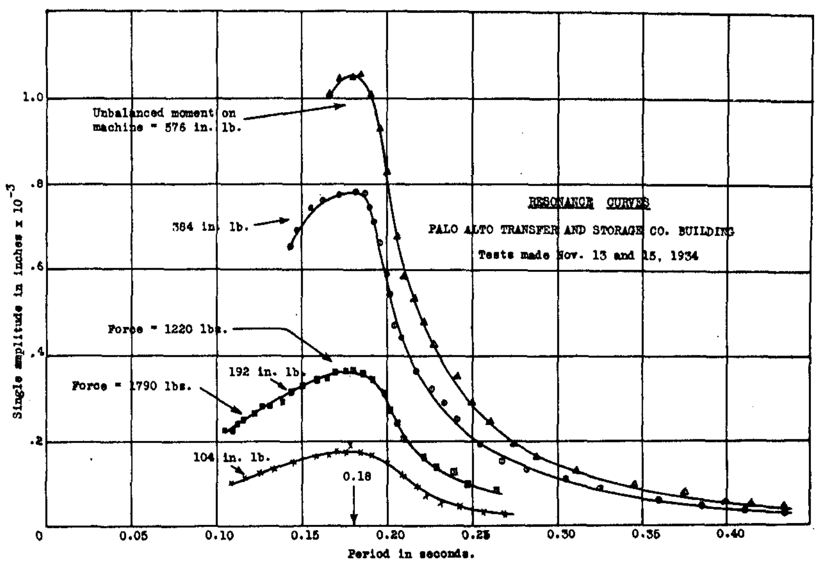


FIGURE 90.—Resonance curves, Palo Alto Transfer and Storage Co. Building.

tude. A resonance curve was plotted for these runs of the vibrator (fig. 91).

Some preliminary computations have been made on the assumption that this structure is a single-mass vibrating system. This assumption will not lead to serious error if the points for the computations are taken near the resonance peak. The method of computation was to select three points from the plotted curve and to solve three equations simultaneously for the viscous friction coefficient, the effective mass, and the spring factor of the system. Since at the time of this writing the work has not been checked for various combinations of points on the resonance curve, we hesitate to present the results beyond stating that the period of the dam is probably slightly shorter than attainable with the vibrator. To one significant figure (which is sufficient for this purpose) the period is 0.08 second. For the sake of safety, it is believed that a speed of 600 revolutions per minute should be arbitrarily assigned as a maximum speed for this machine, and in future tests no attempt will be made to exceed this.

Colorado Street Bridge, Pasadena, Calif. January 22, 1935.—This bridge is a long, reinforced-concrete structure consisting of 9 large arches, 6 of 113-foot span, 2 of 151.5-foot span, and 1 of 223-foot span. The vibrator was placed on the roadway, transverse to the longitudinal axis of the bridge and directly above the crown of the 223-foot arch. The tests were all made at night between the hours of 1 a. m. and 6 a. m. in order that traffic should not be delayed unnecessarily. No traffic was allowed on the bridge during any of the runs. Records of the forced vibration were taken over the long arch, over the two adjacent arches, and over the piers of the long arch.

All the records are complicated by resonance effects, beating, and the interdependence of the motion in the two components. Complicated records of forced vibration are to be expected from a structure such as this, which is composed of a great many structural elements

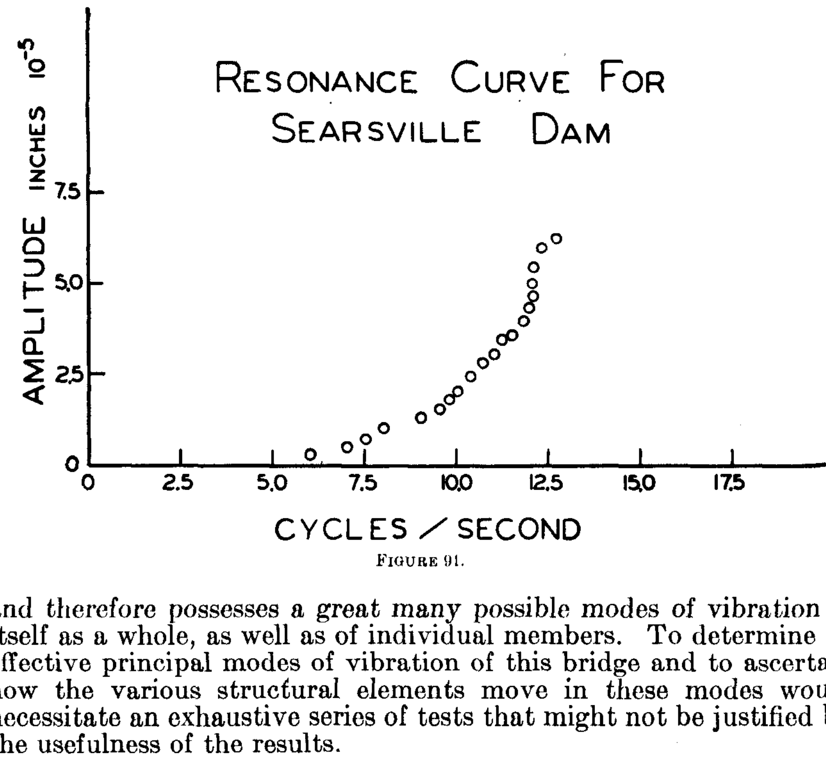


FIGURE 91.

and therefore possesses a great many possible modes of vibration of itself as a whole, as well as of individual members. To determine all effective principal modes of vibration of this bridge and to ascertain how the various structural elements move in these modes would necessitate an exhaustive series of tests that might not be justified by the usefulness of the results.

The tests made are sufficient to indicate that the bridge is very complex from a dynamic consideration, and that it is highly susceptible to resonance phenomena between the limits of 0.10 second and 0.90 second. In none of these tests was the speed of the vibrator held constant; in most cases the time required for the machine to slow down from high speed to 60 revolutions per minute was about 3 minutes. The probability of quasiresonance phenomena occurring during an earthquake is increased by the great number of natural periods of vibration of the structure.

The periods of some of the most pronounced "peaks" on the records are tabulated below. Most of these peaks indicate natural periods of vibration. In many cases the peaks occur simultaneously on both components; this indicates that (at the location of the instrument) the principal direction of motion is not parallel to either component of the vibration meter. Resonance curves for a typical record are shown in figure 92.

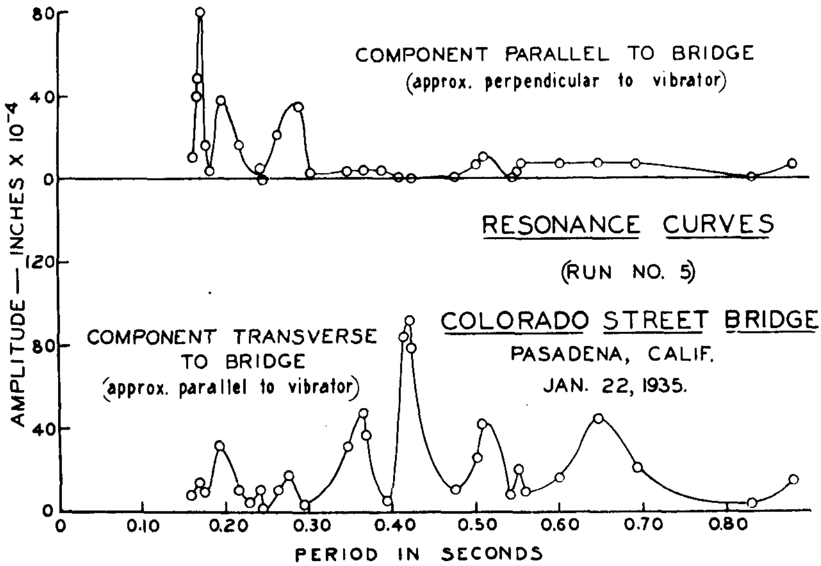


FIGURE 92.—Resonance curves for Colorado Street bridge, Pasadena.

TABLE 20.—*Periods corresponding to some of the most pronounced peaks on the Colorado Street Bridge resonance curves*

Run no.	Location of recorder	Periods	
		Transverse to roadway	Parallel to roadway
		Seconds	*Seconds*
3	Over crown of 223-foot arch	0.17; 0.19; 0.21 [1]; 0.24; 0.27 [1]; 0.42 [1]; 0.87 [1] (end of record 0.87+).	0.17; 0.19; 0.21 [1]; 0.27; 0.38; 0.87.
4	Over west pier of 223-foot arch	0.17; 0.21; 0.24; 0.27 [1]; 0.43; 0.66 (end of record 0.66+).	0.17; 0.19; 0.24.
5	Over crown of arch west of 223-foot span.	0.19; 0.29; 0.37; 0.42 [1]; 0.51; 0.65; (end of record 0.88).	0.17 [1]; 0.19; 0.29; 0.51.
6	Over east pier of 223-foot arch	0.17; 0.19; 0.21 [1]; 0.24; 0.28 [1]; 0.43; (0.86 after vibrator stopped).	0.17; 0.21; 0.24; 0.28 [1]; 0.43; 0.50.
7	Over east pier of 223-foot arch	0.12; 0.14; 0.17; 0.21; 0.24; 0.28; (end of record 0.3. Underlying period 0.87 [1])	0.17; 0.21.
8	Over crown of 223-foot arch	0.12; 0.21; 0.24; 0.27; (forced vibrations not apparent beyond 0.27; underlying period 0.86 [1]).	0.21; 0.27; (some evidence of 0.38).

[1] Very pronounced.

NOTE.—The moment of the vibrator was 384 in.-lb. for runs 3, 4, 5, and 6; and 104 in.-lb. for runs 7 and 8.

It should be pointed out that a structure of the complexity of the Colorado Street Bridge does not yield either experimental or theoretical results that can easily be interpreted. A study by the forced vibration or vibrator method is the most direct way of approaching

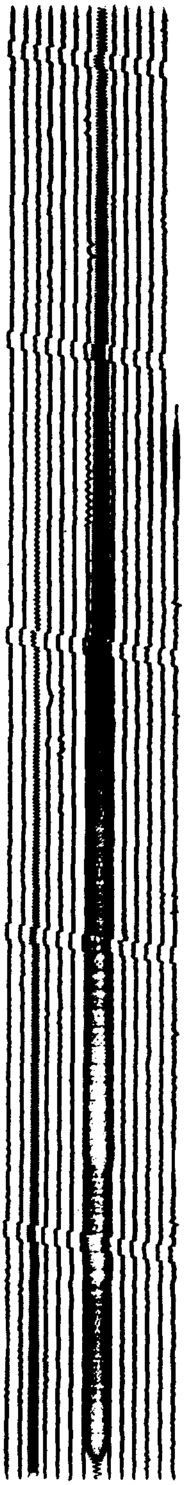


FIGURE 93.—Record from Benioff strain seismometer at Pasadena, showing forced vibration of the solid rock.

the problem, but even such a study—after the bridge has been built—gives very complicated results. It would be well to concentrate on a study of relatively simple structures, for instance regularly shaped buildings, until more knowledge and experience has been gained.

Pasadena Seismological Laboratory, Pasadena, Calif., January 23, 1935.—A few runs were made with the vibrator placed on the ground adjacent to the seismological laboratory. Records were obtained on the Benioff strain seismometer, which has a very high magnification factor. This record shows that the vibrator was affecting the bedrock which outcrops at the site of the laboratory. A portion of the record of two of the runs is shown in figure 93. The indicated time interval is 1 minute. These records have not been analyzed. It is very doubtful if any period is excited. It is interesting to note that the forces were sufficient to vibrate solid rock sufficiently to yield a record.

Morris Dam, San Gabriel Canyon, Calif., January 25 and 26, 1935.—The Morris Dam is a large concrete gravity dam built by the city of Pasadena. It is especially interesting in that a minor and apparently inactive fault intersects the dam foundations in a direction almost normal to the axis of the dam, and that special provisions were made for movement along this fault in the design of the structure. These precautions include an open joint along which motion can occur without hazard to the stability of the dam. The length of the dam at its crest is 780 feet; it is 328 feet high (from its lowest foundation); and it contains 514,000 cubic yards of concrete. The elevation above sea level of the water behind the dam at the time of the tests was 1,088 feet, about 87 feet below the roadway on top of the dam. See figure 41.

A series of 17 runs was made with the vibrator placed in two different positions on top of the dam and with the Wood-Anderson instrument located in many positions across the dam from one abutment to the other. One record was taken in a gallery at the bottom of the dam directly below the vibrator. Particular attention was paid to the open joint in locating the vibrator and the instruments. All of the records with the exception of one taken on a rock abutment show forced vibration. However, the amplitude of motion is very slight on most of the records and analysis has been postponed until a suitable micrometer-scale instrument can be secured. This instrument will facilitate the analysis of all future records and will greatly increase the accuracy of the amplitude readings.

It is believed that the records will prove very interesting. A cursory examination indicates a period of about 0.20 second as well as one of 0.17 second on most of the traces. From some records taken in June 1934 a period of 0.17 second is apparent. A report on the tests made on the Morris Dam will be issued shortly after the micrometer scale has been obtained.

Los Angeles City Hall, January 28 and 29, 1935.—A series of vibrator tests was conducted in the Los Angeles City Hall to learn whether or not the machine could vibrate such a large building, and also to establish a precedent in the forced vibration of large buildings in order that other building owners would feel secure in allowing their buildings to be studied. No definite program of experiments was conducted with the purpose of ascertaining principal modes of vibration or deflection curves, as it was felt that a smaller and more symmetrical structure would be better suited for the initial attempt at such a study.

The building is large and irregular. The first three floors are 430 by 254 feet in plan; the fourth to tenth floors are 54 by 318 feet with a "cross" in the center; the tower, which starts at the tenth story and rises to the twenty-fifth story, is about 75 by 75 feet in plan. For all of the tests made, the vibrator was located on the twenty-fourth floor. Records were taken on several floors with the Survey vibration meters, using a magnification of 230.

It was found that the vibrator, located on the twenty-fourth floor, vibrated the building sufficiently to yield records of forced vibration on each floor tested, namely the twenty-fifth, twenty-fourth, twentieth, fifteenth, twelfth, eighth, and fifth floors. On most of the records the forced vibration was partially obscured by the large waves of the fundamental mode excited by wind and other agencies. In future tests this fundamental wave could be greatly reduced by using a different instrumental period without reducing the amplitude of the forced vibration waves. In the tests made the instrumental period happened to be set very nearly the same as the fundamental periods of the building. Several periods of vibration were excited and they are tabulated below. It was found that for this large building a greater amount of unbalance than 768 pounds (which was the maximum amount available at the time) would be necessary to excite periods longer than 0.4 second. This includes the third mode of vibration which probably lies between 0.4 and 0.5 second.

TABLE 21.—*Periods in Los Angeles City Hall*

Floor	North-south direction		East-west direction	
	Run no.	Periods	Run no.	Periods
		Seconds		*Seconds*
25	2	2.2 [3]; 0.78; 0.47 [4]; 0.29 [1]; 0.23 [1]	10	2.3; 0.83; 0.21.[1]
24	1	2.2; 0.31 [1]; 0.29 [1]; 0.21 [1]; 0.18 [1]	([2])	
20	3	2.2; 0.29 [1]; 0.25 [1]	9	2.3; 0.35 [1]; 0.29.[1]
15	4	2.2; 0.78 [3]; 0.5 [4]; 0.34 [1]; 0.29 [1]	11	2.3; 0.29 [1]; 0.20.[1]
12	5	2.2; 0.75; 0.36 [1]; 0.21 [1]	12	2.3; 0.77 [3]; 0.20.[1]
8	6	2.1; 0.76 [3]; 0.6 [4]; 0.29 [1]	13	2.3; 0.29.[1]
5	7	0.76; 0.6 [4]; 0.34 [1]; 0.20 [1]	14	2.3; 0.78 [3]; 0.29 [1]; 0.24 [1]; 0.20.[1]

[1] Excited by the vibrator.
[2] No record taken.
[3] Approximate value.
[4] Existence of period questionable.

NOTE.—Run no. 8 showed no forced vibration; the instrument was set transverse to the direction of the vibrator.

Bank of America Building, San Jose, Calif., February 8, 11, 12, 13, 20, and 21, 1935.—The only structure that has been tested in detail by the vibrator is the Bank of America Building in San Jose. This building is especially interesting because of the fact that it contains two permanently located strong-motion instruments, one on the thirteenth floor and one on the basement floor. Contrary to the belief of many engineers, this building was not designed to be a flexible first-story structure. Two views of the building are shown in figure 94. A typical vibrator test run is shown on page 41.

As an example, the preliminary report on the Bank of America Building is presented in its entirety. It illustrates what can be

FIGURE 94.—Bank of America Building, San Jose. Two views.

done in building research with the vibrator. A few of the runs might be considered superfluous. However, this was the first attempt at a comprehensive series of tests and it was necessary to proceed to a certain extent by trial and error. In any future tests of this nature we shall have this one as a precedent. This particular building was ideal for these tests, because the top story was not only unoccupied but contained no partitions; thus it was possible to brace between any two columns desired. Moreover, the management was very obliging and courteous in placing the entire thirteenth floor at the disposal of the vibration party.

Site of the new San Francisco Mint, March 15, 1935.—Three runs of the vibrator were made on the site of the new San Francisco Mint. The site is the block bounded by Hermann, Webster, Buchanan, and Duboce Streets. It is a rocky hill with steep cliffs on Duboce,

Preliminary Report

DEPARTMENT OF COMMERCE
U. S. COAST AND GEODETIC SURVEY
CALIFORNIA SEISMOLOGICAL PROGRAM

FORCED VIBRATION TESTS

BANK OF AMERICA BUILDING. Corner of First and Santa Clara Streets, San Jose, Calif.

DESCRIPTION OF BUILDING: This building has 13 stories and a tower. The main part of the structure is 54 ft. by 125 ft. in plan. (See Fig. 95). The frame is steel, fireproofed with concrete; walls, concrete with face brick; partitions, lath and plaster; floors, concrete; foundations, concrete footings on piles; soil; unconsolidated sand and clay (old creek bed). A one-story building abuts on the east.

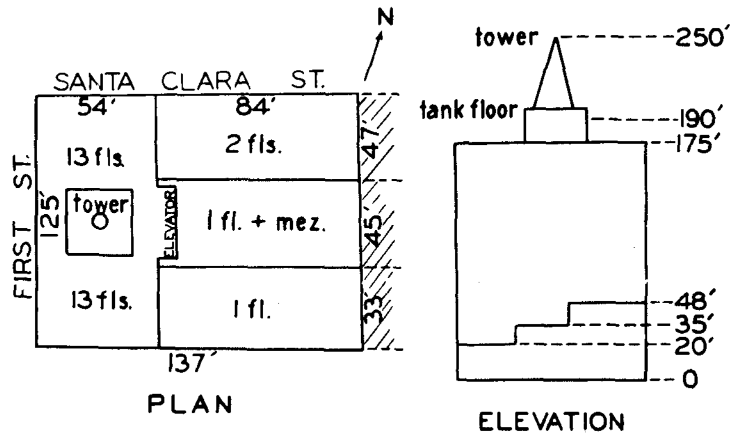


FIG.95

OBSERVERS: Blume, Carder, Taylor, Campbell.

OBSERVATIONS: In each run the unbalanced moment on the vibrator was 768 in. lb. The maximum speeds of the various runs ranged between 320 and 380 R. P. M. Records were taken as the vibrator slowed down after the driving belt was removed.

<u>Run No. 1</u>. February 8, 1935.

Vibrator on 13th floor, about 12 ft. south of the center line of the building, parallel to Santa Clara St.

Vibration meter approximately in the center of the 13th floor. Both components. Magnification, 200.

<u>Run No. 2</u>. February 8, 1935.

Vibrator in same position as in Run 1.

Vibration meter on 13th floor, in line with the vibrator, close to the First St. wall. Both components. Magnification, 200.

<u>Run No. 3</u>. February 8, 1935.

Vibrator in same position as in Run 1.

Vibration meter on the 13th floor, close to the south wall, on the N-S axis of the building parallel to First St. Both components. Magnification, 200.

<u>Run No. 4</u>. February 8, 1935.

Vibrator in same position as in Run 1.

Vibration meter on the 13th floor, close to the Santa Clara St. wall, on the N-S axis of the building. Both components. Magnification, 200.

Run No. 5. February 8, 1935.

Vibrator in same position as in Run 1.

Vibration meter on the 5th floor, in line with the vibrator, close to the First St. wall. Both components. Magnification, 200.

Run No. 6. February 8, 1935.

Vibrator in same position as in Run 1.

Vibration meter on the 5th floor, close to the south wall, on the N-S axis of the building. Both components. Magnification, 200.

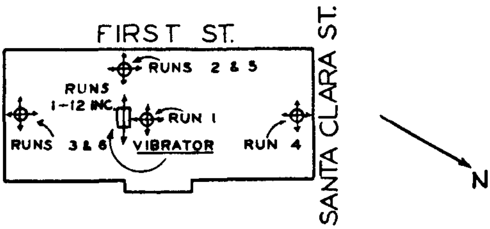


FIG. 96

PLAN OF SET-UPS

Everything on the 13th floor except vibration meter for Runs 5 and 6, when on the 5th floor.

⌖ - indicates Vibration meter.

Run No. 7. February 11, 1935.

Records failed (simultaneous timing trouble).

Run No. 8. February 11, 1935.

Records failed (simultaneous timing trouble).

Run No. 9. February 12, 1935.

(A repetition of Runs 7 and 8).

Vibrator in same position as in Run 1.

Vibration meters. This run and the 3 following runs were recorded simultaneously on three different floors. In this run each vibration meter was located near the east wall about 10 ft. south of the E-W axis of the building parallel to Santa Clara St. (See Fig. 97). The component recorded in each case was parallel to the vibrator. Magnifications on the 13th, 9th, and 5th floors were 100, 100, and 200, respectively.

Run No. 10. February 12, 1935.

Records failed.

Run No. 11. February 12, 1935.

(A repetition of Run 10).

Vibrator in same position as in Run 1.

Vibration meters in same position on each floor as in Run 9. Simultaneous records. Magnifications on 11th, 7th, and 3rd floors were 100, 100, and 200, respectively.

Run No. 12. February 12, 1935.

Vibrator in same position as in Run 1.

Vibration meters in same position on each floor as in Run 9. Simultaneous records. Magnifications on 2nd, and 1st floors, and basement were 100, 200, and 1400, respectively. Time spots on first floor failed.

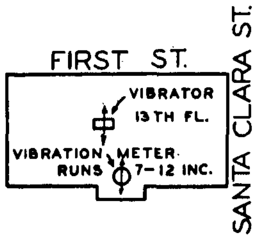


FIG. 97

Run No. 13. February 13, 1935.

Vibrator on 13th floor, about 5 ft. west of the center line of the building, parallel to First St.

Vibration meter on 13th floor, close to the south wall, on the N-S axis of the building. Both components. Magnification, 200.

Run No. 14. February 13, 1935.

Vibrator in same position as in Run 13.

Vibration meter on 13th floor, close to the First St. wall, near the E-W axis of the building. Both components. Magnification, 200.

Run No. 15. February 13, 1935.

Vibrator in same position as in Run 13.

Vibration meter on 5th floor, directly below the position of the set-up for Run 13. Both components. Magnification, 200.

Run No. 16. February 13, 1935.

Vibrator in same position as in Run 13.

Vibration meter on 5th floor, directly below the position of the set-up for Run 14. Both components. Magnification, 200.

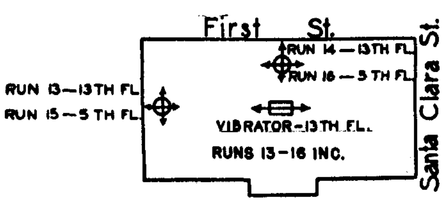


FIG. 98

Run No. 17. February 20, 1935.

Vibrator on 13th floor, parallel to Santa Clara St., about 15 ft. from the south wall.

Vibration meter on 13th floor, close to the south wall, on the

N-S axis of the building. Both components. Magnification, 200.

Run No. 18. February 20, 1935.

Vibrator in same position as in Run 17.

Vibration meter on 13th floor, close to the First St. wall, near the E-W axis of the building. Both components. Magnification, 200. This record was partially spoiled by a sudden and rather severe movement of the building caused by some other agency than the vibrator. This is the only record upon which this movement was noticed.

Run No. 19. February 20, 1935.

An exact repetition of Run 18.

Run No. 20. February 20, 1935.

Vibrator in same position as in Run 17.

Vibration meters. Simultaneous records were made with one instrument on the 13th floor, close to the south wall and on the N-S axis of the building; one instrument on the 5th floor directly below the one on the 13th; and the third instrument on the 5th floor near the E-W axis of the building and close to the First St. wall. Magnifications on the 13th, 5th-south wall and 5th-First St. wall were 100, 100, and 200, respectively.

Run No. 21. February 20, 1935.

Vibrator in same position as in Run 17.

Vibration meters A, B, and C located on the 13th floor as shown in diagram. The records were taken simultaneously. The magnification at meters A, B, and C were 100, 100, and 200, respectively. Time spots on B failed.

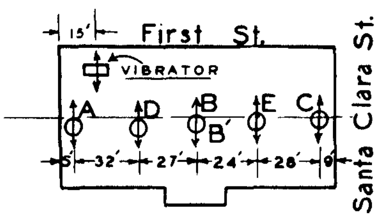


FIG. 99

Run No. 22. February 20, 1935.

Vibrator in same position as in Run 17.

Vibration meters D, B', and E located on the 13th floor as shown under Run 21. The records were taken simultaneously. The magnification at meters D, B', and E were 100, 100, and 200, respectively.

Run No. 23. February 21, 1935.

Vibrator in same position as in Run 17.

Vibration meters F, G, and H located on the 13th floor as shown in diagram. The records were taken simultaneously. The magnification at meters F, G, and H were 100, 200, and 100, respectively.

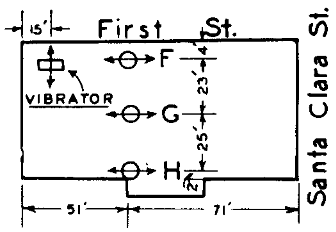


FIG. 100

Run No. 24. February 21, 1935.

Vibrator moved back to same position as in Run 1.

Vibration meters located (on each floor) in the same position as for Run 9. Records were taken simultaneously on the Tank floor (in tower), 10th floor, and 6th floor. Magnifications at the meters were 200, 100, and 100, respectively.

Run No. 25. February 21, 1935.

Vibrator in same position as in Run 1.

Vibration meters in same position (on each floor) as in Run 9. The magnification at the meters on the 12th, 8th, and 4th floors were 100, 100, and 200, respectively.

Run No. 26. February 27, 1935.

Vibrator set-up on the basement floor, approximately below its position for Run 1 on the 13th floor; parallel to Santa Clara St. Vibration meters: One on the 13th floor, centrally located, parallel to Santa Clara St.; the other in the basement, in line with and parallel to the vibrator, about 40 ft. west of same. The records were taken simultaneously. The magnification on the 13th floor and in the basement were 200 and 1400, respectively.

NOTE ON TORSIONAL VIBRATION. It has been noticed on some of the records showing both components that the vibrator excites (very slightly) natural modes of vibration that are peculiar to the direction transverse to that of the vibrator. It is to be expected that torsional modes of vibration be excited in this manner when the vibrator is not located at the center or rotation, and such was definitely the case in these tests. It is interesting that these other transverse vibrations should be set up, even though very slightly. The vibrator was oriented exactly perpendicular to the sides of the building in all tests.

PERIODS. On some of the records for the component parallel to Santa Clara St., the periods 0.24 and 0.26 sec. are apparent. On other records only one period in this range is noticed, 0.24, 0.25, or 0.26 sec. The tests indicate that the 0.26 sec. period is that of a torsional mode of vibration, and that the 0.24 sec. period is that of a transverse mode of vibration, probably the third mode. These two periods are so close on the records that they are often difficult to distinguish.

Following is the most probable classification of periods for this building as shown by these tests:

	Parallel to S. C. St.	Parallel to 1st St.	Torsional
T_1	1.33 sec.	1.20 sec.	0.84 sec.
T_2	0.38 "	0.34 "	0.26 "
T_3	0.24 "	0.23 "	-
T_4	0.18 "	-	-

It is believed that the periods parallel to Santa Clara St. and the torsional periods are fairly well established. There is some doubt regarding the classification of the periods parallel to First St. since very few runs were made in that direction, and they were recorded on the 5th and 13th floors only. A pronounced period of 0.44 sec. is noticed in the First St. direction for these runs, and this period may be that of the second mode of vibration. However, it seems more likely that this 0.44 sec. period is that of the tower on top of the building. An indicated 0.50 sec. period in the other direction would correspond to this as the period of the tower parallel to Santa Clara St. If the 0.34 sec. period be considered as the second mode of vibration, the following table of ratios results:

	Parallel to S. C. St.	Parallel to 1st St.	Torsional
T_1/T_2	3.5	3.5	3.2
T_1/T_3	5.5	5.2	-
T_1/T_4	7	-	-

TABLE 22. PERIODS OBSERVED IN THE BANK OF AMERICA BUILDING

Floor	Run No.	Periods Parallel to Santa Clara St.					Periods Parallel to First St.			
		Wind[1]	Vibrator				Wind[1]	Vibrator		
		sec.	sec.				sec.	sec.		
Tower at Tank level	24	1.32	0.38	0.23	0.17?					
13	1	1.31	1.33	0.38	0.24		1.21	0.23		
13	2	1.32	1.33	0.38	0.26	0.23	1.20	0.26		
13	3	1.31	1.31	0.37	0.26		1.20	0.22?		
13	4	1.31	1.33	0.82	0.38	0.26	1.19			
13	9	1.32	1.33	0.38	0.26?	0.24?				
13	13	1.32					1.21	1.19	0.44 0.34	0.23
13	14	1.31					1.21	1.20	0.44 0.34	0.23
13	17	1.32	1.33	0.84	0.38 0.26	0.18	1.20	0.84	0.26	
13	19	1.31	1.33	0.38	0.26		1.22	0.84	0.26	
13	20	1.33	1.34	0.83	0.38	0.26				
13	21A	1.3ca	1.29	0.84	0.38	0.26				
13	21C	1.32	1.33	0.84	0.38	0.26				
13	22D	1.33	1.33	0.84	0.38	0.26				
13	22B	1.34	1.34	0.38	0.26					
13	22E	1.34	1.34	0.84	0.38	0.26				
13	23F						1.22	0.84	0.26	
13	23G						1.22	0.37?	0.26?	
13	23H						1.22	0.84	0.26	
13	26	1.30	No motion evident				1.20			
12	25	1.33	1.33	0.38	0.24					
11	11	1.33	1.33	0.50?	0.38 0.25	0.19				
10	24	1.32	0.38	0.26?	0.24	0.18?				
9	9	1.32	1.33	0.38	0.26?	0.24				
8	25	1.32	0.38	0.26	0.23					
7	11	1.33	1.33	0.50?	0.38	0.24				
6	24	1.32	0.38	0.26						
5	5	1.32	1.32	0.50	0.38 0.25	0.18	1.22	0.25		
5	6	1.3ca	1.32	0.38	0.26	0.18	1.20			
5	9	1.32	1.33	0.50?	0.38 0.26	0.19				
5	15	1.31					1.20	1.20	0.44 0.34	0.23

5	20s	1.3ca	0.84	0.38	0.26					
5	20[1]	1.3ca	0.83	0.38	0.26					
5	16	1.32	0.34	0.23			1.21	1.21	0.45 0.35	0.23
4	25	1.32	1.33	0.38	0.24					
3	11	1.32	0.38	0.24	0.18?					
2	12	1.32	0.38	0.25						
Basement	12		0.38	0.25						
Basement	26	Ample motion but no periods evident								

[1] Excited by wind or other agencies than vibrator. These periods are average values. The maximum variation is ±0.02 sec.

SECOND MODE OF VIBRATION. The main purpose of the simultaneous measurements on various floors was to obtain data for plotting the deflection curve of the building in its second mode of vibration parallel to Santa Clara St. For each particular run the measured points on the three floors were taken at the same instant with particular attention to phase differences. For example, in Run 9 (taken on the 5th, 9th, and 13th floors), the 5th and 9th floor deflections were in phase, but the 13th floor deflection was approximately 180° out of phase with the other two floors. (See Fig. 101).

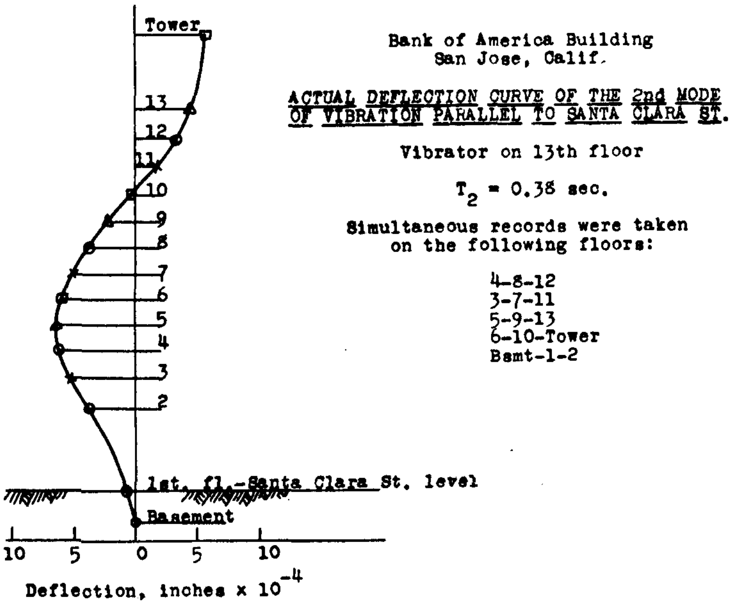


FIG. 101

It is interesting to note that the points for various runs agree. This is because the vibrator causes all of the motion, and its disturbing forces are the same for each run.

FUNDAMENTAL MODE OF VIBRATION. Points for the fundamental mode of vibration have been plotted also. The points for any one run line up approximately as a straight line, which indicates that the building deflects in flexure as well as in shear. This agrees with mathematical studies made last year on a building of similar ratio of height to width. The points for the various runs do not agree since the wind is affecting the motion as well as the vibrator. (See Fig. 102).

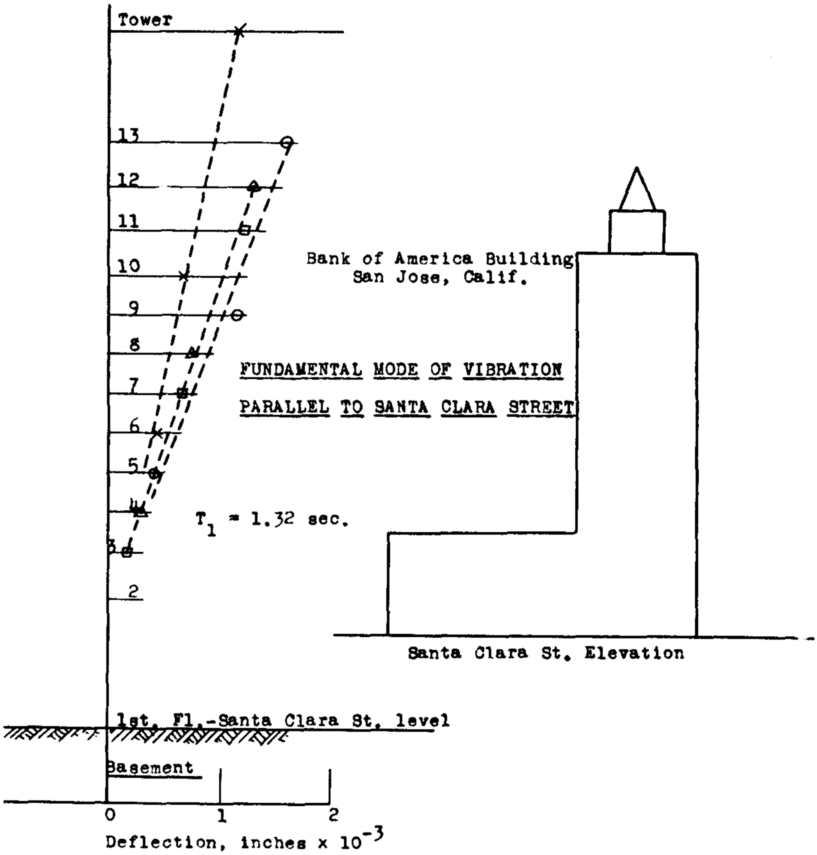
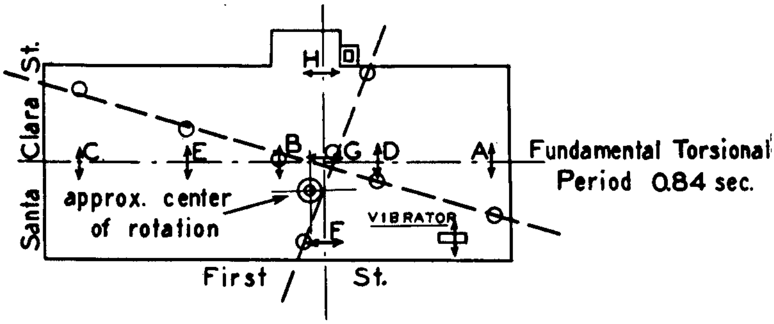


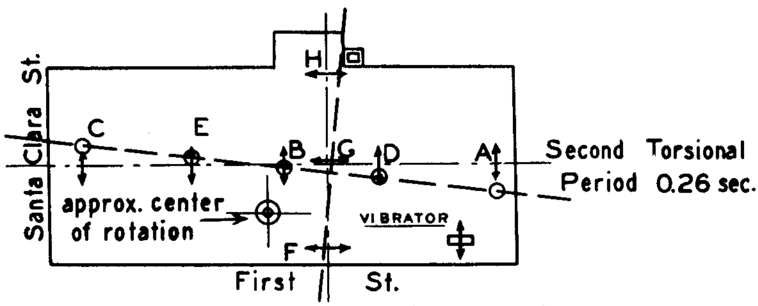
FIG. 102

TORSIONAL MODES OF VIBRATION. From Runs 21, 22, and 23 the deflections of the 13th floor while vibrating in the first and second torsional modes of vibration were plotted with particular attention to phase relationships, and from these points the approximate center of rotation of this floor was located. This center seems to be closer to the First St. wall than to the opposite wall on this floor. The First St. wall on this story has no openings while the opposite wall contains several windows. (See Fig. 103 on following page).

VIBRATOR IN BASEMENT. Run No. 26. Records were taken simultaneously on the 13th floor and on the basement floor with the vibrator set up between two columns in the basement. The record taken on the 13th floor (with a magnification of 200) showed no response to the vibrator. The basement record (with a magnification of 1400) showed a definite response but no natural periods. A resonance curve has been plotted for this record. (See Fig. 104 on following page). The test seems to indicate that the vibrator was shaking the ground but not the building as a whole. When the vibrator was on the 13th floor and the instrument in the basement, there was an indication of periods of 0.38 sec. and 0.25 sec. on the record taken in the basement.



Plan of 13th floor showing deflections for the fundamental torsional mode of vibration.



Plan of 13th floor showing deflections for the second torsional mode of vibration.

FIG. 103

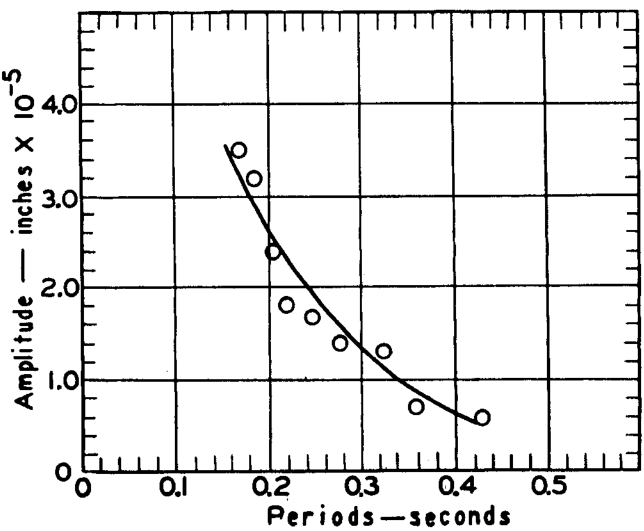
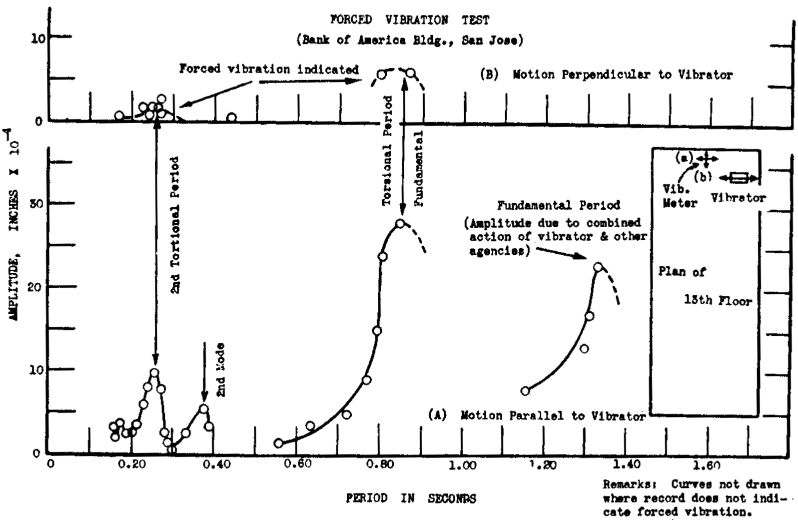


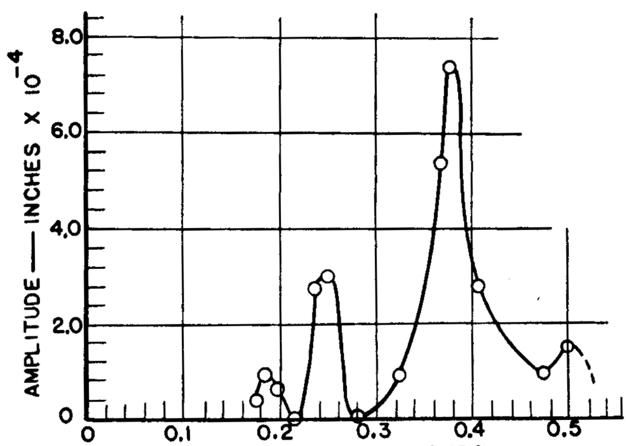
FIG. 104

RESONANCE CURVES. Resonance curves plotted for a few typical runs are shown in Figs. 105, 106, and 107. The fundamental period of the building is not shown in Figs. 106 and 107.



RESONANCE CURVE FOR RUN 17

FIG. 105



RESONANCE CURVE FOR RUN 5—5TH FLOOR

FIG. 106

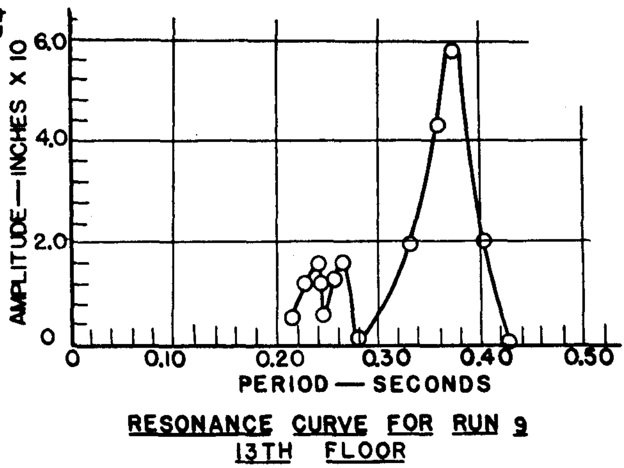


FIGURE 107.

Buchanan, and the easterly end of the Hermann Street side. The hill, which consists mainly of serpentine and chert, is nearly 100 feet above the street level at the corner of Buchanan and Duboce Streets. Loose sand overlies the rock to a depth of several feet at many locations on top of the hill.

The vibrator was in the same position for all runs. It was placed on top of the hill between two masonry piers of an old reservoir. Records were taken with the vibration meter located on top of a neighboring pier and also on the sand about 70 feet from the vibrator. Motion was evident on each record taken, but no "peaks" were obtained. The only results are of a negative character. The plotted resonance curves indicate that there are no predominant periods between the limits of 0.10 second and 0.36 second. However, they also indicate, by the fact that the amplitudes increase out of proportion to the forces at high speeds, that there may be some natural period or periods slightly shorter than 0.10 second. The maximum speed attained by the vibrator was about 580 revolutions per minute because of the excessive line-voltage drop to the motor in the long length of cable.

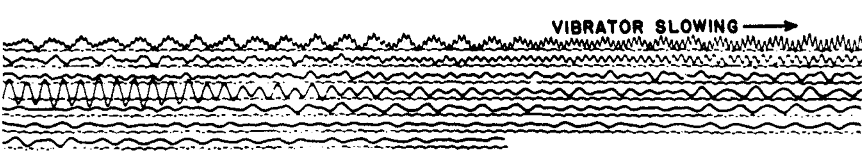


FIGURE 108.—Vibrator test run No. 20. Bank of America Building, San Jose.

Mare Island ground vibrating tests, April 16, 17, and 18, 1935.—A series of tests was conducted on dredger-filled ground at Mare Island. A platform was constructed of 2- by 12-inch lumber upon which ballast was placed (see fig. 88). This platform held the vibrator satisfactorily during all of the tests. Twenty-three runs were made with the vibration meters located in various positions relative to the vibrator. Motion was recorded as far as 1,000 feet from the machine with a Wood-Anderson instrument. Considerable motion was recorded at stations located out to the side of the vibrator as well as at stations located directly in line with it. The motion of the ground within 100 feet of the platform was so great that it could be felt. Two runs were made with the vibrator placed on its side to exert vertical forces. The records are being analyzed. They indicate, at least, that the vibrator can shake soft ground considerably.

Causeway tests, Mare Island, Calif., April 19 and 22, 1935.—An attempt was made to vibrate the new causeway between Mare Island and Vallejo. The tests were difficult because there was no place to anchor the vibrator near the top of the structure (on the roadway), as it was impossible to stop the heavy traffic. The only alternative was to place the vibrator on a platform, which is below the roadway and within a few feet of the high-water line. There was insufficient room on this platform to properly anchor the machine, and consequently high speeds and relatively large amounts of unbalance were prohibitive. This position was too low on the structure for the comparatively small forces produced to be effective, since a great deal of energy was absorbed by the piles and the mud. Consequently, the records obtained are unsatisfactory for plotting resonance curves.

Chapter 8.—A REPORT ON EARTHQUAKE DAMAGE TO TYPE III BUILDINGS IN LONG BEACH

R. R. Martel

This is the report of an investigation of damage caused by the March 10, 1933, earthquake in Long Beach, Calif., to the group of buildings designated as class C in many of the older building codes and as type III in the Uniform Building Code.[1] This particular class was chosen since, in number of buildings (1,261) and spread of distribution, it was second only to wood frame residences, on which a somewhat analogous survey had already been made by a committee of the Structural Engineers' Association of Southern California.

The aims of this investigation were: First, to determine if appreciable differences in the intensity of the earthquake existed inside the Long Beach city limits; second, to determine if significant differences in damage resulted from differences in the building's subtype, occupancy, or adjacency to other buildings.

For a measure of damage, the use of the ratio of cost of repairs to the original cost appeared feasible because of readily available data. While in many cases the figures obtained for such cost of repairs did not constitute an absolute measure of damage, they are tolerably satisfactory as indices of trends when averages for fairly large groups are used comparatively.

As a necessary preliminary to the actual studies, a card index was prepared in which, for each building, the height, subtype, occupancy, and adjacency to other buildings were noted. This information was obtained from the Sanborn Fire Insurance Atlas, and by field checks of the buildings themselves. The building permit files are complete since 1926 so that the original costs for buildings built after 1926 were available as well as the cost of repairs for all buildings repaired up to August 1934. As a result of the building department's large force of inspectors in the field following the earthquake, very few, if any, of the repairs to type III buildings were made without a permit recorded in the department's files.

The card index was completed from data in the office of the city assessor. The information here available was well adapted for the purposes of this investigation due to several unusual circumstances. First, the city assessor had just completed new assessments of all property prior to March 10, 1933. Second, the earthquake's occurrence a few days after the first Monday in March, the date on which assessments are based, was in part responsible for State legislative changes permitting assessed values for 1933 in Orange and Los Angeles Counties to be based as of March 10, 1933, i. e., after the earthquake.

[1] Class C, or type III buildings—ordinary masonry construction; i. e., exterior masonry bearing walls with interior load-bearing construction of wood, steel, or masonry. Partitions, roof, and floor framing may be wood. Type III is not confined to brick; yet, at the time of the earthquake, practically all type III buildings in Long Beach were brick construction.

However, in order for property owners to benefit under these new acts, it was necessary for them to file an earthquake damage claim for each improvement, giving the amount of damage or the cost of repairs. These claims were all checked by the assessor's office and new assessed values for 1933 were determined. The manner of this determination can best be shown by an example.

Consider a store building with an original cost of $10,000. If new, it would be assessed at 50 percent of cost, or $5,000; but, if it is 10 years old, it might, due to depreciation, be assessed at 40 percent of cost, or $4,000. If the owner claimed and was allowed $800 damage on this 10-year-old building, the reduction in assessed value would be 40 percent of the $800, or a $320 reduction in assessed value. On each building's card in the card index, the original 1933 assessed value and the reduction granted were noted.

The assessor's office further checked all buildings before March 1934, so that buildings which had been damaged but not repaired within a year had a lower assessment for 1934 than for 1933. This decrease in assessed value was noted on the cards the same as a reduction in 1933 values.

The information from the assessor thus supplied an assessed value for each building and a reduction in this value due to earthquake damage when claims had been filed or when the building had not been repaired within a year.[2] This source of reductions yielded values for approximately 60 percent of the buildings, but probably included more than an average proportion of high-value or high-damage buildings. The building permits yielded similar information for about 30 percent of the buildings. The rest of the buildings (less than 10 percent), for which no claims for damage were available from either the assessor's rolls or the building permit files, had reductions estimated for them where the field check disclosed damage. Reductions in assessments were also adjusted when they were obviously erroneous or not due primarily to earthquake damage. For instance, a heavily damaged obsolete building which could have been repaired might have been demolished for economic reasons. This would result in a 100 percent reduction in assessed value. In the few cases of this type which required adjustments, modified values based on estimated costs of repairs were used.

Since for the study of variation in intensity no reliable direct measurements were available, the problem was attacked by considering variations in amount of damage. The method employed was to determine the relation in percent between the total assessed value of type III buildings only and the total adjusted reductions in assessed value for each block. These percentages varied from 3 percent to 90 percent and were arranged in 5 groups: 0 percent to 10 percent, 10 percent to 17 percent, 17 percent to 23 percent, 23 percent to 30 percent, and 30 percent to 100 percent. Each group was designated by crosshatching on the map, figure 109, so that heavily damaged areas showed black, and lightly damaged areas were just gray enough to be distinguishable from the unshaded areas having no type III buildings.

In some cases the results plotted on the map indicate sharp differences in damage between adjacent blocks, but these differences can and should be attributed to a number of factors other than variation

[2] No data were available for public buildings, which have, therefore, been excluded from consideration.

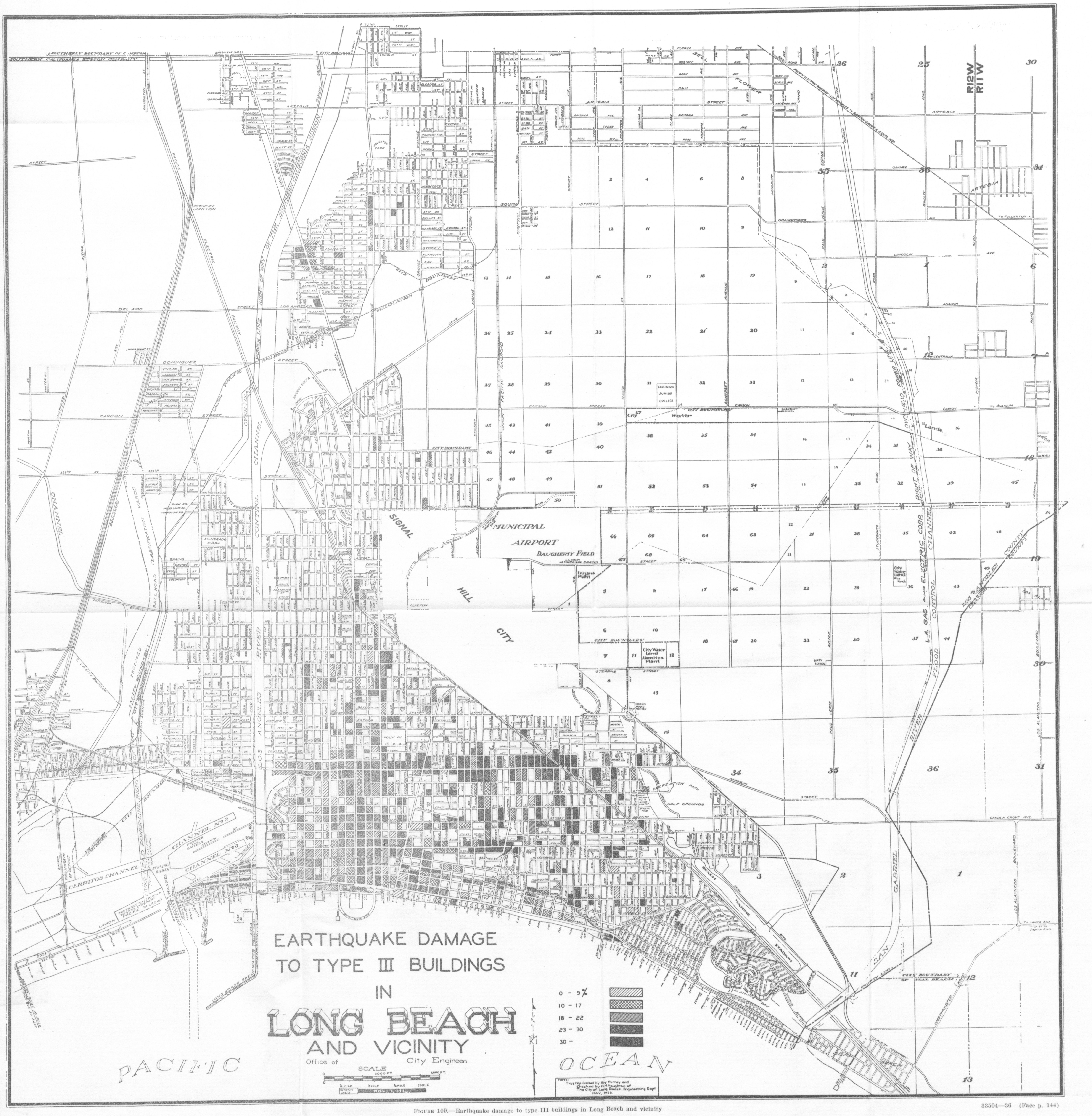


FIGURE 109.—Earthquake damage to type III buildings in Long Beach and vicinity

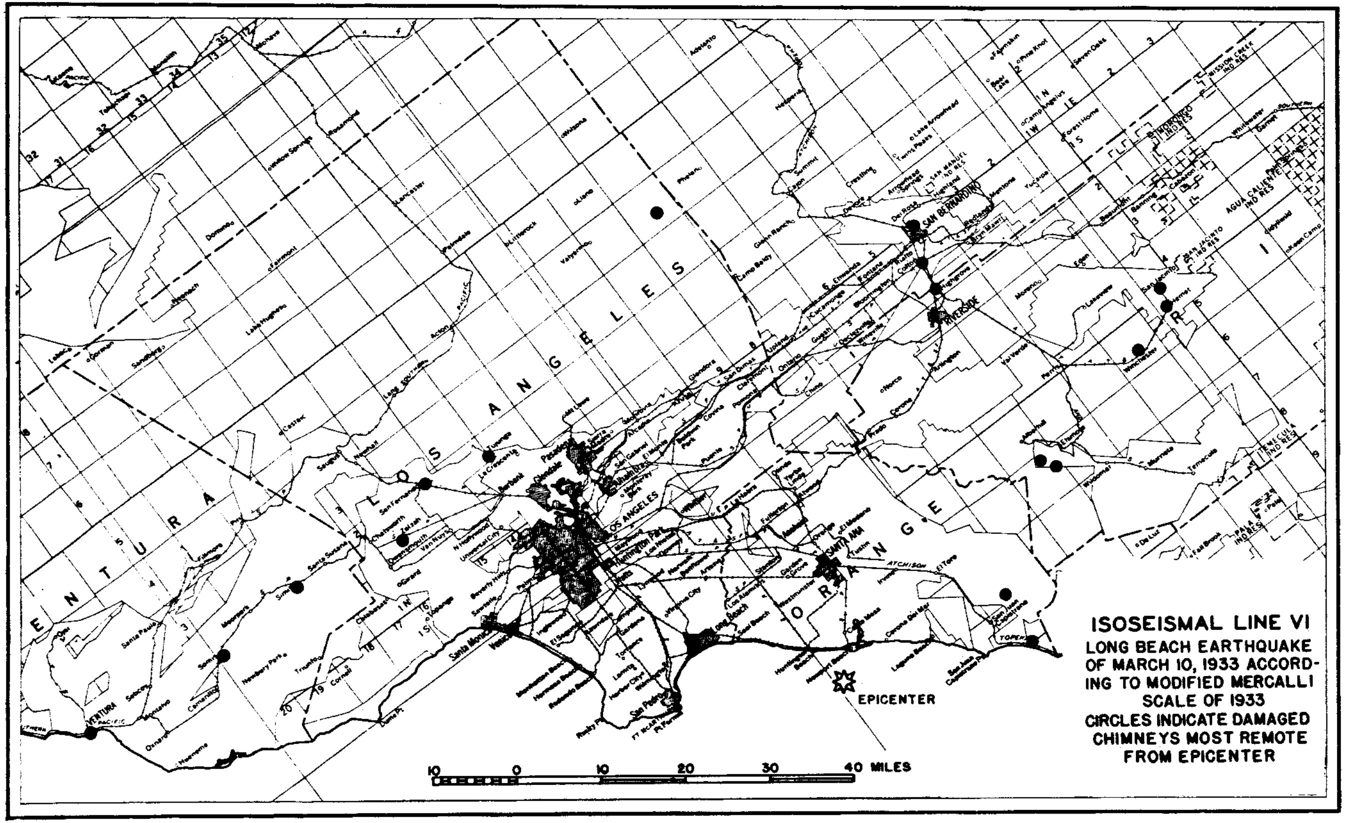


FIGURE 110.—Limits of chimney damage in Long Beach earthquake.

in intensity. However, the opinion of those who prepared the data and made the field surveys is that, in general, appreciable differences in intensity did exist and were related as shown on figure 109.

To convey an idea of the extent of the area strongly shaken by the earthquake and determine also if the shape of this area was materially influenced by underlying soil conditions, a field survey was carried

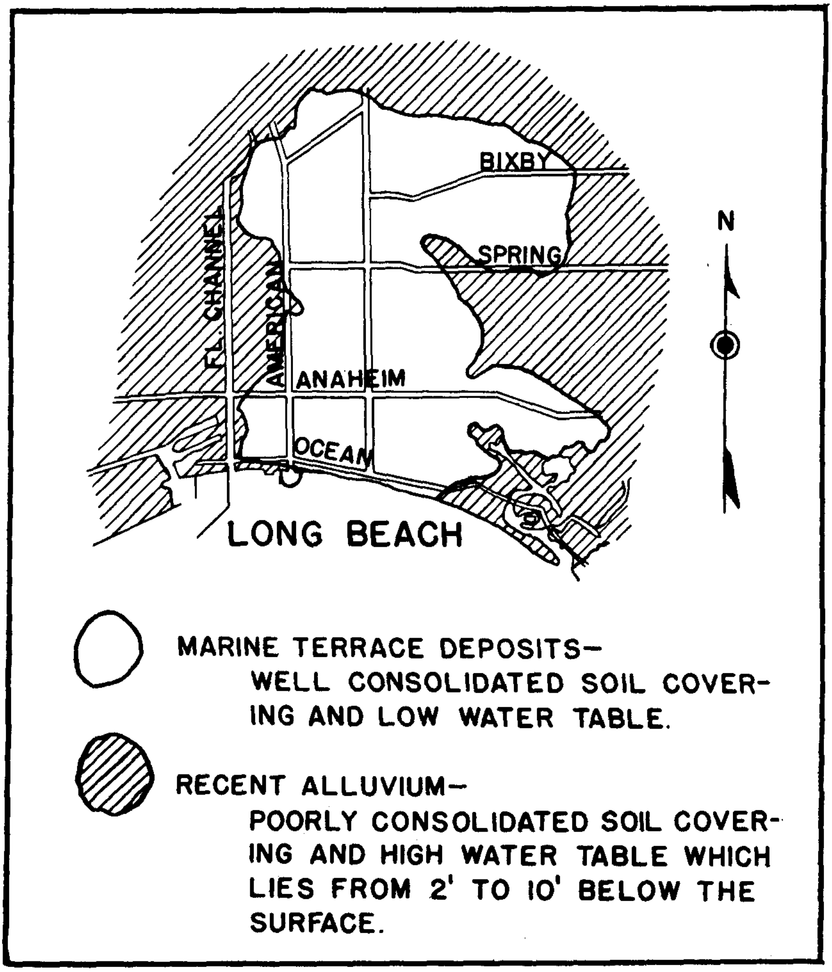


FIGURE 111.—Soil in Long Beach area.

out to locate the damaged chimneys most remote from the epicenter. A line connecting these chimneys would correspond to isoseismal VI on the modified Mercalli scale of 1931. The results of this survey are given in figure 110. It is of interest primarily because the area is larger and extends farther south and north than was popularly supposed.

To aid in the correlation of the damage shown in figures 109 and 110 with soil conditions, a sketch map, figure 111, has been prepared.[3]

[3] Based on a map in Bulletin 45 of the Division of Water Resources, Department of Public Works, State of California.

From this it will be observed that the damage to type III buildings located on the softer, more recently deposited alluvium with ground water at from 2 to 10 feet from the surface is somewhat less than to similar buildings on the slightly older, firmer alluvium with ground water not so close to the surface. Areas of lesser damage are found both at the west end of the city along the old river bed and in the southeast section bordering on Alamitos Bay. Damage along the beach, below the bluff, was also less than on the higher ground.

In the second part of the investigation the brick type III buildings were segregated into groups as follows: First, they were divided as to height measured by the number of stories and indicated by the first number in the key designations. Second, a division was made based partly on the amount of openings in the walls and partly upon the building's size. The letters in the key designate these subtypes. Third, a further classification was made based upon the manner of supporting the roof if the building was one story high or of supporting the upper stories in buildings of two stories or over. The last figure in the key describes this classification. (See summary of key designations below.)

The buildings were further classified according to their use, viz, into garages, stores, apartments, theaters, churches, and residences.

For each group in table 23 of the summary sheet the total assessed value was found, and then the total reductions in assessments and their ratios in percents were tabulated. The summary sheet reads across for subtype classifications and down for the three use or occupancy classifications subject to subtype divisions. The percent figures are, of course, only averages of quite widely divergent individual conditions, and unless the number of buildings is fairly large they are of doubtful significance. By the very nature of the data, they are at best only valuable as a yardstick in making comparisons.

It should be remembered in using the figures for comparison that assessed values of improvements include not only the buildings but their plumbing, wiring, interior decoration, etc. Thus, where the chief damage was to the masonry exterior walls a garage and an apartment house might have appeared to be equally damaged, but the percentage relationship would by no means be the same since in an ordinary garage the walls constitute a very much higher proportion of the total assessed value than do the walls of an apartment house. This is only one example, but the point is important and must not be overlooked.

Summary of key designations.—The first figure indicates the height of the building in stories. The letter shows the building subtype, as in figure 112. The last figure refers to the first-story construction and denotes:

1. Simple span joists spanning between the exterior walls.
2. Interior bearing partitions.
3. Interior posts and girders.
4. Roof trusses. (Applied to one-story buildings only.)

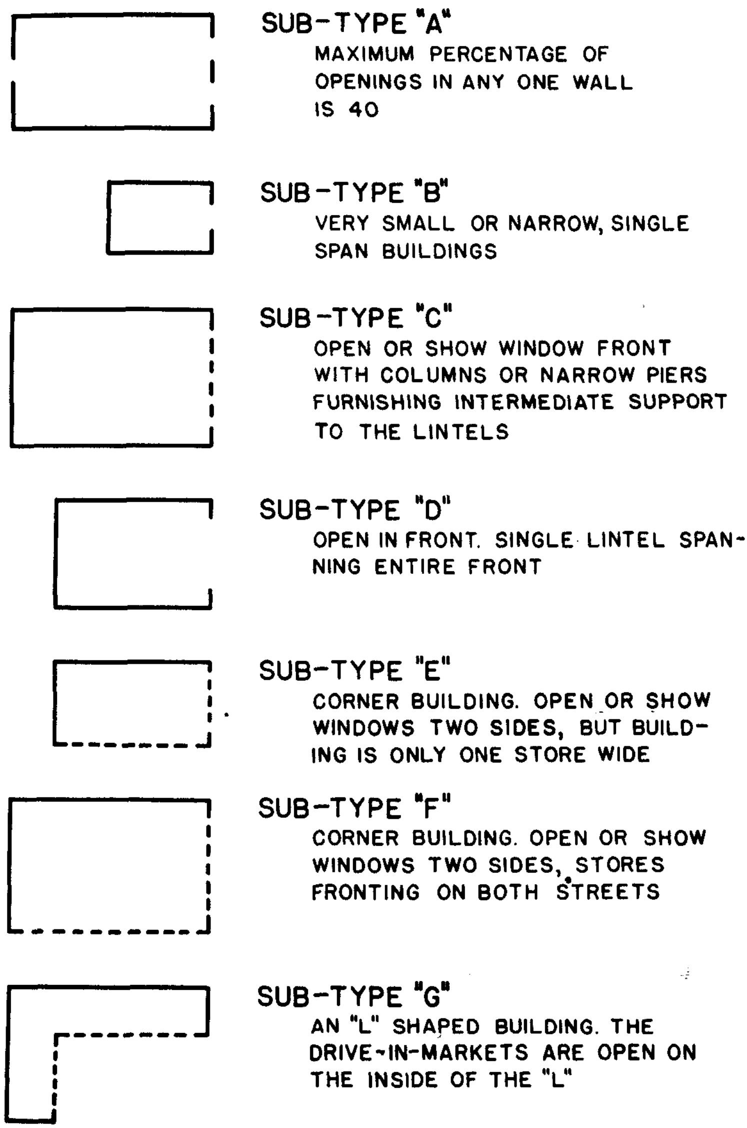


FIGURE 112.—Key to building sub-types.

TABLE 23.—*Summary sheet of weighted averages*

Subtype	Garages: Percent	Garages: Number	Stores: Percent	Stores: Number	Apartments: Percent	Apartments: Number	Mean percent	Number of buildings: Total	Number of buildings: Razed	Assessed value
1-STORY BUILDINGS[1]										
1 A 2					20	5	20	5		$20,750
1 A 4	27	76	25	9			27	85	6	333,780
1 B 1	22	22	24	23			23	45	5	81,830
1 C 2			21	128			21	128	4	397,320
1 C 3			21	86			21	86	4	268,790
1 C 4	24	88	20	56			22	144	8	595,530
1 D 4	18	2	14	15			15	17		68,110
1 E 1			26	27			26	27	1	64,700
1 E 4	35	15	27	10			33	25		99,590
1 F 2			21	21			21	21		120,040
1 F 3			21	14			21	14		82,330
1 F 4	20	10					20	10		90,500
1 G 2, 3	20	5	24	2			21	7		16,730
1 G 4	26	6	22[2]	8			24	14	1	81,450
Total 1	25	224	21	399	20	5	23	628	29	2,316,950
2-STORY BUILDINGS[3]										
2 A 2					23	57	23	57	4	$370,810
2 B 1			21	9			21	9		30,240
2 C 2			20	57			20	57	2	327,710
2 C 3			20	90			20	90	2	628,360
2 E 1			26	7			26	7		35,630
2 F 2			21	13			21	13		142,090
2 F 3			23	23			23	23	1	277,850
Total 2			21	199	23	57	21	256	9	1,812,690
3-STORY BUILDINGS[4]										
3 A 2					16	38	16	38		$541,280
3 C 2			18	14			18	14		164,200
3 C 3			15	20			15	20	1	291,320
3 F 2			21	7			21	7		85,460
3 F 3			10	9			10	9	1	116,700
Total 3			16	50	16	38	16	88	2	1,198,960
4-STORY BUILDINGS[5]										
4 A 2					12	12	12	12		$337,400
4 C 2			16	3			16	3		54,600
4 C 3			16	2			16	2		31,700
4 F 2										
4 F 3			4	2			4	2		56,500
Total 4			11	7	12	12	12	19		480,200
5-STORY BUILDINGS										
5 A 2					6	1	6	1		$38,900
6-STORY BUILDINGS										
6 A 2					6	1	6	1		$65,000

[1] Assessed value of garages, 1-story buildings: $931,440. Assessed value of stores, 1-story buildings: $1,384,760.

[2] Drive-in markets.

[3] Assessed value of stores, 2-story buildings: $1,441,880.

[4] Assessed value of stores, 3-story buildings: $657,680. Assessed value of apartments, 3-story buildings: $541,280.

[5] Assessed value of stores, 4-story buildings: $142,800. Assessed value of apartments, 4-story buildings: $337,400.

TABLE 23.—*Summary sheet of weighted averages*—Continued

SUMMARY FOR 1-, 2-, 3-, 4-, 5-, AND 6-STORY BUILDINGS [6]

Subtype	Garages		Stores		Apartments		Mean per-cent	Number of buildings		Assessed value
	Per-cent	Num-ber	Per-cent	Num-ber	Per-cent	Num-ber		Total	Razed	
Totals for above types	25	224	20	655	16	114	20	993	40	$5, 912, 700
MISCELLANEOUS TYPES										
Miscellaneous							17	146	3	$2, 030, 680
Theaters							21	19	1	275, 810
Churches							24	20	3	524, 160
Residences:										
Brick							24	54	10	180, 800
Tile							19	32	1	51, 040
Total							19	271	18	3, 062, 490
ALL TYPES										
Grand totals							20	1, 264	58	8, 975, 190

[6] Assessed value of garages, 1- to 6-story buildings: $931, 440. Assessed value of stores, 1- to 6-story buildings: $3,607, 120. Assessed value of apartments, 1- to 6-story buildings: $1,374,140.

TABLE 24.—*Summary sheet of simple averages*

1-STORY BUILDINGS

Subtype	Garages		Stores		Apartments		Mean per-cent	Number of buildings		Assessed value
	Per-cent	Num-ber	Per-cent	Num-ber	Per-cent	Num-ber		Total	Razed	
1 A 2					18	5	18	5		
1 A 4	28	76	28	9			28	85	6	
1 B 1	30	22	21	23			25	45	5	
1 C 2			21	128			21	128	4	
1 C 3			25	86			25	86	4	
1 C 4	26	88	25	56			25	144	8	
1 D 4	17	2	14	15			14	17		
1 E 1			27	27			27	27	1	
1 E 4	30	15	25	10			28	25		
1 F 2			21	21			21	21		
1 F 3			26	14			26	14		
1 F 4	22	10					22	10		
1 G 2, 3	20	5	26	2			22	7		
1 G 4	26	6	22	8			24	14	1	
Total 1	27	224	23	399	18	5	24	628	29	
2-STORY BUILDINGS										
2 A 2					23	57	23	57	4	
2 B 1			26	9			26	9		
2 C 2			22	57			22	57	2	
2 C 3			26	90			26	90	2	
2 E 1			25	7			25	7		
2 F 2			27	13			27	13		
2 F 3			24	23			24	23	1	
Total 2			26	199	23	57	25	256	9	

TABLE 24.—*Summary sheet of simple averages*—Continued

3-STORY BUILDINGS

Subtype	Garages		Stores		Apartments		Mean per cent	Number of buildings		Assessed value
	Per-cent	Num-ber	Per-cent	Num-ber	Per-cent	Num-ber		Total	Razed	
3 A 2					20	38	20	38		
3 C 2			18	14			18	14		
3 C 3			21	20			21	20	1	
3 F 2			24	7			24	7		
3 F 3			17	9			17	9	1	
Total 3			20	50	20	38	20	88	2	
4-STORY BUILDINGS										
4 A 2					14	12	14	12		
4 C 2			15	3			15	3		
4 C 3			15	2			15	2		
4 F 3			4	2			4	2		
Total 4			12	7	14	12	13	19		
5-STORY BUILDINGS										
5 A 2					6	1	6	1		
6-STORY BUILDINGS										
6 A 2					6	1	6	1		
SUMMARY FOR 1-, 2-, 3-, 4-, 5-, AND 6-STORY BUILDINGS										
Totals for above t_ pes	27	224	23	655	20	114	24	993	40	
MISCELLANEOUS TYPES										
Miscellaneous							20	146	3	
Theaters							22	19	1	
Churches							26	20	3	
Residences:										
Brick							30	54	10	
Tile							23	32	1	
Total							23	271	18	
ALL TYPES										
Grand totals							24	1,264	58	

In table 23, all the figures for subgroups and totals may be considered as weighted averages and were obtained as previously described. In table 24, the figures for subgroups and totals were obtained by adding the percentage damage for each of the buildings involved and dividing by the number of the buildings. By using simple averages, the values for percent of damage were generally higher than those found from the weighted averages, which reflects the influence of the lesser relative damage to the heavily assessed properties. Comparative trends, however, are generally in the same direction regardless of whether the averages were or were not weighted. In making up the diagrams and as the basis for subsequent discussions, the weighted averages have been used unless otherwise stated.

To avoid the possibility of forming an erroneous impression, it should be borne in mind that while the grand total average percentage reduction is only 19½ percent (table 23), this includes 14 buildings that collapsed during the earthquake, 58 buildings that were demolished or razed following the quake, and 305 buildings whose fronts or other parts were considered so dangerous following the quake that such dangerous parts were pulled down or removed by the building department crews. A graphical representation of the spread of the damage is given in figure 113.

In evaluating the significance of the statistical data obtained, certain qualifying conditions are to be noted. One-story buildings were quite well scattered over the area, 2-story buildings nearly as well, but the 3- and 4-story buildings were nearly all confined to one relatively small area, i. e., south of Seventh Street between Atlantic and Magnolia Avenues; this, as shown on the area map, was an area of less than average damage. In only two groups of any size did the

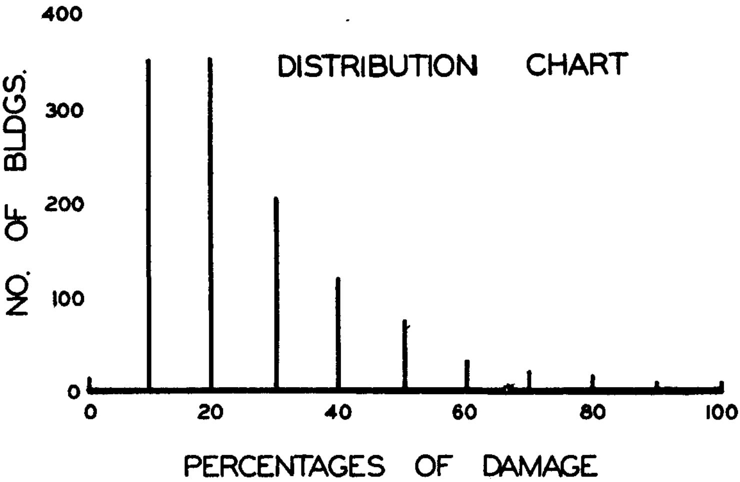


FIGURE 113.—Damage distribution chart.

age of the buildings seem to be similar enough to be important.[4] In the 1 D 4 store group (which is composed of one-story market buildings entirely open in the front, whose roofs are supported by trusses) and in the 1 G 4 store group (L-shaped drive-in markets), the buildings were much newer, the majority having been built about 1930–31.

Of the many comparisons that may be made from the data presented on the summary sheets, the following are of particular interest.

Story heights.—The percent of damage decreased consistently from 23 percent for all 1-story buildings, to 21 percent for 2-story buildings, to 16 percent for 3-story buildings, to 12 percent for 4-story buildings. The restriction of the comparison to store buildings, only eliminates the difference between 1- and 2-story buildings, but not that between the 2- and 3-story buildings. Considering apartments only, the downward trend is confirmed from 2- to 3- to 4-story buildings.

[4] For the influence of age on damage cf.: Earthquake Damage Analyzed by Long Beach Officials, by C. D. Wailes, Jr. and A. C. Horner, Engineering News Record, May 25, 1933.

For the purpose of determining if these results would hold for a smaller number of buildings concentrated in a few blocks, two areas were sampled. In each of 2 areas which were selected because they included several 2-, 3-, and 4-story buildings, the buildings were segregated as to story heights and the average damage percentage found for each height.

The first area selected was south of Seventh Street between Atlantic and Magnolia Avenues in which were located 30 percent of all the 1-story buildings, 48 percent of the 2-story, and 75 percent of all the 3- and 4-story buildings. The damage in this area was fairly uniform and slightly less than the average for the city as a whole.

Part A of the second area included blocks on both sides of East Broadway, from Alamitos to Esperanza Avenues. This area was heavier than average in damage.

Part B of the second area was bounded by Third Street and Fifth Street and by Elm Avenue and Alamitos Avenue. This likewise was an area of heavier than average damage. This second area was chosen because the damage in it appeared from field observation to differ markedly from the general trend.

The results of the tabulations are given below for each area. The fact that the trends for the small group of buildings in the second area do not agree with the trends for all the buildings in the city should probably be regarded more as a caution against the use of small numbers of buildings than as vitiating the conclusions obtained by considering the entire group, especially since the findings for the first area do agree with those for the entire group. The influence of height on damage can be seen in figure 114.

TABLE 25.—*Building damage by height for first and second areas*

Height	First area		Second area, parts A and B	
	Percent	Number	Percent	Number
Stories:				
1	19	192	19	12
2	19	124	28	6
3	12	69	34	5
4	10	14	19	1
Total	16	399	27	24

Subtypes.—The following is a comparison of the different percentages of damage to 1-story buildings of the various sub-types:

The 1 D 4 store group (open-front markets) shows an exceptionally low average percentage of damage—14 percent.

The 1 E 4 garage group (corner garages, open on two sides) sustained an exceptionally high average percentage of damage—35 percent.

The 1 E 1 store group (single-store corner buildings, open on two sides) shows higher damage—26 percent—than the store average of 21 percent.

The 1 A 4 garage group shows a higher percentage of damage—27 percent—than the 1 C 4 garage group—24 percent—even though the latter has more openings in the front.

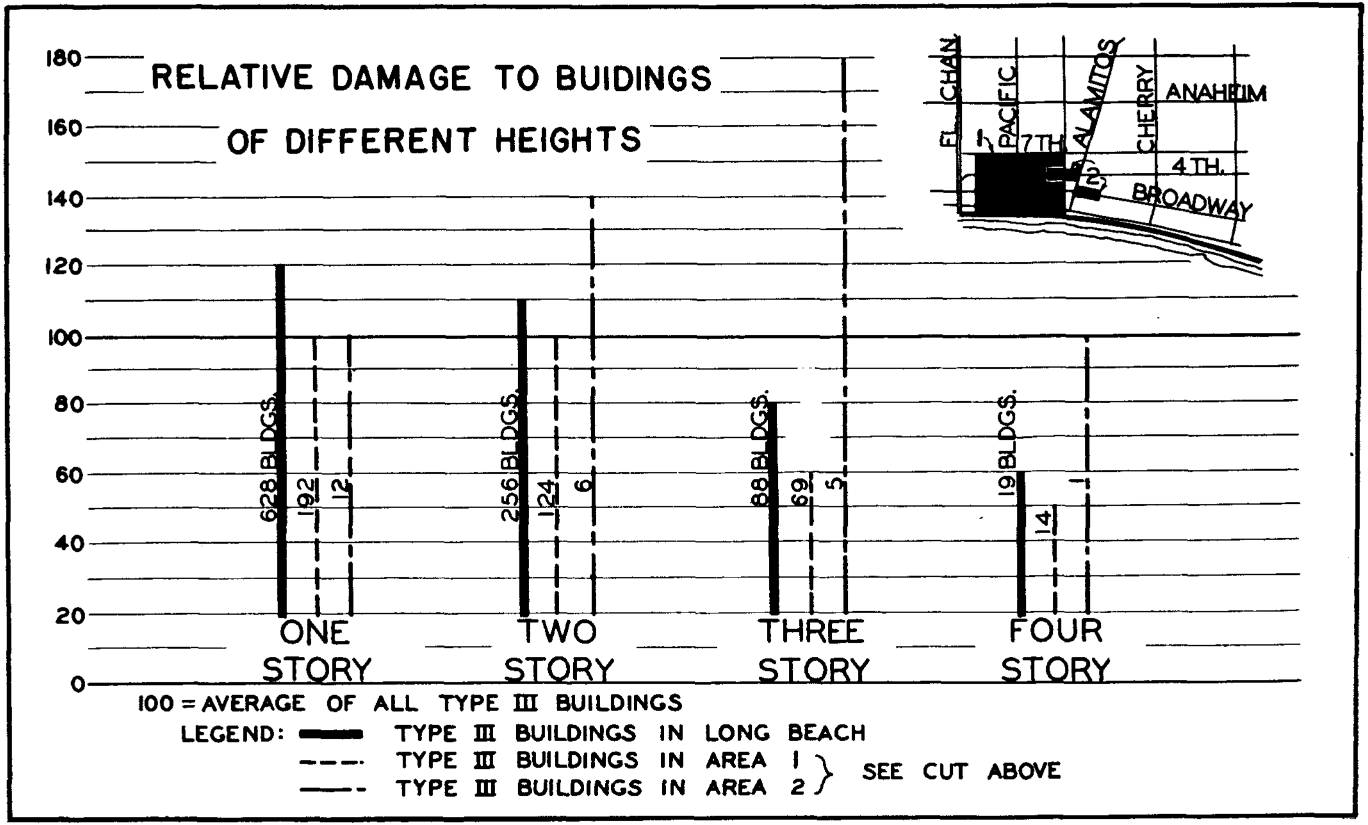


FIGURE 114.—Relative damage to buildings of different heights

The 1 G 4 store group (drive-in markets), while perhaps too small a group (eight buildings) to consider, does not vary appreciably from the general store average of 21 percent.

Based on the data in table 23, figures 115 to 118 have been drawn to represent the difference in damage according to subtypes.

Use or occupany classes.—One-story garages show higher damage—25 percent—than 1-story stores—21 percent. Differences between garages and stores for the same subtypes were, in general, small and consistent.

Apartments and stores have practically the same percentage of damage when considering buildings of the same story height.

The percentage of damage to theaters and residences is close to the one-story building average.

Miscellaneous.—There was less damage to tile residences than to brick residences. In the case of 2-story apartments the tile ones (5 buildings) seemed to have a slightly smaller percentage of damage than the brick—21 percent against 23 percent. This is a small difference, but is in the same direction as in the residence group.

Brick veneer 2-story apartment houses showed only 7 percent damage for 39 buildings. For 18 residences of this construction a 14-percent damage was found. These figures were obtained in the same way as for type III buildings with which they cannot fairly be compared, as the brick veneer buildings are more nearly frame construction. One reason for these low figures is that in many cases where the brick veneer came dangerously loose, or fell off, the veneer was not replaced, but was completely removed and the wood frame walls stuccoed at relatively low cost.

Adjacency.—To ascertain the effect of adjacency all the buildings were classified in three groups: Nonadjacent, or free-standing; one side adjacent; and two sides adjacent. The total assessed value and total reductions for each group were tabulated and their relation expressed in percent. The results are given in table 26. To obtain the simple averages shown in the second part of the table, the sum of the individual percentage figures for each of the buildings was divided by the number of the buildings in the group.

TABLE 26.—*Adjacency*

WEIGHTED AVERAGES

Height	Nonadjacent buildings		1 side adjacent buildings		2 sides adjacent buildings	
	Number	Percent	Number	Percent	Number	Percent
Stories:						
1	239	25	180	21	96	18
2	146	24	79	20	31	16
3 and 4	57	16	27	9	11	9
Total	442	22	286	17	138	16

SIMPLE AVERAGES

Height	Nonadjacent buildings		1 side adjacent buildings		2 sides adjacent buildings	
	Number	Percent	Number	Percent	Number	Percent
Stories:						
1	239	37	180	22	96	18
2	146	24	79	20	31	13
3 and 4	57	17	27	12	11	9
Total	442	30	286	21	138	16

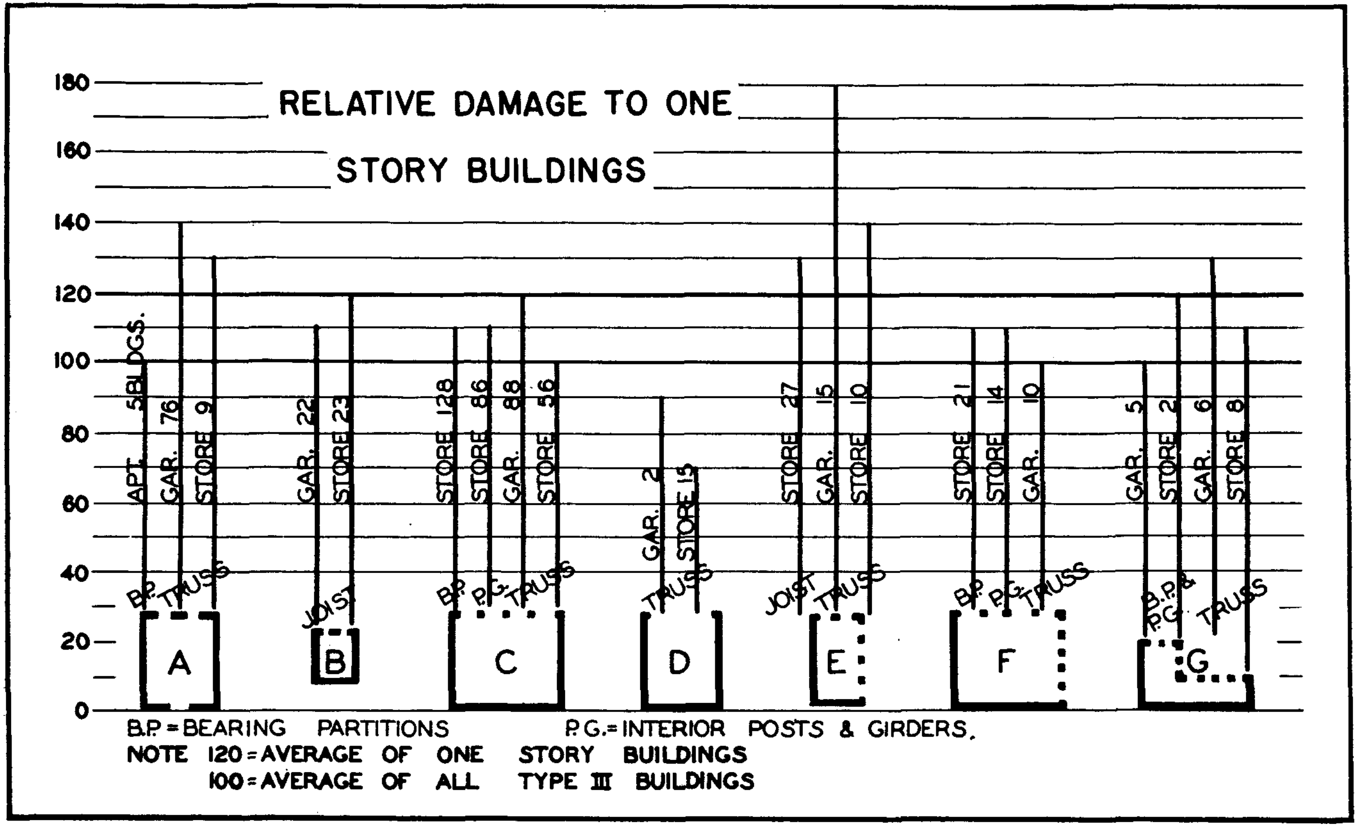


FIGURE 115.—Relative damage to one-story buildings.

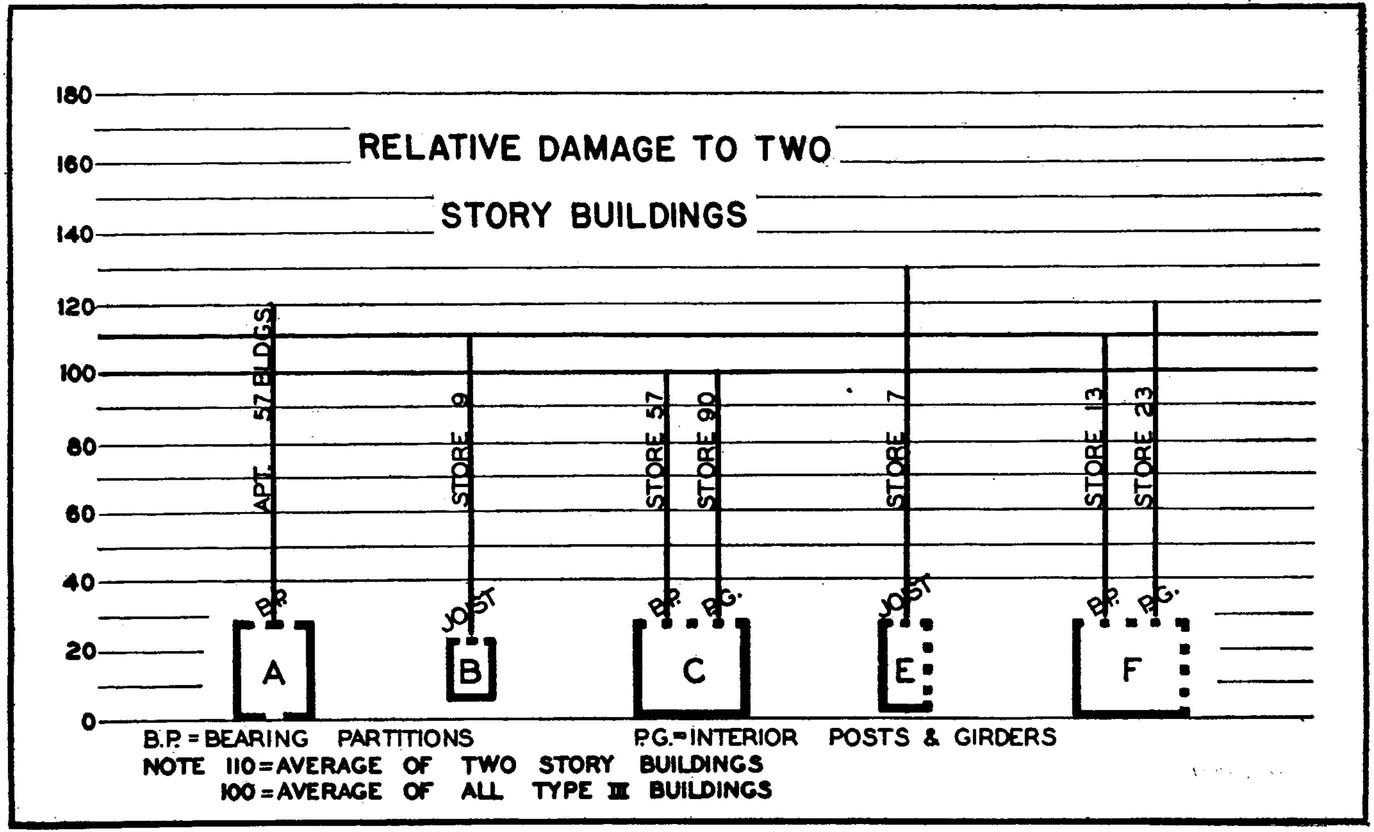


FIGURE 116.—Relative damage to two-story buildings.

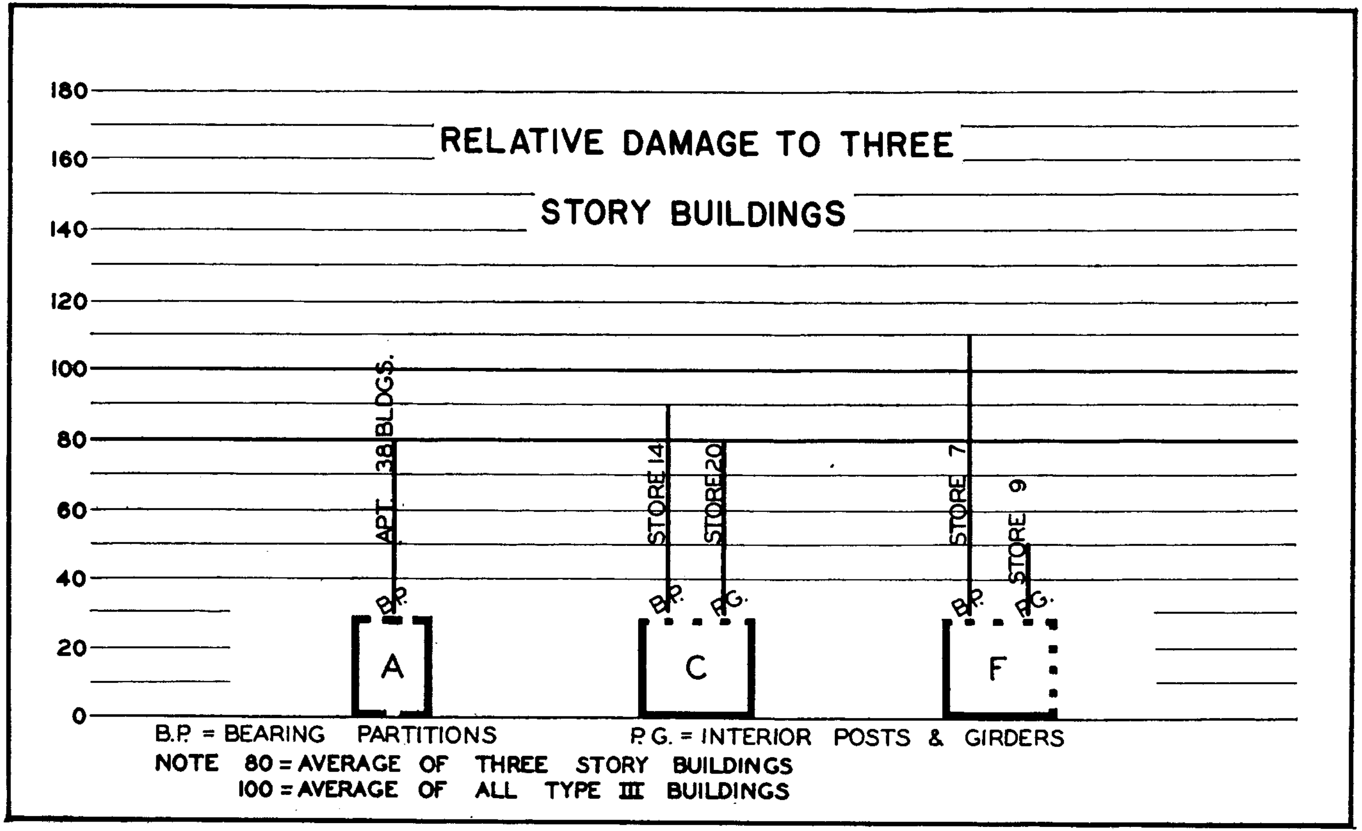


FIGURE 117.—Relative damage to three-story buildings.

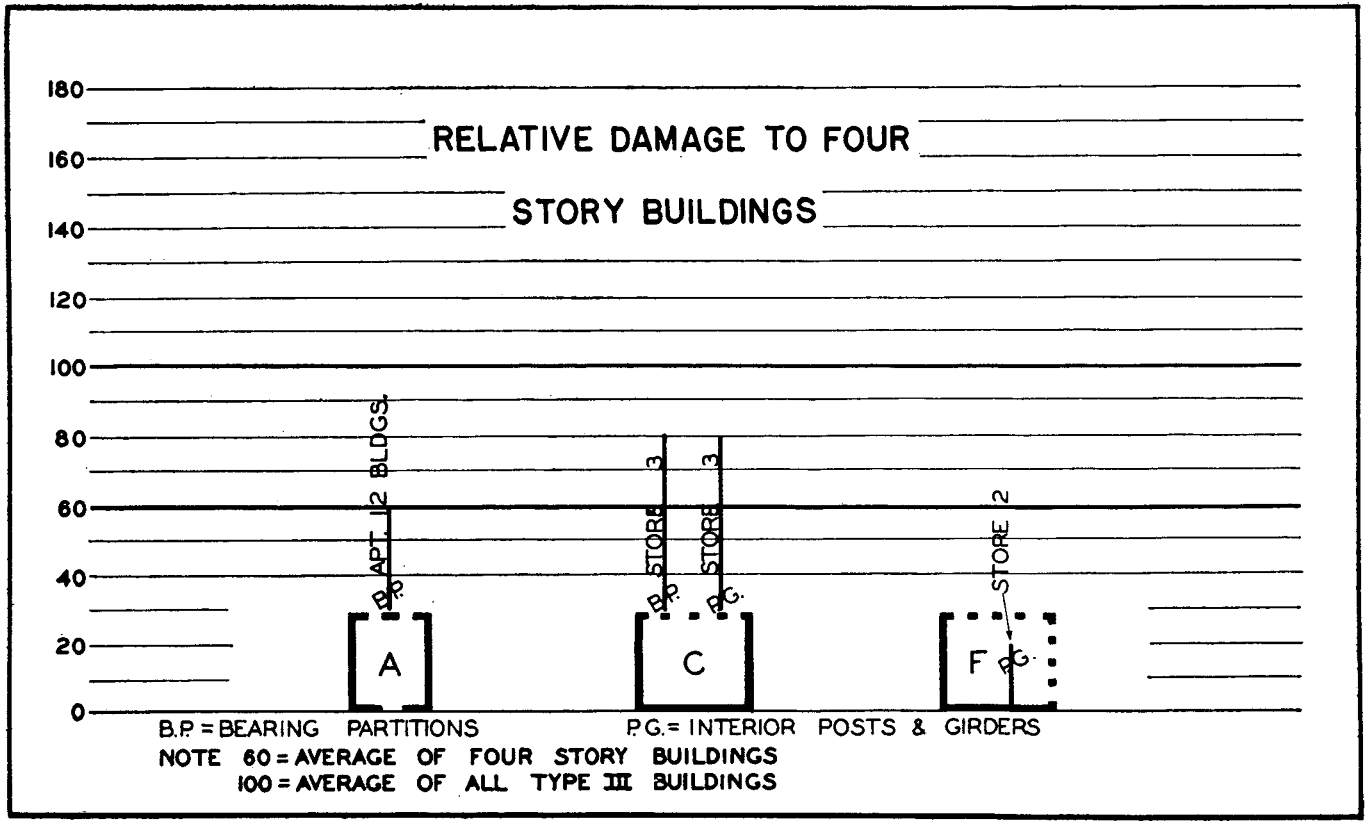


FIGURE 118.—Relative damage to four-story buildings.

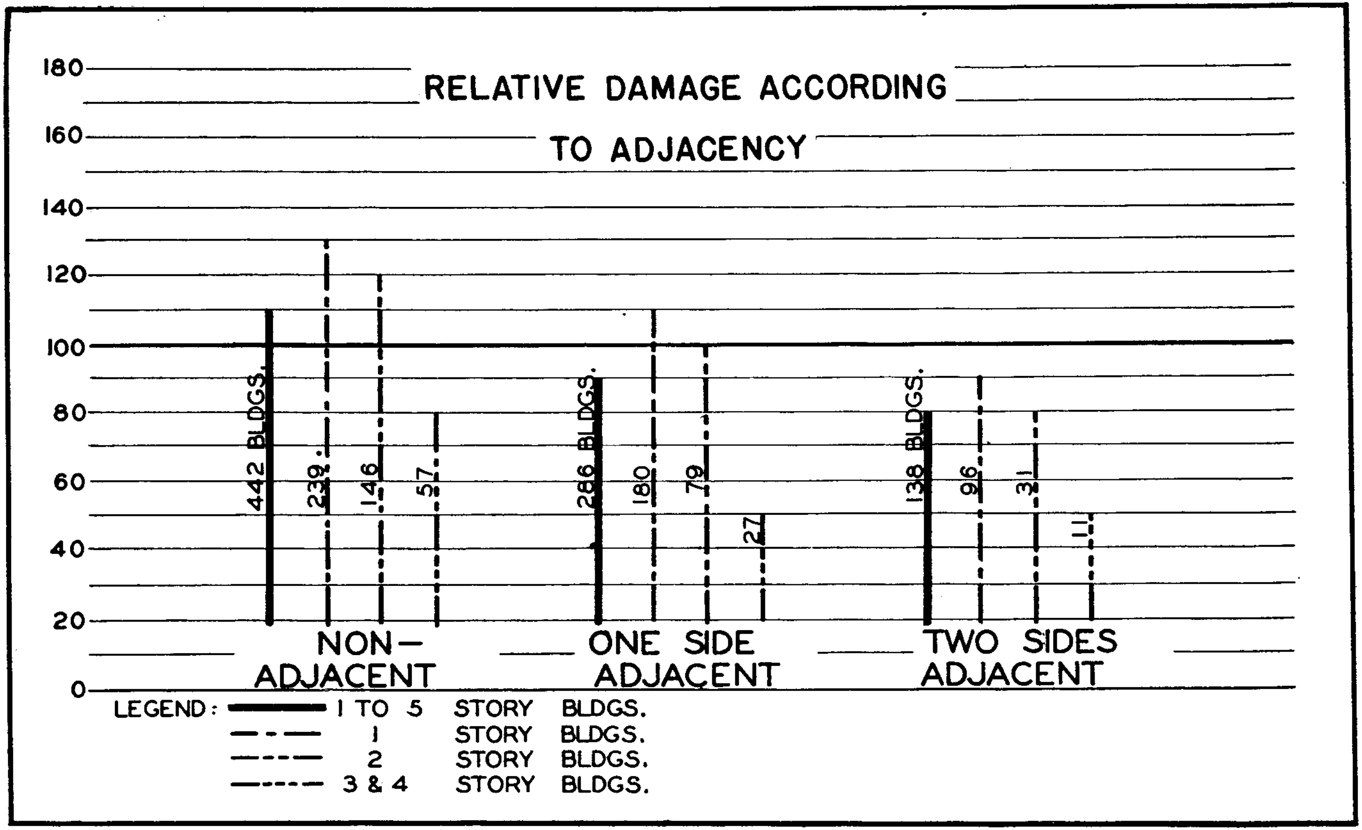


FIGURE 119.—Relative damage according to adjacency.

These results seem very definitely to indicate a trend toward lesser damage for adjacent buildings, and higher damage for free-standing buildings. Confirmation of this trend resulted from considering only the 1 C 2 and 1 C 3 stores, and again from using 2 C 2 and 2 C 3; i. e. using only buildings very much alike. The results are shown in figure 119.

GENERAL CONCLUSIONS

After making allowance for the fact that the basic data used were mainly of relative value and subject to errors, which rendered the averages of smaller numbers of buildings unreliable, and the further fact that the influence of any single one of the numerous causes on the extent of damage cannot be sharply segregated, the following general conclusions appear warranted:

1. The damage to buildings on soft, water-logged soil and on the beach was somewhat less than to those on more firmly consolidated soil.
2. The damage to buildings adjacent to others was less than to those that were isolated, and less for those with buildings adjacent on 2 sides than on 1 side.
3. The percentage of damage decreased with the number of stories.

ACKNOWLEDGMENTS

The generous cooperation of the city assessor and the city engineer of Long Beach in placing all the information in their possession at the disposal of the United States Coast and Geodetic Survey has proved invaluable in the conduct of this study.

Thanks are also due to Messrs. C. D. Wailes, Jr., J. H. Davies, C. J. Derrick, R. W. Binder, and John Sweeney, members of a committee of the Structural Engineers' Association of Southern California, for suggestions and criticisms.

SUPPLEMENTARY REPORT ON EARTHQUAKE DAMAGE IN COMPTON

The investigation of the March 10, 1933, earthquake damage in Compton was undertaken to determine the areal distribution and extent of damage.

The principal source of data, the building department of the county assessor's office, yielded assessed values and reductions in assessed value due to earthquake damage, when granted, for all the buildings.[1] This information was supplemented and checked by use of Compton city building permits and by field surveys.

The results of considering the damage percentage for wood-frame residences (type V) as to location indicated that a central area, several blocks wide and extending north and south to the city limits, received slightly higher damage than either the east or west sides of Compton. However, since many old buildings of low value were in this area, the small increase in percentage damage of this area over the rest of the town does not definitely indicate much difference in intensity.

The extent of damage for wood-frame residences (type V) is very low; in fact in 95 percent of these buildings the damage was less than 5 percent. The complete results are tabulated below:

[1] For detailed explanation see the Long Beach investigation report.

TABLE 27.—*Damage to type V buildings in Compton. Wood-frame residences*

Damage (percent)	Number of buildings	Fraction of total number
		Percent
0-4	4,334	94.7
5-24	131	2.9
25-49	63	1.4
50 and more	36	0.8
Demolished	11	0.2
Total	4,575	100.0

The masonry bearing wall buildings (type III or class C) were too few in number and too concentrated in location to justify consideration as to differences in intensity within Compton city limits.

The extent of damage for type III buildings with commercial buildings and brick residences considered separately shows that the brick residences suffered much less damage than brick commercial buildings. Thus, while 47 percent of the brick residences were damaged less than 5 percent, over 50 percent of the type III commercial buildings were demolished. The complete results are tabulated below:

TABLE 28.—*Damage to type III buildings in Compton. Masonry bearing walls*

Damage (percent)	Commercial		Residential	
	Number of buildings	Fraction of total number	Number of buildings	Fraction of total number
		Percent		*Percent*
0-4	2	2	13	47
5-24	5	4	3	16
25-49	26	21	4	27
50-75	25	20	1	10
Demolished	64	53		
Total	122	100	21	100

General conclusions.—The wood-frame type V buildings suffered very little damage—less than in Long Beach—while type III commerical buildings suffered heavy damage—more than in Long Beach.

Chapter 9.—PERIODS OF THE GROUND IN SOUTHERN CALIFORNIA EARTHQUAKES

B. Gutenberg

HISTORY

From the early seismic records various Japanese scientists found that the microseisms which are produced by traffic, industry, and meteorological conditions, as storms and surf, show certain prevailing periods which are different in different localities. Omori and Kikuchi [1] have especially investigated this problem. Kikuchi expressed his opinion that the prevailing periods constituted the free periods of the ground and their harmonics. The problem of the connection between periods of microseisms produced by local causes and the free periods of the ground has been investigated frequently since. O. Geussenhainer [2] in his investigation of microseisms in Göttingen with periods between 5 and 9 seconds found that they change gradually in the course of time in the vertical component, but that in the horizontal component periods of 6, 7.5, and 9 seconds prevail. He believed these periods to be free periods of Love waves in the ground near Göttingen or harmonics of such.

One of the first who investigated the connection between periods in earthquakes and free periods of the ground was A. Imamura.[3] He found that seismograms may involve, beside seismic waves propagated from the focus to a given station, vibrations induced at the station. From an examination of the seismograms due to the same earthquake and obtained at various seismological stations in the Kwanto district, he found for the induced earth vibrations the following periods:

	Seconds
Station on soil	0. 2–0. 3
Station on alluvium	0. 4–0. 5
Station on tertiary	0. 6–0. 7

The results of Imamura caused K. Sezawa, G. Nishimura, and K. Kanai [4] to investigate thoroughly the possibility of free oscillations of the ground excited by seismic waves.

The importance of such investigations concerning the engineering problems in erecting buildings was recognized relatively early. There are many publications in which the idea has been stressed that in

[1] F. Omori, Observations of Earthquakes at Hitotsubashi, 1900 (Publications of the Earthquake Investigation Committee in foreign languages, 13, 1903).
F. Omori, Tilting of the Ground During a Storm (Bulletin of the Imperial Earthquake Investigation Committee, I, Tokyo, 1907).
Baron Dairoku Kikuchi, Recent *Seismological Investigations (Publications of the Earthquake Investigation Committee* in foreign languages, 19, Tokyo, 1904).

[2] O. Geussenhainer, Ein Beitrag zum Studium der Bodenunruhe . . . ; Diss. Göttingen 1921, Auszug im Jahrb. d. philos. Fakultät *Göttingen* 1921, Nr. 18, Geophysik, p. 73.

[3] A. Imamura, Earth Vibrations Induced in Some Localities at the Arrival of Seismic Waves (Bulletin of the Earthquake Research Institute, *Tokyo, vol. VII, 1929, p. 493*).

[4] K. Sezawa, Possiblity of the Free Oscillations of the Surface Layer Excited by the Seismic Waves (Bulletin of the Earthquake Research Institute, Tokyo, vol. VIII, 1930, p. 1).
K. Sezawa and G. Nishimura, Dispersion of a Shock in Echoing and Dispersive Elastic Bodies (Bulletin of the Earthquake Research Institute, Tokyo, vol. VIII, 1930, p. 321.)
K. Sezawa and K. Kanai, Possibility of the Free Oscillation of the Surface Layer Excited by the Seismic Waves, Part III.(Bulletin of the Earthquake Research Institute, Tokyo, vol. X, 1932, p. 1).

earthquake regions buildings should not have the same free periods as the ground. The high value of investigations concerning free periods of the ground has been clearly recognized by many scientists. For example, K. Suyehiro[5] wrote that the periods of earthquake motions play an important role in the destructive effect of an earthquake on buildings and other structures, and that it is important, therefore, to investigate the periods of habitual motion peculiar to the ground if such motion does really exist at ordinary times and during earthquakes. When we remember that the ground is made up of several strata, it is not difficult to understand the existence of a period of motion peculiar to a given district. For detecting the prevailing periods of earthquakes in a particular locality Suyehiro has devised and used a seismic vibration analyzer.[6] It consists of a number of compound pendulums having different natural periods, the shortest being 0.2 second and the longest 1.8 seconds, which are arranged side by side in a row along the side of a recording drum. With this instrument Suyehiro found that Hongo on the high ground of Tokyo has a habitual motion with a period of about 0.3 second both at ordinary times and during earthquakes. At Marunouchi, on the low ground of Tokyo, the analyzer indicated that the prevailing periods in earthquakes are generally 0.7 to 0.9 second, but that secondary and tertiary free motions of this district with smaller periods may be excited by minor but sharp earthquakes.

Detailed investigations on the periods prevailing in the neighborhood of Tokyo have been published by M. Ishimoto.[7] He gives periods recorded during the maximum phase of four earthquakes at nine different stations. In most cases the periods observed most frequently are between 0.3 and 0.5 second, but there is some difference between the prevailing periods due to the source as well as to the stations. For example, in the third earthquake investigated by him periods of 0.2 second prevail at practically all the stations. On the other hand at a few stations there are maxima in the neighborhood of 0.7 second.

Other investigations concerning the periods observed during earthquakes at Marunouchi (downtown in Tokyo) have been carried out by T. Saito and M. Suzuki.[8] They have found there that in the alluvial layer there are numerous waves having periods of about 0.2 and 0.7 second and in diluvial formations 0.25 second and 0.45 second. They tried to show that 0.25 second and 0.75 second are the free periods of the alluvial layer.

Thorough investigations concerning free periods of the ground have been carried out by Dr. R. Köhler.[9] He measured the periods during different phases in earthquake records and compared them with periods produced by vibrations of a large engine which was run with different frequencies. The records of the movements of the ground taken during these experiments showed clearly an increase in

[5] K. Suyehiro, Engineering Seismology, Notes on American Lectures (Proceedings of the American Society of Civil Engineers, vol. 58, no. 4, 1932, p 9).

[6] K. Suyehiro, A Seismic Vibration Analyzer and the Records Obtained Therewith (Bulletin of the Earthquake Research Institute, Tokyo, vol. I, 1926, p. 59).

[7] M. Ishimoto, Observations accélérometriques des secousses sismiques dans les villes de Tokyo et Yokohama (premier rapport) (Bulletin of the Earthquake Research Institute, Tokyo, vol. XII, 1934, p. 234).

[8] T. Saito and M. Suzuki, On the Upper Surface and Undergound Seismic Disturbances at the Downtown in Tokyo (Bulletin of the Earthquake Research Institute, Tokyo, vol. XII, 1934, p. 517).

[9] R. Köhler, Eigenschwingungen des Untergrundes, ihre Anregung und ihre seismische Bedeutung (Nachrichten von der Gesellschaft der Wissenschaften zu Göttingen, Math.-Phys. Klasse, Fachgruppe II, Neue Folge, Bd. 1, Nr. 2, 1934, p. 11).

amplitude when the engine was running with a certain period and decrease in amplitudes as soon as the period was either increased or decreased. Thus he found that the free periods of the ground corresponding to the maximum amplitudes in these records were between 0.30 and 0.35 second in the valley of the Leine near Göttingen. The same periods prevail in the records of earthquakes recorded at the Seismic Observatory of Göttingen from distances less than 300 kilometers. This result makes it very probable that the frequencies prevailing in earthquake records are largely influenced by the free vibrations of the ground at the station. Sinusoidal waves of quarry blasts recorded in Göttingen show only periods of 0.3 to 0.4 second.

At distances over 300 kilometers longer periods are recorded in the transverse waves and during the maxima of earthquake waves, while in the P-phase of earthquakes recorded in Göttingen the periods of 0.3 to 0.4 second prevail up to distances of about 1,000 kilometers; for all phases at distances over 1,000 kilometers periods of 1.2 seconds are more prevalent. Even at very great distances such periods are not infrequent there. Dr. Köhler, therefore, believes in the possibility that the period of 1.2 seconds is the period of the free vibration of the uppermost crustal layer at Göttingen and that the periods of 0.3 and 0.4 second are harmonics of it, but he considers also the possibility that the period of 0.3 second is the free period of the uppermost layer, while the period of 1.2 seconds is the free period of a thicker crustal layer. The thickness of the first layer would be of the order of 2 kilometers. He tries to explain the fact that the prevailing period at Stuttgart is about 0.2 to 0.3 second, while at Ravensburg it is 0.5 to 0.6 second by the increase in thickness of the surface layer in approaching the Alps. At Ravensburg the thickness of the surface layer is estimated to be about 4 kilometers. Köhler also gives some data for other stations. In Jena, for example, periods between 0.26 and 0.43 second are especially frequent. At Potsdam periods in the neighborhood of 0.41 second prevail. Microseisms at places west of Potsdam show periods of sinusoidal waves between 0.35 and 0.37 second. At Zürich the prevailing period is 0.6 second, with a small secondary maximum at 1.2 seconds. Periods under 0.5 second are recorded there very infrequently. The period of 1.2 seconds which has been mentioned previously in discussing the periods at Göttingen is also frequently recorded at Jena and Pulkovo.

DATA

The fact that at Pasadena and its six auxiliary stations the same type of instruments are operated offers an unusual opportunity to compare periods recorded during different earthquakes at several localities. The constants of these stations are the following:

Pasadena:
Latitude=34°08.9′ N.; longitude=118°10.3′ W.; h=295 m.; deeply weathered granite rock, with inclusions of gneiss and schist.

Mount Wilson Seismologic Station:
Latitude=34°13.5′ N.; longitude=118°03.4′ W.; h=1,742 m.; weathered granite.

Riverside Seismologic Station:
Latitude=33°59.6′ N.; longitude=117°22.5′ W.; h=250 m., approx.; weathered granite.

Santa Barbara Seismologic Station:
Latitude=34°26.5′ N.; longitude=119°42.9′ W.; h=100 m., approx.; heavy, boulder-laden alluvium.

La Jolla (Scripps Institution Seismologic Station):
Latitude=32°51.8′ N.; longitude=117°15.2′ W.; h=7.7 m., approx.; consolidated detrital material.
Tinemaha Seismologic Station:
Latitude=37°05.7′ N.; longitude=118°15.5′ W.; h=1,180 m., approx.; basalt.
Haiwee Seismologic Station:
Latitude=36°08.2′ N.; longitude=117°57.9′ W.; h=1,100 m., approx.; loosely cemented tuff.

It is a well-known fact that in the case of continuous waves of a sinusoidal type the instruments have very different (dynamic) magnification for waves with different periods. For this reason, in the case of a wave train containing waves with different frequencies, the instruments accentuate those waves with periods for which their magnification is large. First, therefore, we have to consider the properties of the instruments used in this investigation.

Originally all stations had two Wood-Anderson horizontal component torsion seismometers with electromagnetic damping and optical recording (Cf. Bull. Seis. Soc. Am., XV, 1, 1925) with free periods of somewhat less than 1 second, static magnification between 2,500 and 3,000, and almost critical damping. During recent years one Benioff vertical component seismometer with galvanometric-optical recording has been added at each station. This seismometer has an inertia-mass of 100 kilograms, a free period between 0.5 and 1 second, about critical damping, and a galvanometer with a free period of about 0.2 second. The torsion seismometers have their maximum magnification for continuous sinusoidal waves with periods less than 0.5 second. In the case of sinusoidal waves with longer periods the magnification decreases and reaches about half the maximum value with periods of 1 second and about one-tenth with periods of 3 seconds. The Benioff vertical combination mentioned above gives its maximum magnification for sinusoidal waves with periods between 0.1 and 0.4 second. The magnification drops down somewhat faster than it does in the case of the torsion instruments and is about one-tenth of the maximum magnification for waves with periods of 1 second. Beside these instruments, at Pasadena short-period horizontal instruments with slightly different properties have been used temporarily. Examples of these are the horizontal instruments of the Benioff type with short-period galvanometers and the Benioff short-period strain seismograph.[1]

At Pasadena the following group of long-period instruments have been in use: One or two torsion instruments with free periods of 6 seconds or more, Benioff horizontal and vertical instruments, and a long-period Strain seismograph. The maximum magnification of the long-period torsion seismometers occurs for all waves with periods less than 3 seconds. Most of the long period Benioff combinations have their maximum magnification for continuous sinusoidal waves with periods between 0.4 and 1 second. It is smaller for shorter and longer waves, being about one-half for waves with periods of 0.1 to 2 seconds.

For the present investigations the records of Pasadena have been separated into two groups, one containing the short-period instruments and the other containing all long-period instruments. Besides the instruments mentioned so far, the geophysical outfit of the Cali-

[1] H. Benioff, A Linear Strain Seismograph, Bulletin of the Seismological Society of America, vol. 25, 1935, p. 283.

fornia Institute of Technology was used during the days following the Long Beach earthquake to record aftershocks. It proved very clearly that waves with periods as small as 0.01 second occur during all phases of local shocks. As earthquakes are frequently accompanied by audible sound waves, there is no doubt that waves with even shorter periods are produced during an earthquake, but as these waves are of no interest in engineering problems we will not deal during these investigations with waves having periods less than 0.1 second. On the other hand, very long waves with periods of a large fraction of a minute have been found occasionally in records of close-by shocks. The study of these long waves also is omitted in the investigation.

TABLE 29.—*List of shocks*

No.	Date	Greenwich standard time	Epicenter	Distance in kilometers						
				Pasadena	Mount Wilson	Santa Barbara	La Jolla	Riverside	Tinemaha	Haiwee
		h. *m.*								
1	Feb. 11, 1932	23 11	San Bernardino Mountains	130	116	262	186	77	208	202
2	Jan. 16, 1930	00 24	do	122	112	264	142	37	350	242
3	do	00 34	do	122	112	264	142	37	350	242
4	June 1, 1931	08 30	Mohave Desert	165	154	300	240	114	300	203
5	Aug. 17, 1930	22 07	do	173	150	278	270	146	235	140
6	Jan. 8, 1931	13 53	do	137	------	253	------	109	262	155
7	Feb. 24, 1930	19 56	do	139	124	------	------	111	258	155
8	Apr. 20, 1930	08 52	do	117	103	245	197	75	286	181
9	Apr. 27, 1931	23 08	Twenty-nine Palms	186	170	320	186	110	------	249
10	Apr. 24, 1931	18 28	Redondo	49	63	132	153	108	366	------
11	Apr. 23, 1931	23 34	North of Barstow	178	144	250	302	178	179	70
12	Feb. 21, 1931	19 27	South of Bakersfield	145	------	124	------	204	205	------
13	Apr. 23, 1931	10 01	West of Parkfield	288	------	175	------	362	250	244
14	Aug. 18, 1930	13 09	West of Santa Barbara	188	140	45	324	267	320	279
15	Aug. 5, 1930	11 25	Near Santa Barbara	135	145	18	302	205	326	230
16	May 29, 1930	07 12	Panamint Valley	176	163	257	292	170	194	80
17	May 12, 1930	17 26	East of La Jolla	178	168	308	60	108	454	339
18	Apr. 29, 1931	12 41	Chatsworth	44	54	100	200	123	300	216
19	Jan. 17, 1931	08 08	Northeast of Bishop	381	------	381	524	403	59	162
20	Nov. 28, 1929	19 53	50 kilometers west of Bishop	360	346	320	520	350	64	180
21	Aug. 31, 1930	00 40	Santa Monica	46	40	113	176	107	354	247
22	Nov. 9, 1929	02 31	Parkfield	276	280	168	436	347	235	235
23	June 8, 1934	04 31	do	289	292	179	451	355	237	226
24	do	04 49	do	289	292	179	451	355	237	226
25	do	05 44	do	289	292	179	451	355	237	226
26	Oct. 2, 1934	20 32	San Francisco	557	558	440	------	560	380	440
27	do	20 22	do	557	558	440	------	560	380	440
28	Oct. 31, 1929	19 39	San Pedro Channel	54	60	165	123	86	378	274
29	Sept. 13, 1929	13 23	do	54	60	165	123	86	378	274
30	Feb. 26, 1930	00 42	Brawley	270	250	410	------	190	510	404
31	do	00 57	do	270	250	410	------	190	510	404
32	do	01 23	do	270	250	410	------	190	510	404
33	do	02 29	do	270	250	410	------	190	510	404
34	do	04 23	do	270	250	410	------	190	510	404
35	do	07 38	do	270	250	410	------	190	510	404
36	Mar. 1, 1930	23 44	do	------	------	420	------	200	510	410
37	Mar. 2, 1930	00 31	do	------	------	420	------	200	510	410
38	do	01 49	do	------	------	420	------	200	510	410
39	Nov. 28, 1929	19 50	50 kilometers west of Bishop	360	346	320	520	350	64	180
40	Sept. 26, 1929	20 00	East of Barstow	171	157	295	226	122	291	180
41	Jan. 2, 1931	01 53	West coast of Mexico	2,053	2,060	2,190	------	1,970	2,770	2,210
42	Apr. 9, 1933	04 02	do	2,220	2,220	2,340	2,150	2,150	2,540	2,420
43	do	21 07	do	2,220	2,220	2,340	2,150	2,150	2,540	2,420
44	Dec. 7, 1932	16 27	do	2,220	2,220	2,340	2,150	2,150	2,540	2,420
45	July 10, 1933	03 27	do	2,442	------	------	------	------	------	------

TABLE 29.—*List of shocks*—Continued

No.	Date	Greenwich standard time	Epicenter	Distance in kilometers: Pasadena	Mount Wilson	Santa Barbara	La Jolla	Riverside	Tinemaha	Haiwee
46	Jan. 4, 1933	21 13	Pacific Ocean	1,000	1,000	950	950	1,020	1,260	1,200
47	do	21 12	do	1,000	------	------	------	------	------	------
48	Aug. 16, 1931	11 43	Texas	1,355	1,355	1,505	------	1,285	1,455	1,400
49	Oct. 1, 1931	11 47	Gulf of California	555	555	680	390	170	870	730
50	July 12, 1932	19 27	do	1,221	1,221	1,365	1,060	1,150	1,540	1,440
51	Nov. 25, 1934	08 19	Lower California	272	274	388	107	224	575	465
52	July 7, 1932	16 17	do	777	------	890	610	720	1,065	955
53	May 26, 1929	22 44	Queen Charlotte Islands	2,209	2,209	2,070	2,370	2,280	------	------
54	Sept. 17, 1929	19 21	do	2,209	2,209	2,070	2,370	2,280	1,890	2,000
55	Mar. 11, 1933	01 55	Long Beach	60	68	183	106	70	380	276
56	do	06 59	do	60	68	183	106	70	380	276
57	Mar. 12, 1933	00 28	do	60	68	183	106	70	380	276
58	Mar. 11, 1933	22 00	do	60	68	183	106	70	380	276
59	do	22 41	do	60	68	183	106	70	380	276
60	do	23 33	do	60	68	183	106	70	380	276
61	do	15 02	do	60	68	183	106	70	380	276
62	Mar. 12, 1933	17 39	do	60	68	183	106	79	380	276
63	do	18 26	do	60	68	183	106	70	380	276
64	Mar. 13, 1933	04 33	do	60	68	183	106	70	380	276
65	Mar. 12, 1933	23 54	do	60	68	183	106	70	380	276
66	Mar. 13, 1933	13 18	do	60	68	183	106	70	380	276
67	do	19 30	do	60	68	183	106	70	380	276
68	Mar. 14, 1933	00 37		60	68	183	106	70	380	276
69	do	12 19	do	60	68	183	106	70	380	276
70	do	19 02	do	60	68	183	106	70	380	276
71	do	22 43	do	60	68	183	106	70	380	276
72	Mar. 15, 1933	04 33	do	60	68	183	106	70	380	276
73	do	05 41	do	60	68	183	106	70	380	276
74	do	23 14	do	60	68	183	106	70	380	276
75	Oct. 2, 1933	09 11	do	41	50	163	131	74	367	261
76	May 5, 1929	00 07	Whittier	29	35	163	137	58	------	------
77	do	06 23	do	29	35	163	137	58	------	------
78	July 8, 1929	16 46	do	29	35	163	137	58	------	------
79	do	17 05	do	29	35	163	137	58	------	------
80	do	17 21	do	29	35	163	137	58	------	------
81	do	17 46	do	29	35	163	137	58	------	------
82	do	21 36	do	29	35	163	137	58	------	------
83	July 9, 1929	00 22	do	29	35	163	137	58	------	------
84	do	04 24	do	29	35	163	137	58	------	------
85	do	04 59	do	29	35	163	137	58	------	------
86	do	08 03	do	29	35	163	137	58	------	------
87	Mar. 17, 1933	16 52	Long Beach	60	68	183	106	70	380	276
88	Jan. 30, 1934	19 25	Nevada	465	456	450	614	490	140	250
89	do	20 17	do	465	456	450	614	490	140	250
90	do	21 05	do	465	456	450	614	490	140	250
91	do	23 41	do	465	456	450	614	490	140	250
92	Jan. 31, 1934	00 26	do	465	456	450	614	490	140	250
93	do	03 56	do	465	456	450	614	490	140	250
94	do	14 28	do	465	456	450	614	490	140	250
95	Feb. 1, 1934	11 02	do	465	456	450	614	490	140	250
96	do	11 47	do	465	456	450	614	490	140	250
97	Dec. 20, 1932	20 11	do	514	507	510	658	535	192	296
98	Dec. 21, 1932	07 42	do	514	507	510	658	535	192	296
99	do	08 49	do	514	507	510	658	535	192	296
100	do	11 34	do	514	507	510	658	535	192	296
101	do	14 40	do	514	507	510	658	535	192	296
102	June 25, 1933	20 47	do	574	568	------	730	610	258	366
103	Mar. 12, 1934	15 08	Utah	980	970	1,030	1,080	970	720	800
104	do	18 23	do	980	970	1,030	1,080	970	720	800
105	May 6, 1934	08 12	do	980	970	1,030	1,080	970	720	800
106	Apr. 7, 1934	02 16	do	980	970	1,030	1,080	970	720	800
107	Nov. 9, 1927	04 20	Point Arguello	235	------	110	365	310	------	------
108	Feb. 1, 1934	11 20	Nevada	465	456	450	614	409	140	250

Records of 108 shocks which are listed in table 29 have been measured for all stations as far as available. Table 29 contains a list of these shocks and the approximate distances of the stations from the epicenter. As the accurate distances were not needed in the present investigation the closest values available from previous investigations have been used without further examination of the accuracy. Many of the distances have been computed by using the epicenters given by Dr. Richter in the "Monthly Report on Local Earthquakes, Pasadena." These shocks did not provide data concerning earthquakes nearer than 50 kilometers to Tinemaha, Haiwee, and La Jolla. Therefore, besides the shocks listed in table 29, small shocks originating within this distance from the stations have been investigated. In a similar way records of 22 blasts recorded at Pasadena and four of the outside stations have been investigated. The constants concerning these blasts are given in table 30. Table 31 gives a list of the seismograms on which periods of microseisms have been measured. Altogether, over 2,000 records have been investigated.

TABLE 30.—*List of blasts*

No.	Date	Epicenter	Distance in kilometers					
			Pasadena Long	Pasadena Short	Mount Wilson	Santa Barbara	La Jolla	Riverside
1	Oct. 21, 1923	Palos Verdes		49				
2	Apr. 27, 1924	Corona		68				
3	Dec. 8, 1924	Hollywood		18				
4	June 26, 1927	Monolith	112	112		145		158
5	Apr. 1, 1928	Slover Mountain	78	72				8
6	Apr. 28, 1929	San Gabriel Dam	32	32	20			50
7	June 11, 1929	do	32	32	20	176		50
8	June 15, 1929	Victorville	98	98	85		195	85
9	June 26, 1929	San Gabriel Dam	32	32	20	176	162	50
10	Apr. 12, 1930	Glendora Road No. 1	30	30				48
11	Apr. 12, 1930	Glendora Road No. 2	30	30				48
12	Sept. 12, 1931	Victorville No. 2	98	98	84			72
13	Apr. 21, 1933	San Gabriel West Fork	32	32	20	176		50
14	June 2, 1933	Signal Hill	41	41	48			
15	Oct. 27, 1934	do		40±				
16	Nov. 17, 1934	do		40±				
17	Dec. 6, 1930	Buena Vista No. 1						2
18	do	Buena Vista No. 2						2
19	Nov. 12, 1930	La Jolla Pit No. 1					1	
20	Nov. 13, 1930	La Jolla Pit No. 2					1	
21	Sept. 17, 1929	Skunk Point Blasts No. 1				56		
22	do	Skunk Point Blasts No. 2				56		

TABLE 31.—*Dates for which microseisms have been measured*

Pasadena	Mount Wilson	Haiwee	La Jolla
Nov. 20–21, 1934.	Sept. 30,-Oct. 1, 1934.	Aug. 9–10, 1934.	Dec. 16–17, 1934.
Aug. 27–28, 1934.	Sept. 28–29, 1934.	Aug. 13–14, 1934.	Dec. 13–14, 1934.
Sept. 5–6, 1934.	Sept. 26–27, 1934.	Aug. 8–9, 1934.	Dec. 9–10, 1934.
Sept. 10–11, 1934.	Sept. 15–16, 1934.	Aug. 5–6, 1934.	Dec. 4–5, 1934.
Oct. 4–5, 1934.	Sept. 12–13, 1934.	Aug. 3–4, 1934.	Nov. 30–Dec. 1, 1934
Oct. 13–14, 1934.	Sept. 2–3, 1934.	Aug. 1–2, 1934.	Nov. 26–27, 1934.
Dec. 16–17, 1934.	Aug. 31–Sept. 1, 1934.	Aug. 1–Sept. 1, 1934.	Nov. 20–21, 1934.
Nov. 18–19, 1934.	Aug. 29–30, 1934.	Aug. 29–30, 1934.	Nov. 15–16, 1934.
Nov. 21–22, 1934.	Aug. 16–17, 1934.	July 22–23, 1934.	Nov. 5–6, 1934.

Santa Barbara	Riverside	Tinemaha
Jan. 13–14, 1935.	July 11–17, 1934.	Dec. 2–3, 1934.
Jan. 12–13, 1935.	July 10–11, 1934.	Nov. 30–Dec. 1, 1934.
Jan. 11–12, 1935.	July 9–10, 1934.	Nov. 28–29, 1934.
Jan. 4–5, 1935.	July 8–9, 1934.	Nov. 27–28, 1934.
Dec. 31, 1934–Jan. 1, 1935.	July 5–6, 1934.	Nov. 15–16, 1934.
Dec. 22–24, 1934.	July 4–5, 1934.	Nov. 6–7, 1934.
Dec. 16–17, 1934.	July 29–30, 1934.	Nov. 3–4, 1934.
Dec. 12–13, 1934.	June 27–28, 1934.	Oct. 21–22, 1934.
Dec. 10–11, 1934.	June 20–21, 1934.	Oct. 12–18, 1934.

THE PERIODS MEASURED IN RECORDS OF EARTHQUAKES

In each record a certain number of periods has been measured concerning the P-phase, the S-phase, and the maximum phase. For shocks less than 100 kilometers distant there is usually no clear distinction between "S" and "M." In these cases the first waves of the large phase have been called "S" and the later waves "M." All measurements were finished before calculations were begun in order to avoid any influence of known data on the measurements. For each phase on each seismogram the number of occurrences of each period found in that phase has been tabulated. It is not possible to reproduce the results of these measurements in extenso in this paper. Table 32 gives the complete data concerning the short-period instruments at Pasadena while tables 33 to 38 show the tabulations concerning the outside stations for distances less than about 200 kilometers.

The outstanding result of these tabulations is that the periods which have been measured change very slightly between different shocks recorded from about the same distances at the same station and that they usually cover a very narrow range.

TABLE 32.—*Frequencies of periods recorded by the short-period instruments at Pasadena during the earthquakes listed in table 29*

The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

PASADENA P-PHASE

No.	Distance	Periods in seconds																									
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
	km																									
76	29		9	4	2	1	1																			
77	29		10	3	1	5	2	1	1																	
78	29				1	1			1																	
79	29	1	1	4	6	9																				
80	29		9	6	2																					
81	29		7	8		3																				
82	29		6	5	5																					
83	29		6	7	3	4	1																			
84	29			8	6	2																				
85	29		3	3	3	6	1																			
86	29		6	6	1																					
75	41	3	7	1		3	3	3	1	1	2															
18	44		8	6		7																				
21	46	2	12	15	2	2																				
10	49	5	7	9	6	8	4																			
28	54				2	4	3																			
29	54	6	8	5																						
56	60				4	5	1																			
57	60		3	3	5	9	4	1	1	2																
58	60			7	7	4																				
59	60			3	10	6																				
60	60		6	6	4	6	4																			
61	60		5	4	3	9	2																			
62	60	5	6	2	2	5	4																			
63	60	4	6	5	5	5																				
64	60		2	6	9	4																				
65	60	2			1	8	6	4																		
66	60	4	2	4	5	5	3					1														
67	60				5	12	1																			
68	60		1	5	5	10	3																			

TABLE 32.—*Frequencies of periods recorded by the short-period instruments at Pasadena during the earthquakes listed in table 29*—Continued

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

PASADENA P-PHASE—Continued

No.	Distance	Periods in seconds																								
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
	km																									
69	60	1	3	2	7	8	1																			
70	60			1	8	7	2																			
71	60		4	6	11	1																				
72	60		3	3	8	7																				
73	60	2	6	4	6	4																				
74	60	3	5		1	7	4		2																	
87	60		2	9	7	1																				
8	117	1	6	9	10	15	4																			
2	122				8	11	5																			
3	122		1	4	6	8																				
1	130	12	14	4	7	8	2																			
15	135	5		2	4	7	5																			
6	137		2	3	1	2																				
7	139		4	14	1																					
12	145	1	5	7	8	17	5																			
4	165		1	5	10	14	7	2	1																	
40	171		4	6	5	11	4																			
5	173	9	11	11	10	8																				
16	176	2	4	2	5	8	2																			
11	178	1	3	4	11	12	6	2																		
17	178	5	6	7	7	7	3	3																		
9	186		3	6	8	9	6	3																		
14	188	4	6	2	5	9	7	2	1																	
107	235		1	2	2	10	5	4																		
30	270	1	2	1		5	7	6	3	1	1															
31	270			1	1	2	6	6	4	3	2	1														
32	270	3	5	5	6	9	5	5	3	1	1															
33	270	3	4	4	6	8	7		1	1		2	3													
34	270		3	2	3	8	11	8	8	6	2															
35	270	1	2	6	7	7	6	6	4																	

36	270		2	5	7	6		1	1	1	4															
37	270			1	1	10	6	2			1															
38	270				3	4	2																			
51	272			5	17	18	10	9	12	4	12	7	7	10		1	2	2			2					
13	288		2	2	2	8	3	3	4																	
23	289		8	4	1	12	14	8	5	1	6		2	1	2	3	3	1								
24	289		2	1		5	10	10	8	4	8	3	2	1	1	1	4	1	1							
25	289		3	2	2	8	11	16	6	3	2															
20	360		5	7	5	8	4																			
39	360		1	1	5	8	8																			
19	381	1	3	1		3	2	3			2															
88	465					2	3	5	5	5	11	6	4													
89	465					6	2	5	5	5	8	2	1	1		1										
90	465				2	4	4	4	6	6	4	3														
91	465						6	6	6	8	3	5														
92	465					1	4	4	3	5	10	3	1													
93	465				1	4	4	6	4	8	6	1														
94	465				2	6	5	5	6	7	3	2														
95	465			2	3	6	7	6	5	1	4	5	1		1											
96	465			1	4	6	1	5		3	7	5	4	3	1											
108	465			2	3	4	5	5	5		4	10	2	4												
97	514	3	10	8	4	6	8	10	6	4	6															
98	514	5	9	5	6	12	4	2	3	4	1	4														
99	514	1	3	4	5	9	9	5	4	6	5	4														
100	514	2	5	5	5	10	9	6	6	3	2															
101	514	2	10	8	5	7	6	6	7	2	8	1														
49	555				1	7	7	5	4	5	2		1													
26	557		1	4	2	1	1	3								1	1	2	1							
27	557					2	3	3	2																	
102	574			1	5	18	10	3		2	14	4	2													
52	777					2	2	5	3	5	11	13	5	3												
103	980						2	8	10	15	20	18	5	10	1	8		3								
104	980					5	8	10	9	9	21	23	6	2	2											
105	980					1		5	2	3	25	26	11	13	2	1	2									
106	980						4	8	5	7	20	17	1	5	2											
46	1,000	1	12	12	1																					
47	1,000			2		2	2	1	1		7	7	3	1												
50	1,221					3	5	4			7	4	4	4	1	7	3	4	2							
48	1,351						3	9	3		5	11	4	7	4	4		2								
41	2,053																	3	1	2	1	2	2			
53	2,209										2			3		8	7		2							
54	2,209									1	2	1		1		2	2	6	7	1	2					
42	2,220													1		4	2	3	1		5	4	1			
43	2,220												1	1		2		3	2	2						
44	2,220									1	2			1		1	3	9	4	1	3					
45	2,442							1			1	2	1	2	1	8	3	3								

TABLE 32.—*Frequencies of periods recorded by the short-period instruments at Pasadena during the earthquakes listed in table 29*—Continued

The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

PASADENA S-PHASE

No.	Distance	Periods in seconds																				
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
	km																									
76	29		4	4	4	4	1	1	1																	
77	29		1	4	3	9	6	3	1																	
78	29				1	2	1	1	1	2																
79	29		3		4	9																				
80	29		3	6			2	1			1															
81	29		2		6	5	5																			
82	29		3	9	1	3	2		1																	
83	29		1	5	3	2	5	2																		
84	29			1	2	3	8	2																		
85	29			3	4	5	5	5	2																	
86	29		4	6	4	4	2																			
75	41	2	11	12	3	1																				
18	44		3	5	4	6	1	2	1																	
21	46		10	13	2	1	2	2																		
10	49	5	8	8	8	7	3	2	1																	
28	54					2	5	6	1																	
29	54	2	7	7																						
56	60				2	4	3	2																		
57	60					1	4	8	1	2																
58	60		1	2	7	5	2																			
59	60				3	8	7	2		1	1															
60	60					6	11	4																		
61	60					2	11	4	2																	
62	60	1	3	7	5	4																				
63	60				4	7	7	2																		
64	60				2	12	3																			
65	60			2	6	11	10	1																		
66	60				3	9	5																			
67	60				5	11	3																			
68	60				5	12	5																			

69	60				7	7	3	2																		
70	60				7	5	1	2	2		3															
71	60		1	4	11	3																				
72	60	1	1			4	3	1	2	2	3															
73	60		3	9		4	2	2	1																	
74	60						2	6	7	2																
87	60					5	4	3	4	1	1															
8	117	2	5	4	6	9	5																			
2	122				2	9	10																			
3	122				9	10	5																			
1	130	10	10	1	3	5	1	3	2																	
15	135	6	1	2	8	8	5	1																		
6	137		1	1	3	1	1	2																		
7	139		6	7		4																				
12	145		1	3	8	12	6	3																		
4	165	4	8	6	4	11	6	4	2	3	2	2														
40	171		2	6	7	9	7	2																		
5	173	7	10	10	9	6			1	1																
16	176	6	7	3	6	7	1																			
11	178	1	3	5	5	11	9	1	2																	
17	178	9	10	4	3	11	6	3	3	1																
9	186		1	1	5	11	9	4	1																	
14	188	5	6	4	4	4	1	2	2	4	4															
107	235					3	3	4	6		1	3														
30	270					3	8	7	5	5																
31	270					2		3	3	5	3	9	3													
32	270				2	4	7	8	6	4																
33	270	1	3	2	3	1		1	1	2	1	3	2	1	4	4	4									
34	270	1	3	3	3	6	9	7	5	2	1															
35	270					2	6	6	4	6	5	1														
36	270			1	8	10	1					1	2													
37	270					6	9	1	1	1	1															
38	270				1	8	9	1																		
51	272				5	11	7	5	8	4	7	4	2	4	1	2	1	1								
13	288		1		4	5	6	3	2																	
23	289		1	1	1	10	10	11	8	2	12	3	8	4	4	2										
24	289			1	1	7	4	3	2	1	7	5	7	10	3	4										
25	289					8	13	9	7	3	13	3	1													
20	360		2	3	4	5	4	3	2	3	5															
39	360			4	7	7	8	4	2	3	3															
19	381		1	4	1	4		1	1																	
88	465								4	4	4	11	3	2	2	3										
89	465					4	2	4	2	4	6	8	2													
90	465					3	8		8	6	4															
91	465					3	6		5	12	5	6														

TABLE 32.—*Frequencies of periods recorded by the short-period instruments at Pasadena during the earthquakes listed in table 29*—Continued

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

PASADENA S-PHASE—Continued

No.	Distance	Periods in seconds																								
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
	km																									
92	465	----	----	----	----	2	1	5	4	5	7	7	2	----	----	----	----	----	----	----	----	----	----	----	----	----
93	465	----	----	----	----	1	5	3	4	10	7	7	----	----	----	----	----	----	----	----	----	----	----	----	----	----
94	465	----	----	----	----	----	----	7	4	8	11	11	----	----	----	----	----	----	----	----	----	----	----	----	----	----
95	465	----	----	----	----	2	3	3	2	5	3	4	3	5	1	3	----	1	----	----	----	----	----	----	----	----
96	465	----	----	----	----	----	----	2	2	----	1	7	9	13	4	5	1	----	----	----	----	----	----	----	----	----
108	465	----	----	----	----	2	1	3	5	----	4	5	6	7	5	2	2	----	----	----	----	----	----	----	----	----
97	514	1	3	----	2	5	9	8	6	4	6	3	1	----	----	----	----	----	----	----	----	----	----	----	----	----
98	514	2	3	6	12	4	6	3	2	1	5	7	3	----	2	----	----	----	----	----	----	----	----	----	----	----
99	514	----	2	3	4	11	13	6	1	5	7	2	----	----	----	----	----	----	----	----	----	----	----	----	----	----
100	514	----	2	3	2	5	14	8	6	3	5	2	2	1	----	----	----	----	----	----	----	----	----	----	----	----
101	514	----	2	2	5	10	15	8	2	3	6	2	----	----	----	----	----	----	----	----	----	----	----	----	----	----
49	555	----	----	----	1	4	2	1	4	9	9	----	1	2	----	6	1	----	----	----	----	----	----	----	----	----
26	557	----	----	----	----	3	3	1	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----
27	557	----	----	----	----	2	3	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----
102	574	----	----	----	----	7	10	7	7	4	10	7	3	----	----	----	----	----	----	----	----	----	----	----	----	----
52	777	----	----	----	----	1	1	1	----	5	9	11	4	8	2	3	1	----	----	----	1	2	----	----	----	----
103	980	----	----	----	----	----	----	3	7	3	6	14	12	12	7	2	5	10	5	1	1	1	----	----	----	----
104	980	----	----	----	----	1	2	1	1	6	19	23	16	14	----	8	----	----	----	----	----	----	----	----	----	----
105	980	----	----	----	----	2	2	1	1	1	9	13	9	17	13	5	7	3	3	----	----	----	----	----	----	----
106	980	----	----	----	----	1	----	2	1	7	8	19	----	17	5	9	5	2	----	----	----	----	----	----	----	----
46	1,000	----	7	8	5	1	----	----	----	----	----	----	----	----	2	----	----	----	----	----	----	----	----	----	----	----
47	1,000	----	----	2	1	1	1	2	----	----	1	2	----	2	3	2	3	1	----	----	----	----	----	----	----	----
50	1,221	----	----	----	----	----	1	----	----	3	4	6	6	7	3	2	3	4	2	----	----	----	----	----	----	----
48	1,351	----	----	----	----	----	----	----	----	----	3	4	2	5	7	4	4	8	5	1	1	1	----	----	----	----
41	2,053	----	1	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	3	4	2	----	----	----	----	----
53	2,209	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	1	2	12	1	----	----	----
54	2,209	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	1	3	1	1	4	2	2	----
42	2,220	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	1	----	2	3	2	1	----	----
43	2,220	----	----	----	----	----	----	----	----	----	----	2	----	----	----	----	----	2	1	----	----	4	----	----	----	----
44	2,220	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	2	3	----	----	4	7	8	----	----
45	2,442	----	----	----	----	----	----	----	----	----	----	----	----	2	----	1	2	----	1	----	----	3	----	----	----	----

PASADENA M-PHASE

76	29	2		1	3	3	4	6	3	3	2	2														
77	29					5	3	8	5	3																
78	29							3	3	1	1	1														
79	29		3		8	9																				
80	29		4	1		3	7		5	1	1															
81	29				5	6	6	5	1			1														
82	29			8	4	3	4																			
83	29			4	4	3	3	4	1																	
84	29					3	2	4	8		4															
85	29			1		3	6	7	4	3		1														
86	29		5		2	2	6	1																		
75	41	1	7	1		1	4	5	2	2	9	10	5													
18	44			3		7	2	6	3	2	3															
21	46	4	8			6		1	1	3	1															
10	49	1	1	2	7	8	7	3	2	1																
28	54						1	3	4	2																
29	54	1	1	3	6	7	2																			
56	60	1		2	1	3																				
57	60						5	6	6	2	1															
58	60				2	8	6																			
59	60					1	6	3	5	3																
60	60						6	5	2	3	2															
61	60					1	7	2	3	3	2															
62	60	6	6		2	9	3	1		1	1	1														
63	60		2			2	5	6	4	1																
64	60				2	9	8																			
65	60						4	5	10																	
66	60				2		3	4	5	2	3	2														
67	60						3	6	7	3																
68	60						4	6	7																	
69	60					3	6	3	7																	
70	60						1	1	3	5	9	2														
71	60				3	4	5	3	2	2																
72	60					5	2	1	2	6	2															
73	60			1	2	6	8	1																		
74	60					1	2		2	6	5	4														
87	60				4	10	4																			
8	117		3	5	6	6	5																			
2	122					9	7	8	4																	
3	122					5	8	3	3	3	3	1														

TABLE 32.—*Frequencies of periods recorded by the short-period instruments at Pasadena during the earthquakes listed in table 29*—Continued

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

PASADENA M-PHASE—Continued

No.	Distance	Periods in seconds																								
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
	km																									
1	130	8	7	1	5	3	4	3	3																	
15	135	10	10	1	1	6	5	4	1																	
6	137		5	6		4	2																			
7	139		7	4	3	2																				
12	145		1	3	6	9	1	2																		
4	165		1	2	4	8	8	4	4	1																
40	171				1	10	7			1	10	6		1												
5	173	1	8	12	12	12	2																			
16	176	4	4	2	3	5	3	2	1	1																
11	178	1		3	6	12	4	4	3																	
17	178		1	3	3	5	7	3	6	4	2	2														
9	186		2	2	6	8	6	3	1																	
14	188	1	5	5	6	5	4	3	2	1		1														
107	235	1	1		1	1		1	1	3	4	2	3	2												
30	270					1	3	4	7	6	2															
31	270								4	5	8	10	3													
32	270					4	9	11	2																	
33	270									1	1	2	4	2	4	2	7	1	2							
34	270						7	8	9	6	5															
35	270					1	1	3	6	5	5	4														
36	270			1		4	3		6	3																
37	270					1	4	4	2	1	2															
38	270					4	7	2	2	2																
51	272				5	12	7	5	8	4	12	13	9	15	6	2	3	1	2	3	2	1				
13	288	1	1			3	1	3	1	1																
23	289						4	1	4	5	12	8	5	1	4	4	2	2	1	1						
24	289					1	1		1	1	1	6	8	9	3	1	3	2	1		5					
25	289					1	1	4	6	9	20	3	8	1	2											
20	360			3	6	3	4	5	4	1	3															
39	360		3	5	5	5	3	3	4	4	4															

19	381		3	1	1	3			2			2														
88	465											4	4	19	5	1										
89	465							2	3	6	5	4	1	5			1	2								
90	465						2	3	9	8	5	2														
91	465						4	3	4	6	8	12	5	2		2										
92	465					3	3	4	5	5	3		2	6												
93	465							3	5	10	11	7														
94	465							3	3	10	13	4														
95	465					2	5	2	3	2	5	4	5	11	2	2	1									
96	465								1			3	3	10		5	7	2	1	4	1					
108	465								1			7	6	11	4	7	3	4	5							
97	514		2	1		1	2	4		8	12	3	3	3												
98	514					1		1		3	10	19	12	4												
99	514					3	2	2	3	12	13	10	2	2		3										
100	514					5	4	3	1	6	12	13	5	3	2											
101	514	2			2	8	5	5	5	8	6	6			3	4										
49	555										1	3	2	4	1		2	10	7	1						
26	557					1	4	3																		
27	557							5	5																	
102	574						5	2	4	7	6	4	4	3	1	1										
52	777										1		1	3	1	5		2	7	1	1	12	3			
103	980											1		1		2	1	5	3	3	4	35	17	2		
104	980						1				5	6	4	10	3	6	2	2	5	1	5	25	1			
105	980								1		1	4	5	8	8	4	4	7	5		11	35	9			
106	980						2			3	8	6	1	7	5	4	7	13	2	2	2	13	1			
46	1,000																			2	5	5	6			
47	1,000															2		1			1	7	9	1		
50	1,221										3	3		1	1	3	2	2	1					1	12	
48	1,351										1			6	3	3	4	6	1	5	3	7	6	2		
41	2,053																								3	5
53	2,209																						3	13	1	
54	2,209																						8	8	2	
42	2,220																						1	5	4	
44	2,220																						2	11	14	
45	2,442																								6	8

TABLE 33.—*Frequencies of periods recorded at Riverside at distances less than 200 kilometers during the earthquakes listed in table 29*

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

RIVERSIDE P-PHASE

No.	Distance	Periods in seconds																			
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4
	km																				
2	37	3	2		1																
3	37	9	3	2																	
76	58	9																			
77	58	2	5	5	1																
78	58	4	6	6	5																
79	58	5	5	5	3																
80	58	9	7	5	2																
81	58	9	8	2	1																
82	58	7	6	5	2																
83	58	7	6																		
84	58	14	4	2																	
85	58	6	7	3	2																
86	58	13	3	3																	
57	70	2	4	2	5																
58	70	6	4	5																	
59	70	14				8	3		10												
60	70	10	8	2	6	4															
61	70	2	2	8	8	2															
62	70	4	8	8	10	4	2														
63	70	12	5	7	5																
64	70	14	4		2		4														
65	70	16	1	3																	
66	70	8	5	4	1																
67	70	9		7	2																
68	70	2	12	11	3																
69	70	5	11	4							4			2	2						
70	70	5	5	3																	
71	70	9	8	2																	
72	70	11	9																		
73	70	8	8	1																	
74	70	9	8	1																	
87	70	10	9	1																	
75	74	6	3			4															
8	75	11	9	3	1	1	1														
1	77	3	3	4	4	3	1														
29	86		4	1	3	9	4	1	1												
28	86	4	9	5	1																
21	107		1	5	2	1	1														
10	108		1	4	4	5	1														
17	108	7	7	3																	
6	109	1	11	7	4	2															
7	111		5	5		3															
4	114		4	1	2	9	4	1	1												
40	122				7	5	2														
18	123		8	16	13	3															
5	146	1	4	5	6	5	6														
16	170	4	7	5	4	3	1														
11	178		2	4	4	6	1														
30	190		3	4	3	6	3	3	1												
31	190	2	3	3	4	6	6			1											
32	190	2	1			8	6	2													
33	190				1	6	6	2	2	1											
34	190		2	3	2	4	4	4	4												
35	190	2	2	4	3	6	2														

TABLE 33.—*Frequencies of periods recorded at Riverside at distances less than 200 kilometers during the earthquakes listed in table 29*—Continued

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

RIVERSIDE S-PHASE

No.	Distance	Periods in seconds																			
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4
	km																				
2	37	3	4	5	2	1	1														
3	37	3	4	3	2	1															
76	58	4	8	3	1																
77	58	6	7	3	2																
78	58	7	12																		
79	58		11	4																	
80	58		4	11	3																
81	58	5	10	3																	
82	58	3	10	5																	
83	58	2	10	8																	
84	58	5	12	4																	
85	58	4	11	4	2																
86	58	9	9	4																	
57	70	1	6	2																	
58	70	6	6		2	6	6	5													
59	70		12	8																	
60	70	4	11	4	3	3	3	2	2												
61	70		12	11																	
62	70	14	5	5	3	3	2		2												
63	70	9	9	3	2	3		2	2												
64	70		10	8		2															
65	70	9	2						2		2	1	1								
66	70		6	4	2																
67	70		14	6	2	1			2	2	2										
68	70	5	8	2																	
69	70	4	9	6	3						1	1	1	1	1						
70	70		13	7																	
71	70	5	10	4																	
72	70	4	10	7																	
73	70	9	6	4																	
74	70	6	10	3																	
87	70	5	12	4																	
75	74	8	4																		
8	75	7	4																		
1	77		1	4	3	4	2														
28	86	6	5	5	5																
29	86	2	5	5	1																
21	107			3	3	3	1														
10	108		3	3	3	4	1														
17	108	6	9	4	3																
6	109		3	8		3															
9	110		4	3		2	1	1		1	1										
7	111		6	6	6	8															
4	114	2	1	5	3	5	1	2													
40	122				1	5	4	1													
18	123	3	2	11	2	8	1	1													
5	146	3	4	5	7	11															
16	170	5	7	3	4	6															
11	178	3	4	3	1	7	3	1													
30	190				4	7	6	3	1												
31	190	4	5	3	4	5	3														
32	190	2	4	2	3	8	7	1													
33	190				3	4	3	1	3	1	3	2		2	1						
34	190	6	6	5	4	4															
35	190	3	2			2	3	2	2	4	3	1	2								

TABLE 33.—*Frequencies of periods recorded at Riverside at distances less than 200 kilometers during the earthquakes listed in table 29*—Continued

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

RIVERSIDE M-PHASE

No.	Distance	Periods in seconds																			
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4
	km																				
2	37	3	9	4	3	2	3		2	2	2										
3	37	7	5		3	11	4		4	3	5										
77	58		5	1	7	2															
78	58		7	5	4	5	4	2													
79	58	1	8	6	1																
80	58	2	9	8	1																
81	58	3	9	5	4	4															
82	58	5	13	2																	
83	58	7	10	1																	
84	58	2	7	13																	
85	58	4	12	4																	
57	70	9	1	1	3																
58	70	3	4	4		1	6	7	2	3	2	1									
59	70		9	5				2		3	1										
60	70		6		4	5	7	4													
61	70		6				3		2	4	5	2			2						
62	70	7	3		1	5	5	5	3	3											
63	70		6		3	6	4	3													
64	70		6				3	1	2	5	2	5	3	3	2						
65	70	6	6	3					1	1	4	3	2	1	1	1	4				
66	70		9			4	2		1	3	3			4	3	1		3			
67	70		6		2	4	6	1	3		2	1	3								
68	70	6	6							4	2	4	6	5							
69	70	6	6		3	5					3	2	5	6							
70	70	3	2			4	2	2	4	3	3										
71	70	4	9	4	1	2															
72	70	5	4	6	4																
73	70	9	5	2																	
74	70	4	8	6	3	1	1														
87	70	1	5	9	3																
75	74			10	8	3	2	2													
8	75	3	5	4	5																
1	77		4	2	5	6	2														
28	86	8	8	6	3	2															
29	86		5	5	5	5	2														
21	107			9	2	1	5	3	2	1											
10	108	1	4	6	4	5	2														
17	108	8	12	1	1		1	1	1	3											
6	109	2	6	4	2	2	2	1													
9	110		5	4	4	6	3	3	2	1											
7	111		4	5	6																
4	114	2	3	5	3	4															
40	122					7	7	4	3	1	1										
18	123		5		5	6	4	4													
5	146		1	2	5	8	3														
16	170	1	3	5	4	2															
11	178	2	5	2	1	4	3	3	2												
30	190			1	4	5	5	3	2												
31	190		1	1	3	5	3	1													
32	190					7	8	2													
33	190													2	1	6	1	2	2		1
34	190						2	6	5	5	3	1	1		4						
35	190								4	2	5	6	5	2							

TABLE 34.—*Frequencies of periods recorded at Mount Wilson at distances less than 200 kilometers during the earthquakes listed in table 29*

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

MOUNT WILSON P-PHASE

No.	Distance	Periods in seconds												
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6
	km													
76	35	----	14	7	----	----	----	----	----	----	----	----	----	----
77	35	----	11	6	----	----	----	----	----	----	----	----	----	----
78	35	----	----	----	5	3	----	----	----	----	----	----	----	----
79	35	----	5	6	2	----	----	----	----	----	----	----	----	----
80	35	3	14	4	1	----	----	----	----	----	----	----	----	----
81	35	1	14	5	2	----	----	----	----	----	----	----	----	----
82	35	2	10	8	2	----	----	----	----	----	----	----	----	----
83	35	2	9	10	----	----	----	----	----	----	----	----	----	----
84	35	2	15	1	5	----	----	----	----	----	----	----	----	----
85	35	----	9	2	----	----	----	----	----	----	----	----	----	----
86	35	----	8	6	----	----	----	----	----	----	----	----	----	----
21	40	----	7	7	1	2	----	----	----	----	----	----	----	----
75	50	5	3	----	1	1	----	----	----	----	----	----	----	----
18	54	2	3	6	3	----	----	----	----	----	----	----	----	----
28	60	4	5	5	4	4	----	----	----	----	----	----	----	----
29	60	5	4	1	----	----	----	----	----	----	----	----	----	----
10	63	9	8	5	9	10	5	----	----	----	----	----	----	----
57	68	----	7	7	3	----	----	----	----	----	----	----	----	----
58	68	----	14	10	1	1	----	----	----	----	----	----	----	----
59	68	4	17	4	3	----	----	----	----	----	----	----	----	----
60	68	11	12	2	----	----	----	----	----	----	----	----	----	----
61	68	6	14	4	----	----	----	----	----	----	----	----	----	----
62	68	14	11	2	----	----	----	----	----	----	----	----	----	----
63	68	12	9	3	----	----	----	----	----	----	----	----	----	----
64	68	----	8	5	----	----	----	----	----	----	----	----	----	----
65	68	----	8	2	----	----	----	----	----	----	----	----	----	----
66	68	----	5	5	----	----	----	----	----	----	----	----	----	----
67	68	7	9	2	----	----	----	----	----	----	----	----	----	----
68	68	9	16	2	----	----	----	----	----	----	----	----	----	----
69	68	----	10	13	5	2	----	----	----	----	----	----	----	----
70	68	----	14	10	2	----	----	----	----	----	----	----	----	----
71	68	----	3	14	7	3	2	----	----	----	----	----	----	----
72	68	7	12	8	5	----	----	----	----	----	----	----	----	----
73	68	----	18	4	----	----	----	----	----	----	----	----	----	----
74	68	2	15	7	----	2	----	----	----	----	----	----	----	----
87	68	4	15	9	2	----	----	----	----	----	----	----	----	----
8	103	8	8	4	----	----	----	----	----	----	----	----	----	----
2	112	----	2	3	5	6	----	----	----	----	----	----	----	----
3	112	----	1	4	7	7	1	----	----	----	----	----	----	----
1	116	----	5	9	4	2	----	2	----	----	----	----	----	----
7	124	----	3	8	1	3	----	----	----	----	----	----	----	----
14	140	2	2	4	2	2	3	2	2	1	----	----	----	----
11	144	----	2	1	6	6	4	1	1	----	----	----	----	----
15	145	5	7	8	3	1	----	----	----	----	----	----	----	----
5	150	5	10	5	2	1	----	----	----	----	----	----	----	----
4	154	----	4	8	5	3	----	----	----	----	----	----	----	----
40	157	1	7	8	4	2	----	----	----	----	----	----	----	----
16	163	3	6	8	4	2	----	----	----	----	----	----	----	----
17	168	6	9	5	5	1	1	----	----	----	----	----	----	----
9	170	----	2	8	3	7	----	----	----	----	----	----	----	----

TABLE 34.—*Frequencies of periods recorded at Mount Wilson at distances less than 200 kilometers during the earthquakes listed in table 29*—Continued

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

MOUNT WILSON S-PHASE

No.	Distance	Periods in seconds												
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6
	km													
76	35		9	8	4									
77	35	2	6	4	3	4			3					
78	35				2	1								
79	35	2	2	7		2								
80	35	10	10	3	5	4								
81	35		10	6	2									
82	35		8	13	2									
83	35		10	10	3	2								
84	35	3	10	7	1	1								
85	35		4	7	3	3								
86	35		3	8										
21	40		3	9	1	7	2							
75	50				1	7	1							
18	54	2	2	2	2	3	1							
28	60	2	4	5	5	4								
29	60	3	3	3										
10	63	4	4	8	13	13	7							
57	68	2		6	6	3								
58	68		9	5	10	1								
59	68		10	9	6						1			
60	68		6	12	5	2								
61	68		14	10	2					1				
62	68		3	4	1									
63	68		6	15	4									
64	68			7	3	1	1	1						
65	68		1	8	2	1								
66	68		5	6	2									
67	68		7	5	1									
68	68	3	12	13										
69	68		11	13										
70	68	2	9	14	11									
71	68	3	10	10	5	3	2							
72	68	4	8	9	2	4	1							
73	68		15	8										
74	68		17	8	1									
87	68		14	11										
8	103	6	7	6	2	1								
2	112			1	4	8	4							
3	112		2	4	8	8	1							
1	116		1	5	6	4	3	2	1		1			
7	124			1	1	1								
14	140	5	5	6	3	2	3	3	2	3	1			
11	141	3	2	2	2	7	1							
15	145	5	7	8	3	2	2							
5	150	7	8	6	2									
4	154		1	5	6	9								
40	157		4	5	8	2								
16	163	5	2	3	8	7	1							
17	168	2	7	6	3	2	2							
9	170			2	5	8	4	2						

TABLE 34.—*Frequencies of periods recorded at Mount Wilson at distances less than 200 kilometers during the earthquakes listed in table 29*—Continued

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

MOUNT WILSON M-PHASE—Continued

No.	Distance	Periods in seconds												
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6
	km													
76	35	6	4	----	1	1	2	----	1	----	5	4	4	4
77	35	----	2	----	----	9	8	3	3	1	----	1	----	----
78	35	----	3	3	3	9	6	----	----	----	----	----	----	----
79	35	----	6	2	----	----	2	1	----	----	2	----	----	----
80	35	----	16	4	1	1	----	1	2	----	----	----	----	----
81	35	----	2	9	8	2	1	1	1	----	----	----	----	----
82	35	----	4	15	2	5	----	----	----	----	----	----	----	----
83	35	----	15	9	3	4	----	----	----	----	----	----	----	----
84	35	1	9	6	6	----	----	----	----	----	----	----	----	----
85	35	1	3	7	1	----	----	----	----	----	----	----	----	----
86	35	----	9	6	----	----	----	----	----	----	----	----	----	----
21	40	----	----	2	3	3	2	4	3	----	----	----	----	----
75	50	----	----	----	4	5	----	----	----	----	----	----	----	----
18	54	----	----	3	2	3	1	2	1	2	1	----	----	----
28	60	----	----	1	8	9	----	----	----	----	1	----	----	----
29	60	----	----	----	6	3	----	----	----	----	----	----	----	----
10	63	3	8	9	6	7	3	2	1	1	----	----	----	----
57	68	----	2	4	2	----	----	1	1	3	1	1	----	----
58	68	----	6	7	3	6	4	----	----	----	----	----	----	----
59	68	----	3	10	4	4	2	----	----	----	----	----	----	----
60	68	----	2	6	4	4	6	2	----	----	----	----	----	----
61	68	----	3	11	8	5	----	----	----	----	----	----	----	----
62	68	----	4	2	----	1	1	----	----	1	2	----	----	----
63	68	----	3	6	4	7	2	1	----	----	----	----	----	----
64	68	----	1	2	6	2	----	----	----	----	----	----	----	----
65	68	----	6	3	3	4	----	----	----	----	----	----	----	----
66	68	----	1	4	5	1	----	----	----	----	----	----	----	----
67	68	----	2	9	2	----	----	----	----	----	----	----	----	----
68	68	----	8	13	4	1	----	----	----	----	----	----	----	----
69	68	4	7	7	1	----	----	----	----	4	1	----	----	----
70	68	----	8	5	4	3	----	----	----	2	5	4	----	----
71	68	4	9	6	2	2	3	2	2	2	----	----	----	----
72	68	8	1	3	2	----	4	2	----	3	----	1	----	----
73	68	----	9	12	1	1	----	----	----	----	----	----	----	----
74	68	----	12	4	----	----	----	2	----	3	3	----	----	1
87	68	----	11	11	2	----	----	2	----	2	----	----	----	----
8	103	----	5	7	5	1	----	----	----	----	----	----	----	----
2	112	----	----	----	3	8	8	----	2	1	1	----	----	----
3	112	----	----	----	7	10	3	3	----	----	----	----	----	----
1	116	----	4	2	3	3	2	----	5	2	2	----	----	----
7	124	----	3	5	1	----	----	----	----	----	----	----	----	----
14	140	1	1	3	3	6	5	1	1	----	----	----	----	----
11	144	----	3	4	4	7	1	----	----	----	----	----	----	----
15	145	----	----	2	8	8	5	1	----	----	----	----	----	----
5	150	2	4	6	6	3	----	----	----	----	----	----	----	----
4	154	----	2	7	3	3	3	----	----	----	----	2	----	----
40	157	----	----	----	2	11	6	3	1	----	----	----	----	----
16	163	1	6	6	5	4	2	3	----	----	----	----	----	----
17	168	4	5	4	4	6	2	----	----	----	----	----	----	----
9	170	----	1	8	3	7	3	1	1	1	----	----	----	----

TABLE 35.—*Frequencies of periods recorded at Santa Barbara at distances less than 200 kilometers during the earthquakes listed in table 29*

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

SANTA BARBARA P-PHASE

Number	Distance	Periods in seconds															
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4
	km																
15	18	5	10	5	----	----	----	----	----	----	----	----	----	----	----	----	----
14	45	3	4	3	3	6	1	----	----	----	----	----	----	----	----	----	----
18	100	3	18	3	8	3	----	----	----	----	----	----	----	----	----	----	----
107	110	----	1	2	----	5	5	----	----	----	----	----	----	----	----	----	----
21	113	4	4	1	2	2	----	1	----	1	----	----	----	----	----	----	----
12	124	----	1	3	6	7	1	----	----	----	----	----	----	----	----	----	----
10	132	----	2	2	2	7	4	3	2	----	----	----	----	----	----	----	----
75	163	----	2	2	----	6	7	12	6	4	3	4	----	----	----	----	----
76	163	----	4	2	----	4	4	3	1	1	1	----	----	----	----	----	----
77	163	----	----	1	3	1	6	2	----	----	2	----	----	----	----	----	----
78	163	----	----	----	1	4	2	----	----	----	----	----	----	----	----	----	----
79	163	----	4	3	3	9	2	2	----	----	----	----	----	----	----	----	----
80	163	----	2	3	3	7	5	----	----	----	----	----	----	----	----	----	----
81	163	----	2	7	6	6	----	----	----	----	----	----	----	----	----	----	----
32	163	----	----	2	5	10	4	----	----	----	----	----	----	----	----	----	----
84	163	----	----	----	10	9	----	----	----	----	----	----	----	----	----	----	----
85	163	----	----	----	5	12	2	----	----	----	----	----	----	----	----	----	----
28	165	----	1	4	3	11	3	----	----	----	----	----	----	----	----	----	----
29	165	2	2	3	6	6	2	----	----	----	----	----	----	----	----	----	----
22	168	----	2	6	3	6	4	4	1	----	----	----	----	----	----	----	----
13	175	----	1	2	3	7	4	2	----	----	----	----	----	----	----	----	----
23	179	----	----	1	----	9	12	11	5	1	1	----	----	----	----	----	----
24	179	----	----	3	3	6	6	6	4	2	3	----	----	----	----	----	----
25	179	----	----	2	5	6	8	5	3	----	2	1	----	----	----	----	----
56	183	----	----	10	7	6	----	----	----	----	----	----	----	----	----	----	----
57	183	----	----	----	1	10	4	2	----	----	----	----	----	----	----	----	----
58	183	----	----	----	9	10	4	----	----	----	----	----	----	----	----	----	----
59	183	----	----	----	4	9	7	2	----	----	----	----	----	----	----	----	----
60	183	----	----	----	----	8	7	2	----	----	----	----	----	----	----	----	----
61	183	----	----	----	1	11	5	2	----	----	----	----	----	----	----	----	----
62	183	----	----	1	6	10	----	----	----	----	----	----	----	----	----	----	----
63	183	----	----	----	5	11	4	----	----	----	----	----	----	----	----	----	----
64	183	----	----	2	7	11	2	2	----	----	----	----	----	----	----	----	----
65	183	----	----	----	6	10	5	----	----	----	----	----	----	----	----	----	----
66	183	----	----	----	----	9	11	----	----	----	----	----	----	----	----	----	----
67	183	----	----	2	8	14	----	----	----	----	----	----	----	----	----	----	----
68	183	----	----	----	8	10	1	----	----	----	----	----	----	----	----	----	----
69	183	----	----	----	4	15	3	----	----	----	----	----	----	----	----	----	----
70	183	----	2	2	7	10	5	----	----	----	----	----	----	----	----	----	----
71	183	----	----	2	10	8	----	----	----	----	----	----	----	----	----	----	----
72	183	----	----	3	8	8	2	----	----	----	----	----	----	----	----	----	----
73	183	----	2	4	6	8	1	----	----	----	----	----	----	----	----	----	----
74	183	----	----	----	2	11	10	----	----	----	----	----	----	----	----	----	----
87	183	----	----	----	8	13	3	----	----	----	----	----	----	----	----	----	----

TABLE 35.—*Frequencies of periods recorded at Santa Barbara at distances less than 200 kilometers during the earthquakes listed in table 29*—Continued

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

SANTA BARBARA S-PHASE

Number	Distance	Periods in seconds															
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4
	km																
15	18	3	3	5	----	4	1	2	2	1	----	----	----	----	----	----	----
14	45	----	2	1	4	5	5	1	----	----	----	----	----	----	----	----	----
18	100	3	8	7	2	3	3	2	3	----	----	----	----	----	----	----	----
107	110	----	2	3	1	1	1	1	2	----	----	----	----	----	----	----	----
21	113	----	3	3	3	8	2	4	2	----	----	----	----	----	----	----	----
12	124	----	----	----	3	10	5	2	----	----	----	----	----	----	----	----	----
10	132	----	----	3	4	8	6	1	----	----	----	----	----	----	----	----	----
75	163	----	----	4	11	4	9	9	5	5	2	----	----	----	----	----	----
76	163	----	----	----	----	----	----	2	1	3	2	1	----	2	----	----	----
77	163	----	----	----	----	3	2	----	3	3	3	----	1	----	----	----	----
79	163	----	----	----	5	9	7	4	----	----	----	----	----	----	----	----	----
80	163	----	----	----	3	7	6	1	----	----	----	----	----	----	----	----	----
81	163	----	----	----	3	13	3	2	----	----	----	----	----	----	----	----	----
82	163	----	----	----	----	10	8	----	----	----	----	----	----	----	----	----	----
84	163	----	3	----	----	3	5	----	----	----	----	----	----	----	----	----	----
85	163	----	----	----	2	8	4	----	----	----	----	----	----	----	----	----	----
28	165	----	----	----	5	12	4	----	----	----	----	----	----	----	----	----	----
29	165	----	----	5	3	5	5	----	----	----	----	----	----	----	----	----	----
22	168	----	----	8	3	6	2	5	1	----	----	----	----	----	----	----	----
13	175	----	3	----	3	7	6	3	2	----	----	----	----	----	----	----	----
23	179	----	----	3	3	5	5	5	5	2	7	----	----	----	----	----	----
24	179	----	----	1	1	4	1	4	2	----	4	----	2	----	----	----	----
25	179	----	----	----	2	9	5	4	4	3	5	----	1	----	----	----	----
56	183	----	----	----	1	2	8	2	3	----	2	----	----	----	----	----	----
57	183	----	----	----	1	9	7	1	----	----	----	----	----	----	----	----	----
58	183	----	----	----	2	10	9	----	2	----	----	----	----	----	----	----	----
59	183	----	----	----	----	3	11	5	----	----	----	----	----	----	----	----	----
60	183	----	----	----	----	2	13	6	----	----	----	----	----	----	----	----	----
61	183	----	----	----	2	6	8	----	----	----	----	----	----	----	----	----	----
62	183	----	----	----	----	4	7	6	----	----	----	----	----	----	----	----	----
63	183	----	----	----	8	2	9	----	----	----	----	----	----	----	----	----	----
64	183	----	----	1	5	7	1	----	----	----	----	----	----	----	----	----	----
65	183	----	----	----	----	3	9	4	1	----	----	----	----	----	----	----	----
66	183	----	----	----	2	7	6	----	----	----	----	----	----	----	----	----	----
67	183	----	----	----	2	12	7	----	----	----	----	----	----	----	----	----	----
68	183	----	----	----	3	10	2	3	----	----	----	----	----	----	----	----	----
69	183	----	----	----	----	12	9	4	----	----	----	----	----	----	----	----	----
70	183	----	----	----	2	5	11	2	----	----	----	----	----	----	----	----	----
71	183	----	----	----	----	----	7	6	5	1	----	----	----	----	----	----	----
72	183	----	----	----	2	10	7	----	----	----	----	----	----	----	----	----	----
73	183	----	2	2	6	7	8	----	----	----	----	----	----	----	----	----	----
74	183	10	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----
87	183	----	----	----	----	12	8	----	----	----	----	----	----	----	----	----	----

TABLE 35.—*Frequencies of periods recorded at Santa Barbara at distances less than 200 kilometers during the earthquakes listed in table 29*—Continued

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

SANTA BARBARA M-PHASE

Number	Distance	Periods in seconds															
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4
	km																
15	18						1	1	2	8	8	6					
14	45					2	5	4	2	3	2						
18	100	3	6	4	2	5		6	3	4	3						
107	110						3	2	3	3	3	3	2				
21	113		3	6			2	5	1	3	2	1	1				
12	124			2	1	6	3	1									
10	132			1		7	4	3	2	2							
75	163					7	4	7	7	4	6	9					
76	163				2	3	3		2		2	2					
77	163					2	1	5	3		2	2					
78	163					3	3	2	2	2							
79	163					2	6	6									
80	163			5		10	4										
81	163				3	12	2	1									
82	163		2	2	5	8	4										
84	163				3	7	5	3									
85	163				3	6	6										
28	165					5	7	4	2	1							
29	165					5	4	4	5	1							
22	168					2	6	5	6	3							
13	175			3	2	6		1	1	3	3						
23	179				1	3	3	5	4	7	8	6	1				
24	179				1	5	1	2	1	1	8	5	3	6	1	2	1
25	179				1	8	8	5	5	1	3	2	1				
56	183					1	2	4	1	2	4	2					
57	183					4	5	5	1		1	1					
58	183						14	7									
59	183							9	7	5							
60	183					5	9	6	2								
61	183					2	8	7	4								
62	183					2	4	6	4								
63	183				1	4	3	4	7	4							
64	183					4	9	4	3	2							
65	183							5	9	6							
66	183					5	7	3	1								
67	183					5	8	6	2								
68	183					4	8	6	1								
69	183						2	7	6	4	3						
70	183							3	5	5	7	1					
71	183					4	3	4	6	7							
72	183					6	6	7	3								
73	183					5	7	6	6								
74	183							6	8	7							
87	183		3			3	8	6	5	2							

TABLE 36.—*Frequencies of periods recorded at Haiwee at distances less than 200 kilometers during the earthquakes listed in table 29*

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

HAIWEE P-PHASE

No.	Distance	Periods in seconds											
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4
	km												
11	70	2	4	5	4	2							
16	80		2	1	3	5							
5	140	2	3	3	6	6							
6	155		7	3		5	4	1	1				
7	155			3	2	7	5		1		2		
19	162		4	11	6	3	3	1					
20	180	2		1	5	8	5						
39	180		5	3	7	8	1						
40	180		2	5	3	1	1						
8	181	1	2	7	8	5							
1	202		1	1	3	10	6						
4	203		2	2	6	7	4						

HAIWEE S-PHASE

No.	Distance	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4
11	70		2	2	4	5	3						
16	80	1	3	1	2	5	1						
5	140		1	1	2	6	4	4	2				
6	155		2	6	3	4	2	3	5				
7	155			3	3	5	4	2	2	1			
19	162		9	10	6	5	5	5	3				
20	180				4	8	8	3					
39	180				1	10	9						
40	180					1	3	2	3	1	1		
8	181			1	4	8	6	1					
1	202				3	3	4	4	7	3			
4	203			3	5	2	4	4	3				

HAIWEE M-PHASE

No.	Distance	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4
11	70		2	2	2	4	3	3	1	2			
16	80			1	4	5	1						
5	140			1	3	8	5	1					
6	155			3		9	3	5		2	2	1	
7	155			5	1	1	2	1	2	1	1	1	
19	162		2	10	5	6	7	3					
20	180				3	7	7	7	5				
39	180					4	6	5		4	4		
40	180										3	3	3
8	181			1	2	7	6	2					
1	202	1		3		2	4	3	4	4	1		
4	203			3	2	6	3	1	4	2			

TABLE 37.—*Frequencies of periods recorded at Tinemaha at distances less than 200 kilometers during the earthquakes listed in table 29*

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

TINEMAHA P-PHASE

No.	Distance	Periods in seconds																			
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4
	km																				
19	59	----	8	7	6	4	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----
20	64	----	4	6	4	5	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----
39	64	----	4	5	4	6	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----
88	140	----	2	4	1	3	7	6	6	4	----	----	----	----	----	----	----	----	----	----	----
89	140	----	2	2	8	13	7	----	----	----	----	----	----	----	----	----	----	----	----	----	----
90	140	----	7	15	6	4	3	4	----	----	----	----	----	----	----	----	----	----	----	----	----
91	140	----	5	4	7	9	2	----	----	----	----	----	----	----	----	----	----	----	----	----	----
92	140	----	5	----	2	7	3	----	----	----	----	----	----	----	----	----	----	----	----	----	----
93	140	----	6	4	2	4	6	4	2	2	2	----	----	----	----	----	----	----	----	----	----
94	140	3	6	7	5	7	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----
95	140	----	5	4	5	10	8	----	----	----	----	----	----	----	----	----	----	----	----	----	----
96	140	----	3	2	5	13	5	----	----	----	----	----	----	----	----	----	----	----	----	----	----
108	140	----	3	5	4	11	8	4	2	----	----	----	----	----	----	----	----	----	----	----	----
11	179	2	4	7	4	6	3	----	----	----	----	----	----	----	----	----	----	----	----	----	----
97	192	----	----	1	1	5	9	7	2	----	----	----	----	----	----	----	----	----	----	----	----
98	192	----	----	1	----	6	5	7	4	2	2	----	----	----	----	----	----	----	----	----	----
99	192	----	----	----	----	4	6	5	4	2	1	3	----	----	----	----	----	----	----	----	----
100	192	----	----	2	8	7	4	3	2	1	----	----	----	----	----	----	----	----	----	----	----
101	192	----	----	----	5	3	5	8	1	2	2	----	----	----	----	----	----	----	----	----	----
16	194	----	----	----	5	11	3	----	----	----	----	----	----	----	----	----	----	----	----	----	----
12	205	2	2	3	5	8	3	----	----	----	----	----	----	----	----	----	----	----	----	----	----
1	208	----	----	1	1	4	3	1	1	----	----	----	----	----	----	----	----	----	----	----	----

TINEMAHA S-PHASE

No.	Distance	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4
19	59	2	5	5	3	5	4	1	----	----	----	----	----	----	----	----	----	----	----	----	----
20	64	----	----	----	4	9	6	3	----	----	----	----	----	----	----	----	----	----	----	----	----
39	64	----	1	2	5	7	5	----	----	----	----	----	----	----	----	----	----	----	----	----	----
88	140	----	----	----	4	10	9	3	5	----	----	----	----	----	----	----	----	----	----	----	----
89	140	----	----	----	2	4	1	2	3	2	3	1	----	----	----	----	----	----	----	----	----
90	140	----	4	----	7	9	9	4	2	----	----	----	----	----	----	----	----	----	----	----	----
91	140	----	4	2	15	7	4	----	----	----	----	----	----	----	----	----	----	----	----	----	----
92	140	----	5	5	5	2	5	----	----	----	----	----	----	----	----	----	----	----	----	----	----
93	140	----	4	4	3	2	8	3	4	2	----	----	----	----	----	----	----	----	----	----	----
94	140	----	5	5	5	11	3	----	----	----	----	----	----	----	----	----	----	----	----	----	----
95	140	----	2	3	2	3	3	5	4	----	----	4	4	1	----	----	----	----	----	----	----
96	140	----	----	----	2	3	4	6	4	2	3	3	----	3	3	----	----	----	----	----	----
108	140	----	----	----	----	2	3	4	3	2	9	4	3	2	1	----	----	----	----	----	----
11	179	----	1	3	4	8	5	----	----	----	----	----	----	----	----	----	----	----	----	----	----
97	192	----	----	----	----	----	----	2	1	----	4	3	----	----	----	----	----	----	----	----	----
98	192	----	----	----	2	4	7	4	3	3	1	----	----	----	----	----	----	----	----	----	----
99	192	----	----	----	----	----	6	7	4	----	3	4	1	----	----	----	----	----	----	----	----
100	192	----	----	----	----	2	3	7	2	5	4	2	----	----	----	----	----	----	----	----	----
101	192	----	----	----	----	2	3	6	5	3	4	3	----	----	----	----	----	----	----	----	----
16	194	----	----	----	----	9	6	4	1	1	----	----	----	----	----	----	----	----	----	----	----
12	205	----	2	4	3	5	4	----	----	----	----	----	----	----	----	----	----	----	----	----	----
1	208	----	----	----	----	2	2	4	1	1	----	----	----	----	----	----	----	----	----	----	----

TABLE 37.—*Frequencies of periods recorded at Tinemaha at distances less than 200 kilometers during the earthquakes listed in table 29*—Continued

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

TINEMAHA M-PHASE

No.	Distance	Periods in seconds																			
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4
	km																				
19	59	----	5	5	2	6	5	4	3	2	----	----	----	----	----	----	----	----	----	----	----
20	64	----	----	----	----	----	3	----	2	2	6	3	2	----	----	----	----	----	----	----	----
39	64	----	----	----	----	----	----	----	----	6	10	1	----	----	----	----	----	----	----	----	----
88	140	----	----	----	4	8	11	3	3	4	2	1	1	----	----	----	----	----	----	----	----
89	140	----	----	----	2	3	2	2	2	6	5	4	----	1	1	5	1	2	----	----	----
90	140	----	----	5	----	9	4	8	6	1	----	2	----	----	----	----	----	----	----	----	----
91	140	----	----	5	4	7	5	7	4	----	----	----	----	----	----	----	----	----	----	----	----
92	140	----	6	6	----	----	8	5	1	----	----	----	----	----	----	----	----	----	----	----	----
93	140	----	----	3	4	3	2	3	3	4	3	3	----	----	----	----	----	----	----	----	----
94	140	----	1	2	4	7	5	2	2	2	3	----	----	----	----	----	----	----	----	----	----
95	140	----	----	----	3	5	4	3	2	1	2	4	1	2	2	2	----	3	4	----	----
96	140	----	----	----	----	----	2	----	2	----	3	6	4	9	2	2	----	----	3	1	----
108	140	----	----	----	2	2	----	2	3	3	6	2	1	3	2	1	----	2	2	2	1
11	179	----	1	2	1	4	4	3	3	3	1	----	----	----	----	----	----	----	----	----	----
97	192	----	----	----	----	----	1	----	----	1	1	3	3	2	----	----	----	----	----	----	----
98	192	----	----	----	----	2	2	----	4	3	4	5	4	2	----	----	----	----	----	----	----
99	192	----	----	----	----	1	2	3	2	2	2	8	4	2	1	----	----	----	----	----	----
100	192	----	----	----	----	2	2	3	1	3	5	4	----	2	3	----	----	----	----	----	----
101	192	----	----	----	----	----	2	1	1	3	4	6	4	3	----	----	----	----	----	----	----
16	194	----	----	----	----	1	3	5	5	4	2	1	----	----	----	----	----	----	----	----	----
12	205	----	----	----	3	6	5	3	----	----	----	----	----	----	----	----	----	----	----	----	----
1	208	----	----	----	1	3	1	3	1	1	1	----	----	----	----	----	----	----	----	----	----

TABLE 38.—*Frequencies of periods recorded at La Jolla at distances less than 200 kilometers during the earthquakes listed in table 29*

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

LA JOLLA P-PHASE

No.	Distance	Periods in seconds													
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0
	km.															
17	60	4	7	----	8	6	----	----	----	----	----	----	----	----	----	----
56	106	----	4	8	6	1	----	----	----	----	----	----	----	----	----	----
57	106	----	----	6	6	1	6	2	----	1	3	5	----	----	----	----
58	106	3	5	8	7	1	----	----	----	----	----	----	----	----	----	----
59	106	----	6	6	7	2	----	----	----	----	----	----	----	----	----	----
60	106	2	4	2	2	1	----	----	----	----	----	----	----	----	----	----
61	106	4	4	8	4	----	----	----	----	----	----	----	----	----	----	----
62	106	----	2	9	8	----	----	----	----	----	----	----	----	----	----	----
63	106	3	8	8	2	----	----	----	----	----	----	----	----	----	----	----
64	106	----	3	6	9	4	----	----	----	----	----	----	----	----	----	----
65	106	----	----	7	10	2	----	----	----	----	----	----	----	----	----	----
66	106	----	4	9	7	----	----	----	----	----	----	----	----	----	----	----
67	106	----	10	10	----	----	----	----	----	----	----	----	----	----	----	----
68	106	9	7	6	5	2	----	----	----	----	----	----	----	----	----	----
69	106	1	10	6	5	----	----	----	----	----	----	----	----	----	----	----
70	106	----	5	5	2	----	2	----		----	2	2	----	1	----	----
71	106	----	5	8	5	----	----	----	----	----	----	----	----	----	----	----
72	106	4	10	4	1	----	----	----	----	----	----	----	----	----	----	----
73	106	----	3	9	10	----	----	----	----	----	----	----	----	----	----	----
74	106	8	3	6	9	----	----	----	----	----	----	----	----	----	----	----

TABLE 38.—*Frequencies of periods recorded at La Jolla at distances less than 200 kilometers during the earthquakes listed in table 29*—Continued

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated vertically in the order of increasing distance]

LA JOLLA P-PHASE—Continued

No.	Distance	Periods in seconds														
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0
	km.															
87	106	1	2	10	7	1										
51	107			10	7	3										
28	123		2	4	9	8										
29	123	2	3	3												
75	131	2	3	3	8	12	3									
76	137		4	3	7	5										
77	137		9	6	3	2										
78	137			4	7	8										
79	137	3	7	5	2	1										
80	137		1	4	3											
81	137		5	8	6	2										
84	137		2	8	3	6	1									
85	137		2	1	2	4	5	5								
2	142		3	4	8	6										
3	142		4	3	5	10	2									
10	153		4	6	6	7										
21	176		6	6	2	9	2									
1	186		3		3	4	2									
9	186				4	8	7	2								
8	197	1	3	3	4	7										
18	200			6	8	8	6									

LA JOLLA S-PHASE

No.	Distance	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0
17	60			2	9	8	1									
56	106					2	3	2	1	1	1	3	1			
57	106					3	5	5	2							
58	106				5	11	2			1						
59	106	10	8	2												
60	106		2	4	3	2										
61	106	1	5	8	3	2										
62	106		2	5	10	4										
63	106			7	9	2										
64	106				4	9	7									
65	106				3	11	4									
66	106		4	6	7	2										
67	106		6	9	8	2										
68	106		5	10	5											
69	106	6	4	9	5	4										
70	106		2	3	7	6										
71	106			3	3	5	6	1								
72	106	6	8	9	4											
73	106		6	7	6											
74	106			9	7	4	2									
87	106			3	7	8										
28	123	1	2	4	9	6										
29	123			2	3	3	2									
75	131		3		10	13	3									
76	137		3	4	7	4										
77	137			4	3	3	2	2								
79	137				5	9	6	4								
80	137		3	3	3	1										
81	137		8	7												
84	137			3	3	5	6	3	1	3						
85	137			6	4	3	3	5		2						
2	142				3	12	9									
3	142				7	12	8									
10	153		4	3	4	7	3	1	1							
21	176		1	3	5	4	1	2	1	1						

TABLE 38.—*Frequencies of periods recorded at La Jolla at distances less than 200 kilometers during the earthquakes listed in table 29*—Continued

[The numbers in the first column correspond to those in table 29. The frequencies are tabulated verticaly in the order of increasing distance]

LA JOLLA S-PHASE—Continued

No.	Distance	Periods in seconds														
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0
	km.															
1	186				1	2	3	2	2	1						
9	186				3	4	6	6	4	1						
8	197			2	6	10	2									
18	200		1	5		7	4	2								

LA JOLLA M-PHASE

No.	Distance	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0
17	60		3	3	7	6										
56	106					1	2	2	5	3	3	2	2			
57	106					2	1	2	3	6	3					
58	106					3	6	6	4		1					
59	106					5	2	4	3	8	2	1				
60	106						1	3	5							
61	106				2	6	8	2	3							
62	106				7	8	3	2								
63	106				1	8	9									
64	106					2	5	2	7	4	2					
65	106					1	3	7	8	3						
66	106					2	8	7	4							
67	106				2	10	7	2								
68	106				4	11	7									
69	106				7	8	2									
70	106				2	5	3	6	5							
71	106					2	6	4	3							
72	106					4	7	7	3							
73	106				5	9	4									
74	106				4	5	6	7	4							
87	106			5	6	8										
51	107			2	2	10	2	8	9		5	4	1	2	2	1
28	123	3		4	7	9										
29	123				1	4	5	4								
75	131	2			4	14	6									
76	137			6	9	9	2	2								
77	137			3	3	3	1									
78	137					3	3	2	2	2						
79	137					2	6	6								
80	137		1	3	1											
81	137		2	5	3	4	2	1								
84	137			2		2	2	3	1	3						
85	137					1		3	3		4					
2	142					10	7	4	4	2						
3	142				3	8	6	3								
10	153	4	3	3		6	5	2	2							
21	176			1	7	3	3	5		1	2					
1	186		2				1	4	3	1						
9	186		3	1	3	6	5	2	1	1						
8	197			1	3	10	4									
18	200			4		1	5	6	3	1						

DIFFERENCES IN DIFFERENT COMPONENTS

O. Geussenhainer (see footnote on p. 163) found in his investigations on microseisms in Göttingen that the horizontal components showed the free periods of the ground more clearly than the vertical component. To investigate if the same is true concerning the periods in earthquakes, a part of the material has been treated separately for the three components, especially seismograms obtained at short epicentral distances. Unfortunately, it is only recently that vertical components have become available at our stations, so that the number of records of the vertical component is very much smaller than that of the horizontal components. Only intervals of distance have been used in this section for which a sufficient number of seismograms with vertical components—always more than 50 and if possible more than 100—were available. In considering the results we must always keep in mind that the constants of the vertical instruments differ from those of the horizontal. The special differences were mentioned in the section on "Data."

The two horizontal components in general agree very well, as may be seen from some of the following tabulations. Some other examples which make this fact clear are the following: At Riverside, out of over 300 seismograms for distances between 50 and 100 kilometers, 42 percent of the P-phases showed periods of 0.2 seconds in the north-south component, while the corresponding figure in the east-west component was 47 percent. For distances between 100 and 200 kilometers 27 percent out of about 200 seismograms had a period of 0.5 second in the P-phase in the north-south component and 26 percent in the east-west component. In the S-phase the dominant period for 50 to 100 kilometers (about 300 seismograms) was 0.2 second with 45 percent in the north-south component and 48 percent in the east-west component. In the range from 100 to 200 kilometers, 47 percent out of about 200 seismograms showed periods between 0.5 and 0.6 second in the north-south component and 51 percent in the east-west component. In the M-phase at distances between 50 and 100 kilometers the period of 0.2 second has been measured in 24 percent of the cases in the north-south component and 27 percent in the east-west component, while at distances between 100 and 200 kilometers 35 percent of the measurements in the north-south component and the same percentage in the east-west component showed periods of 0.5 and 0.6 second.

At Mount Wilson periods of 0.2 and 0.3 second prevail in all phases for distances between 0 and 50 kilometers as well as 50 to 100 kilometers, as shown in comparison 1, table 39. The corresponding frequencies for these periods and others at La Jolla and Santa Barbara are shown in comparisons 2 and 3, table 39. The most frequent periods are always shown in bold-face type.

TABLE 39.—*Relative frequency, in percent of occurrence, of various periods on different components*

[Bold-face numbers indicate the most frequent periods]

Comparison no.	Station	Distance	Phase	Component	Periods in seconds											
					0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4
		km														
1	Mount Wilson	0 to 50	P	NS	1	**57**	35	7	1	0	0	0	0	0	0	0
		do	P	EW	14	**59**	16	6	6	0	0	0	0	0	0	0
		do	S	NS	0	36	**44**	7	11	1	0	1	0	0	0	0
		do	S	EW	3	30	**34**	13	15	2	0	2	0	0	0	0
		do	M	NS	1	**34**	25	7	11	6	3	2	1	5	1	1
		do	M	EW	3	**21**	18	12	17	10	6	6	0	3	3	2
		51 to 100	P	NS	9	**45**	32	11	3	0	0	0	0	0	0	0
		do	P	EW	11	**56**	25	4	3	1	0	0	0	0	0	0
		do	S	NS	3	**41**	**39**	11	5	2	0	0	0	0	0	0
		do	S	EW	2	33	**43**	15	6	0	0	0	0	0	0	0
		do	M	NS	1	**34**	**25**	7	11	6	3	2	1	5	1	1
		do	M	EW	2	16	**28**	19	17	7	3	1	5	2	1	0
2	La Jolla	101 to 200	P	NS	4	20	**32**	**25**	15	3	1	0	0	0	1	0
		do	P	EW	4	19	**29**	**25**	14	5	1	0	0	1	1	0
		do	S	NS	1	10	**25**	**26**	**23**	11	4	2	0	0	0	0
		do	S	EW	2	10	**20**	**23**	**26**	10	5	2	2	0	1	1
3	Santa Barbara	101 to 200	P	NS	1	3	8	23	**43**	16	5	2	0	0	0	0
		do	P	EW	0	2	9	21	**42**	18	5	2	1	1	0	0
		do	S	NS	0	1	2	7	**31**	**35**	12	4	2	5	1	0
		do	S	EW	0	1	4	17	**32**	**29**	10	4	1	1	0	0
4	Pasadena, short period	101 to 200	P	NS	3	11	15	24	**34**	11	3	1	0	0	0	0
		do	P	EW	3	10	14	20	**39**	13	1	0	0	0	0	0
		do	P	Vertical	5	20	**30**	19	20	4	2	0	0	0	0	0
		do	S	NS	2	9	13	19	**31**	18	4	2	2	0	1	0
		do	S	EW	3	7	5	10	**32**	22	10	7	2	2	0	0
		do	S	Vertical	4	19	**23**	**20**	**23**	8	2	0	0	0	0	1
		do	M	NS	3	11	13	15	**20**	11	4	7	8	5	2	0
		do	M	EW	1	4	4	7	**23**	**27**	15	10	3	4	2	0
		do	M	Vertical	1	9	20	20	**39**	11	0	1	0	0	0	0

TABLE 39.—*Relative frequency, in percent of occurrence, of various periods on different components*—Continued

[Bold-face numbers indicate the most frequent periods]

Comparison no.	Station	Distance	Phase	Component	Periods in seconds											
					0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4
		km														
5	Pasadena, short period	201 to 300	P	Vertical	3	7	10	12	**14**	**17**	10	11	4	4	3	2
		do	P	All	1	5	6	9	**19**	**16**	13	9	4	6	2	2
		do	S	Vertical	2	5	6	6	**23**	**22**	17	10	2	5	3	0
		do	S	All	0	2	2	5	**16**	**17**	13	11	6	9	6	5
		401 to 600	P	Vertical	0	1	6	9	**24**	19	15	9	7	5	4	1
		do	P	All	2	5	5	6	**16**	13	12	10	9	13	8	2
		do	S	Vertical	0	1	0	1	12	**20**	16	9	**14**	9	5	2
		do	S	All	0	2	2	4	9	**14**	10	9	11	**14**	12	1
6	Pasadena, long period	101 to 200	P	NS	5	13	**20**	12	**29**	9	2	1	2	3	1	5
		do	P	EW	6	15	**21**	12	**25**	14	4	1	1	1	0	0
		do	P	Vertical	0	0	7	23	**50**	12	5	1	0	1	0	0
		do	S	NS	1	7	14	11	**21**	17	11	7	3	5	2	0
		do	S	EW	4	7	12	12	**21**	10	7	9	8	4	0	2
		do	S	Vertical	0	3	3	13	**36**	**24**	9	6	4	1	0	1
		do	M	NS	1	3	9	7	**20**	11	10	6	5	5	3	4
		do	M	EW	1	6	8	12	**16**	10	6	2	6	10	10	3
		do	M	Vertical	0	8	7	10	**26**	16	14	12	6	1	0	0
7	Haiwee	0 to 50	P	Horizontal	23	**43**	**28**	2	4	0	0	0	0	0	0	0
		do	P	Vertical	20	**34**	**30**	11	5	0	0	0	0	0	0	0
		do	S	Horizontal	6	**32**	**34**	7	18	1	1	0	0	0	0	0
		do	S	Vertical	16	**24**	**27**	18	16	0	0	0	0	0	0	0
		do	M	Horizontal	0	9	17	25	**36**	4	6	2	0	0	0	0
		do	M	Vertical	2	23	**36**	13	**23**	0	0	0	0	0	0	0
8	Tinemaha	do	P	Horizontal	15	**32**	28	13	12	0	0	0	0	0	0	0
		do	P	Vertical	24	**34**	22	14	6	0	0	0	0	0	0	0
		do	S	Horizontal	3	13	14	22	**44**	5	0	0	0	0	0	0
		do	S	Vertical	3	8	23	24	**41**	1	0	0	0	0	0	0
		do	M	Horizontal	0	0	3	6	**39**	35	9	5	1	3	0	0
		do	M	Vertical	0	4	10	27	**46**	13	0	0	0	0	0	0
9	do	101 to 200	P	NS	0	1	13	22	**27**	18	11	5	2	1	0	0
		do	P	EW	0	1	5	10	**31**	23	13	6	5	4	1	0
		do	P	Vertical	3	**47**	28	13	8	0	0	0	0	0	0	0
		do	S	NS	0	1	2	6	**18**	**20**	12	14	7	8	6	3
		do	S	EW	0	0	0	11	**21**	**23**	17	8	2	8	7	2
		do	S	Vertical	0	15	**18**	15	**18**	5	15	8	2	5	0	0
		do	M	NS	0	1	1	1	6	12	**14**	**15**	9	9	10	4
		do	M	EW	0	0	1	1	9	**17**	12	6	11	10	**14**	7
		do	M	Vertical	0	3	6	**24**	**25**	8	6	9	2	8	5	0

10	do	201 to 300	P	Vertical	0	13	21	8	**25**	14	15	4	0	0	0	0
		do	P	All	1	3	13	11	**30**	15	7	3	2	3	6	5
		do	S	Vertical	0	4	6	14	**27**	16	10	10	8	4	2	0
		do	S	All	2	4	6	8	**19**	14	11	9	7	7	8	3
		do	M	Vertical	0	2	4	11	**17**	**20**	13	7	11	7	9	0
		do	M	All	0	1	3	8	**19**	12	11	8	7	9	6	6
		301 to 400	P	Vertical	0	12	19	17	**34**	14	3	2	0	0	0	0
		do	P	All	1	5	10	12	**35**	20	11	4	1	1	0	0
		do	S	Vertical	0	0	3	9	**36**	**32**	15	5	1	0	0	0
		do	S	All	0	0	3	5	**25**	**27**	21	8	3	3	2	1
		do	M	Vertical	0	0	1	4	**28**	**26**	**25**	9	4	1	1	0
		do	M	All	0	1	3	3	14	**22**	**25**	12	8	5	2	1
		601 to 800	P	Vertical	0	0	3	6	**21**	**21**	**21**	11	8	6	2	0
		do	P	All	0	0	2	3	11	**17**	**17**	11	8	**14**	11	3
		do	S	Vertical	0	0	0	8	**36**	**27**	8	10	3	5	3	0
		do	S	All	0	0	0	3	**16**	**15**	12	8	7	**15**	9	4
		do	M	Vertical	0	0	0	0	8	11	9	13	3	**17**	8	10
		do	M	All	0	0	0	0	3	3	6	7	4	**14**	10	11

At Haiwee the dominant periods at distances between 50 and 200 kilometers are 0.5 to 0.6 second in the S- and M-phases. These periods have been measured in 62 percent of all cases of S in the east-west component and in 51 percent in the north-south component, while the corresponding figures for the M-phase are 50 percent in the east-west component and 45 percent in the north-south component.

Data sufficient to compare the frequencies in the vertical component with those in the horizontal components are available at Pasadena for distances between 100 and 200 kilometers. The results are shown in comparison 4, table 39.

The differences between the vertical components and the horizontal components may be due partly to the different sensitivity of the instruments. This is especially true for the P-phase, where the vertical component shows a prevailing period of 0.3 second while the horizontal components show a considerably larger percentage for a period of 0.5 second. At distances from 200 to 300 kilometers and from 400 to 600 kilometers in the P- and S-phases, the values in comparison 5 seem to indicate that in the vertical component the dominant periods of 0.5 to 0.6 second occur slightly more frequently than in the average of all components.

In the M-phase, on the other hand, the period of 1 second has been observed more frequently at these distances in the horizontal components than in the vertical component. The latter shows quite a scattering, the frequency for distances from 200 to 600 kilometers being higher than 10 percent for each of the periods from 0.7 to 1.1 seconds.

The records of the long-period vertical instruments at Pasadena again show the periods of 0.5 and 0.6 second more frequently than the corresponding horizontal components as may be seen from comparison 6, table 39.

At Mount Wilson the number of seismograms available in the vertical component is large enough only for earthquakes at distances between 400 and 600 kilometers. In the P-phase the vertical component shows periods of 0.5 and 0.6 second in 58 percent of the cases, while the corresponding value for all measurements is 48 percent.

In the S-phase the corresponding figures are 55 percent in the vertical component and 38 percent for all measurements. In the M-phase the periods do not show clearly dominant values either in the vertical component or in the average for all instruments. There is no noticeable difference between the different components.

The same interval of distance (400 to 600 kilometers) can be used for Riverside. The periods of 0.5 and 0.6 second have been measured in 44 percent of all P-waves in the vertical component while the corresponding value is 28 percent for all components. In the S-phase periods of 0.6 and 0.7 second have been found in 35 percent of all cases in the vertical component and in 23 percent of all measurements. In the M-phase neither the vertical component measurements nor any other measurements show clearly the dominant periods in that distance interval, but periods of 1 second are somewhat more frequent in the horizontal components, where they form more than 15 percent of the measurements, while in the vertical component periods of 0.6, 0.7, 0.8, 0.9, and 1.0 second are about equally frequent.

At La Jolla there are fewer data for the vertical component than for the other stations. For distances between 100 and 200 kilometers, the period of 0.5 second has been measured in 28 percent of the cases concerning the vertical component of the M-phase, while the corresponding values are 24 percent for the east-west component, and 29 percent for the north-south component.

At Santa Barbara about 50 measurements are available for the vertical component at distances between 100 and 200 kilometers. The scanty data show periods of 1.0 and 1.1 seconds with noticeably greater frequency in the vertical component of the M-phase than in the horizontal components. In the first this frequency is 44 percent while the corresponding values for the east-west and north-south components are 4 percent and 5 percent respectively. At distances between 400 and 600 kilometers, on the other hand, the differences between the components are small.

At Haiwee measurements made especially on records of nearby shocks contain a relatively large amount of data for the vertical component which are tabulated in comparison 7, table 39.

At distances between 200 and 300 kilometers and 600 and 800 kilometers, periods of 0.5 and 0.6 second prevail very clearly in the vertical component with frequencies of 42 and 50 percent respectively. The corresponding values for all measurements are 41 percent and 21 percent so that here again the periods of 0.5 and 0.6 second are more frequently recorded in the vertical component than in the horizontal components. On the other hand, the average frequency of periods of 1.0 to 1.2 seconds in the P-phase is 36 percent for all measurements for distances between 600 and 800 kilometers, but only 7 percent for the vertical component. Similar results are to be found for the S-phase, where again the periods of 0.5 and 0.6 second have been observed with a higher frequency in the vertical component than in the horizontal, while the latter shows very much more frequently periods between 1.0 and 1.2 seconds. In the M-phase, again, the observed number of periods in the neighborhood of 0.6 second is somewhat larger in the vertical component, but in this case periods of 1.0 and 1.2 seconds are shown quite frequently too—the frequency being 28 percent for distances between 600 and 800 kilometers in the vertical component and 32 percent for all measurements.

For Tinemaha we have again the special measurements for local shocks at distances less than 50 kilometers. They do not show large differences between the horizontal and the vertical components, as may be seen from comparison 8, table 39.

For distances between 100 and 200 kilometers, between 60 and 70 observations are available for the vertical component. The results derived from the measurements are shown in comparison 9, table 39. Here short periods again prevail more clearly in the vertical than in the horizontal components.

At large distances the frequency of periods of 0.5 and 0.6 second is about the same in the vertical and horizontal component in all phases, except that at distances between 600 and 800 kilometers these periods have been observed somewhat more frequently in the vertical component than in the horizontal, as shown in comparison 10, table 39. Again, periods in the neighborhood of 1 to 1.2 seconds are more frequently recorded in the horizontal component than in the vertical. At distances between 600 and 800 kilometers, the P-phase shows these periods in only 7 percent of the observations for the vertical component, while the corresponding figure for all measurements together is over 25 percent. In the S-phase at the same distance the vertical component has these periods in 8 percent of the observations while all measurements give 25 percent, but in the M-phase they have been observed in 25 percent of the cases in the vertical component which equals within the limits of error the 25 percent of observations of these periods in all measurements.

If we finally summarize the results of this section, we see that the vertical components show more clearly the dominant short periods than do the horizontal components, but the latter indicate a larger number of waves with periods of 1.0 to 1.2 seconds. It is not possible to determine how much this result is influenced by the constants of the instruments, but it is not impossible that the vertical component of the ground movement really contains the dominant periods, especially those which are below 1 second, somewhat more frequently than the horizontal components.

THE EFFECT OF DISTANCE ON THE RECORDED PERIODS

In order to get the mean change of period with distance, all readings have been separated into groups with distances of 0 to 50, 51 to 100, 101 to 200, etc., kilometers for each station and for each phase. The results are given in tables 40 to 47 with the prevailing periods printed in heavy figures. In table 48 the prevailing periods are listed for each station and each distance. These tables show very clearly two facts: The first is that the range of periods prevailing in records belonging to a certain distance is very narrow, and differs from station to station only at the shorter distances. The second is that certain periods, especially those in the neighborhood of 0.5 to 0.6 second and from 1.0 to 1.2 seconds, prevail at certain distances at all stations. These facts are of high importance to structural engineers, as the data show the range of periods which prevail at a certain locality and for this reason are most dangerous to buildings there. Therefore, buildings should not have free periods in that range. The second result makes it very clear that at all seven stations the ground showed much more frequently waves with periods of 0.5 and 1.0 to 1.1 seconds than it should in case of forced vibrations. These periods correspond very probably to free vibrations of the ground at all stations.

TABLE 40.—*Frequencies of different periods recorded at Pasadena during earthquakes originating at different ranges of distance*

PASADENA P-PHASE, LONG PERIOD INSTRUMENTS

Distance (km)	Periods in seconds																			
	0.1 to 0.2	0.3 to 0.4	0.5 to 0.6	0.7 to 0.8	0.9 to 1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50	**79**	**71**	31	16	3	7		3			2									
51 to 100	29	**120**	94	29	2															
101 to 200	72	117	**170**	22	20	4	1	2												
201 to 300	6	**48**	**157**	59	69	16	23	16	11	6	1		1							
301 to 400	4	11	**15**	6	5															
401 to 600	2	22	66	**83**	**79**	68	62	64	49	49	9	17	6	3	1	3	2	1		
601 to 800			1	3	**7**	2	2	5	3	1	1		3	2	2					
801 to 1,000	3	5	21	22	**48**	36	21	21	7	**26**	15	9	7	13	10	9	2			
1,001 to 1,500		2	8	6	1			3	5	**10**	**4**	**12**	7	4	4	8				
2,001 to 2,500					5	1	1	7		22	9	**33**	**23**	**26**	**31**	20	4	1		
Total	195	396	**563**	246	239	134	110	**119**	75	**112**	38	71	47	48	48	40	8	2		

PASADENA S-PHASE, LONG PERIOD INSTRUMENTS

Distance (km)	0.1 to 0.2	0.3 to 0.4	0.5 to 0.6	0.7 to 0.8	0.9 to 1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50	46	**82**	54	26	8	15	4	1	6	2	2									
51 to 100	19	63	**92**	53	22	6	1	1	1	2			1	1	3	1		1		
101 to 200	50	92	**142**	60	42	9	6	3		2										
201 to 300	9	18	**91**	52	77	41	29	27	13	10	11		1							
301 to 400		10	12	12	**16**	4	3	1	2	2										
401 to 600		16	28	32	**61**	39	36	**58**	42	**72**	51	48	36	10	26	44	17			
601 to 800					2	1	4	**13**	1	2	2	1	2							
801 to 1,000	3	2	9	5	19	18	7	18	18	**31**	**23**	29	4	11	11	9	3			
1,001 to 1,500				3	4	1		2	6	5	8	7	6	1	3	8	6	1	1	
2,001 to 2,500										4		2	3	5	13	**61**	35	14	6	
Total	127	283	**428**	243	**251**	134	90	124	89	**132**	97	87	53	28	56	**123**	58	16	7	

TABLE 40.—*Frequencies of different periods recorded at Pasadena during earthquakes originating at different ranges of distance*—Continued

PASADENA M-PHASE, LONG PERIOD INSTRUMENTS

Distance (km)	Periods in seconds																			
	0.1 to 0.2	0.3 to 0.4	0.5 to 0.6	0.7 to 0.8	0.9 to 1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50	22	40	**67**	50	20	21	5	3	12	8	17	16	3	----	----	----	----	----	----	----
51 to 100	9	14	37	51	**87**	36	17	13	----	8	6	3	1	----	2	2	----	----	----	----
101 to 200	32	60	**96**	44	44	19	16	18	8	7	4	2	----	----	----	----	----	----	----	----
201 to 300	----	3	**59**	**48**	**54**	47	33	31	16	24	21	11	7	3	20	1	----	----	----	----
301 to 400	----	2	1	4	**17**	1	3	1	5	8	1	----	----	----	----	----	----	----	----	----
401 to 600	----	5	6	13	**57**	28	30	41	21	**57**	43	38	49	22	**87**	76	5	----	----	----
601 to 800	----	----	----	----	2	1	1	2	----	2	2	3	**7**	2	6	----	----	----	----	----
801 to 1,000	----	----	----	----	4	3	1	5	5	11	8	10	16	4	26	**108**	49	3	----	----
1,001 to 1,500	----	----	----	----	----	1	5	2	6	1	2	7	6	1	4	4	**13**	----	**11**	----
2,001 to 2,500	----	----	----	----	----	----	----	----	----	----	----	----	----	----	1	3	27	40	**62**	7
Total	63	124	**266**	210	285	154	111	116	77	**126**	104	90	89	32	120	194	94	43	73	**7**

TABLE 41.—*Frequencies of different periods recorded at Pasadena during earthquakes originating at different ranges of distance*

PASADENA P-PHASE, SHORT PERIOD INSTRUMENTS

Distance (km)	Periods in seconds																							
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50	11	**91**	**85**	38	52	18	4	3	1	2	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----
51 to 100	27	62	75	**115**	**127**	37	5	3	2	----	1	----	----	----	----	----	----	----	----	----	----	----	----	----	----
101 to 200	40	70	86	**106**	**148**	56	12	2	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----
201 to 300	8	34	41	58	**120**	**103**	84	59	25	40	13	14	12	3	5	9	4	1	----	2	----	----	----	----	----
301 to 400	1	9	9	10	**19**	**14**	3	----	----	2	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----
401 to 600	13	38	40	48	**111**	**98**	94	77	74	**98**	65	16	8	2	2	1	2	1	----	----	----	----	----	----	----
601 to 800	----	----	----	----	2	2	5	3	5	**11**	**13**	5	3	----	----	----	----	----	----	----	----	----	----	----	----
801 to 1,000	1	12	14	1	8	16	32	27	34	**93**	**91**	26	31	7	9	2	3	----	----	----	----	----	----	----	----
1,001 to 1,500	----	----	----	----	3	8	13	3	----	**12**	**15**	8	11	5	11	3	6	2	----	----	----	----	----	----	----
2,001 to 2,500	----	----	----	----	----	----	1	----	2	7	3	2	9	1	**25**	17	**27**	17	6	11	6	3	----	----	----
Total	101	316	350	381	**590**	352	253	177	147	**265**	200	71	74	18	**52**	32	42	21	6	13	6	3	----	----	----

PASADENA S-PHASE, SHORT PERIOD INSTRUMENTS

Distance (km)	Periods in seconds																							
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50	7	53	**76**	49	61	43	21	8	2	1															
51 to 100	4	16	31	67	**122**	91	45	20	8	8															
101 to 200	60	71	57	82	**128**	72	20	13	9	6	2														
201 to 300	2	8	8	29	**86**	**92**	69	58	35	**51**	32	25	19	12	12	5	1								
301 to 400		3	11	12	**16**	12	8	5	6																
401 to 600	3	12	14	26	68	**101**	69	68	83	**100**	85	35	30	14	19	4	1								
601 to 800					1	1	1		5	9	**11**	4	8	2	3	1				1	2				
801 to 1,000		7	10	6	6	5	9	10	17	43	**71**	37	**62**	28	26	22	16	8	1	1	1				
1,001 to 1,500						1			3	7	10	8	**12**	10	6	7	12	7	1	1	1				
2,001 to 2,500											2		2		1	2	4	10	8	7	27	14	11	2	
Total	76	170	207	271	**488**	**418**	242	182	168	**225**	213	109	133	66	67	41	34	25	10	12	35	14	11	2	

PASADENA M-PHASE, SHORT PERIOD INSTRUMENTS

Distance (km)	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50	8	28	21	33	**62**	54	53	38	19	21	15	5													
51 to 100	8	9	6	24	69	**91**	56	69	39	25	9														
101 to 200	25	54	49	62	**109**	**73**	39	28	11	15	9		1												
201 to 300	2	2	1	6	33	48	46	**59**	52	**72**	48	45	30	19	10	15	6	6	4	7	1				
301 to 400		6	9	**12**	**11**	7	8	**10**	5	7	2														
401 to 600	3	2	1	2	24	36	45	52	**93**	**110**	**105**	54	83	18	25	14	18	13	5	1					
601 to 800										1		1	3	1	5		2	7	1	1	**12**	3			
801 to 1,000						3		1	3	14	17	10	26	16	18	14	28	15	8	28	**115**	43	3		
1,001 to 1,500										4	3		7	4	6	6	8	2	5	3	7	6	3	**12**	
2,001 to 2,500																						14	**37**	30	5
Total	46	101	87	139	**308**	**312**	247	257	222	**269**	208	115	150	58	59	49	62	43	23	40	**135**	66	43	42	5

TABLE 42.—*Frequencies of different periods recorded at Riverside during earthquakes originating at different ranges of distance*

RIVERSIDE P-PHASE

Distance (km)	Periods in seconds																						
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 and more
0 to 50	**12**	5	2	1																			
51 to 100	**265**	**196**	118	67	35	15	1	9			4		2	2									
101 to 200	22	64	**89**	**84**	**100**	53	13	8	2														
201 to 300	4	7	16	**22**	**28**	14	6	3	2	3	1	2	1										
301 to 400	2	10	25	26	**46**	36	30	13	8	12	5	2	2	3									
401 to 600	5	13	19	39	**82**	50	56	**47**	41	**45**	39	13	20	6	2								
601 to 800				3	3	1	4	3	7	5	**20**	10	6	3	5	1							
801 to 1,000					7	8	13	7	12	**34**	26	18	10	7	6								
1,001 to 1,500			4	3	2	4	8	1	7	**13**	6	12	14	2	2								
1,501 to 2,000								1	1	2					**4**	3	2	2	1	3	3		
2,001 to 2,500							1	1	4	7	11	13	15	**28**	**27**	**26**	9	2	3	3			
Total	**310**	**295**	273	245	**303**	181	132	93	84	**121**	112	70	70	51	46	30	11	4	4	6	3		

RIVERSIDE S-PHASE

Distance (km)	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 and more
0 to 50	**6**	8	8	4	2	1																	
51 to 100	149	**302**	151	38	24	14	16	6	6	8	3	2	2	1									
101 to 200	37	60	64	51	**92**	34	13	6	5	7	3	2	2	1									
201 to 300	4	16	20	20	**47**	**41**	17	6	5	3	2	2	2										
301 to 400	4	1	6	16	**45**	**41**	21	15	8	19	12	7	5	5	5	3							
401 to 600		5	5	14	**54**	**57**	**46**	**47**	**42**	**42**	**40**	20	18	17	16	9							
601 to 800					1	5	9	**4**	**4**	5	**13**	9	5	5	3	1							
801 to 1,000			4	2						5	13	6	17	11	**20**	14	8	**4**	1	1	5	1	
1,001 to 1,500			2	1	6	2	1	1	2	3	2	4	5	**7**	8	**9**	2	3	1			1	3
1,501 to 2,000														3	3	5	**8**						
2,001 to 2,500		1	1		1	2	2	4			2		1	3	4	3	2	17	11	10	30	32	2
Total	200	**393**	261	146	**272**	197	125	89	72	**92**	90	52	57	53	59	44	20	24	13	11	35	34	5

RIVERSIDE M-PHASE

0 to 50	**10**	**14**	4	6	13	7		6	5	7													
51 to 100	98	**209**	112	70	64	49	29	18	29	27	18	19	19	8	2	4	3						
101 to 200	16	**52**	45	44	**70**	**63**	47	31	16	15	7	6	7	5	6	1	2	2		1			
201 to 300		1	5	11	**23**	**17**	10	5	3	4	2	1	3	3									
301 to 400	3	3	3	5	13	18	18	19	24	**37**	22	20	12	7	7	1	2						
401 to 600		2	5	13	16	26	20	28	43	**58**	34	32	24	27	34	16	9	2	5				
601 to 800						3	3	1	3	3	5	**9**	7	5	4	2	2			2	3	4	**6**
801 to 1,000										2	1		7	2	**10**	8	1	8	13	**22**	16	3	2
1,001 to 1,500								1	1		1	2	3	4	**13**	7	5	5		1	2	2	**17**
1,501 to 2,000						1					1	1	1				2						**15**
2,001 to 2,500															2	4	9		2	10	2	6	**63**
Total	127	**281**	174	149	**199**	**184**	127	109	124	**153**	91	90	83	61	**78**	43	35	17	20	36	23	15	**103**

TABLE 43.—*Frequencies of different periods recorded at Mount Wilson during earthquakes originating at different ranges of distance*

MOUNT WILSON P-PHASE

Distance (km)	Periods in seconds																								
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50	15	**114**	57	19	6																				
51 to 100	96	**237**	130	44	22	7																			
101 to 200	30	65	**83**	51	43	9	5	3	1																
201 to 300	8	54	**78**	**77**	**73**	32	7	15	10	19	4														
301 to 400		2	6	9	**16**	6																			
401 to 600	1	29	39	71	**148**	111	34	26	26	21	21	5	6	1											
601 to 800																									
801 to 1,000				12	15	15	23	22	11	**61**	**32**	9	10	2	1	1									
1,001 to 1,500					3	5	7	6	3	6	10	5	**12**	6	1										
2,001 to 2,500				1	1		3	2	1	14	17	8	13	11	27	23	**29**	**33**	5	1					
Total	160	**501**	393	284	**327**	185	76	74	52	**121**	84	27	41	20	29	24	29	33	5	1					

TABLE 43.—*Frequencies of different periods recorded at Mount Wilson during earthquakes originating at different ranges of distance*—Cont.

MOUNT WILSON S-PHASE

Distance (km)	Periods in seconds																								
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50	17	**75**	**82**	27	31	3		3																	
51 to 100	25	**170**	**188**	81	35	12				1															
101 to 200	33	46	**60**	**61**	**61**	21	7	3	3	2															
201 to 300	1	17	**47**	30	**86**	53	30	20	11	34	18	11	9	2	1	2	2	1							
301 to 400		4	2	6	**16**	8																			
401 to 600	3	3	9	29	85	**109**	66	54	38	36	23	7	27	8	4	3	4								
601 to 800																									
801 to 1,000				5	17	10	9	8	12	36	**55**	16	18	9	8	2	9	2							
1,001 to 1,500							1			2	2	2	6	5	8	**7**	8	**7**					1	**4**	3
2,001 to 2,500										2			1	1	4	4	10	20	14	27	**60**	19	1		
Total	79	315	**388**	239	**331**	216	113	88	64	113	98	36	61	25	25	18	33	30	14	27	60	19	2	4	3

MOUNT WILSON M-PHASE

Distance (km)	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50	8	**73**	**63**	32	39	21	6	7	1	7	5	4	4												
51 to 100	19	106	**138**	76	63	26	16	5	23	14	6		1												
101 to 200	8	34	54	57	**77**	40	12	10	4	3	2														
201 to 300		9	23	10	**39**	**43**	29	25	24	52	32	17	16	11	11	2	6	4	4						
301 to 400				2	8	**12**	8	4	6	6															
401 to 600		5	6	12	40	**60**	**54**	**47**	**61**	**57**	**60**	**53**	28	9	5		3	2	1	1					
601 to 800																									
801 to 1,000								1	3	11	**18**	**18**	**21**	11	**21**	10	15	5	9	15	**21**	**22**			
1,001 to 1,500										2			4	5	3	7	4	4	2				7	**14**	
2,001 to 2,500														3	2		6	5	6	5	27	**55**	43	10	6
Total	35	227	**284**	189	**266**	202	125	98	122	**152**	123	92	74	39	42	19	34	20	22	21	48	**77**	50	24	6

TABLE 44.—*Frequencies of different periods recorded at Santa Barbara during earthquakes originating at different ranges of distance*

SANTA BARBARA P-PHASE

Distance (km)	Periods in seconds																								
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50	8	**14**	8	3	6	1																			
51 to 100	3	**18**	3	8	3																				
101 to 200	6	30	75	176	**342**	155	61	22	9	12	5														
201 to 300		9	22	29	**69**	40	13	7	2	1															
301 to 400	2	14	29	29	**40**	27	16	4	2																
401 to 600		15	72	70	127	**141**	120	79	54	43	11						2								
601 to 800			1	6	**11**	5	2																		
801 to 1,000		2		8	2		2	3	7	**15**	2	2	1												
1,001 to 1,500					3	12	29	20	18	**42**	26	11	4	2	5		2	4							
1,501 to 2,000					2	2	1	1	3	**6**	1	2	1												
2,001 to 2,500					1	5	5	5	**13**	9	7	11	9	7	16	4	**26**	8	6						
Total	19	102	210	329	**606**	388	249	141	108	**128**	52	26	15	9	21	4	**30**	12	6						

SANTA BARBARA S-PHASE

Distance (km)	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50	3	5	6	4	**9**	6	3	2	1																
51 to 100	3	**8**	**7**	2	3	3	2	3																	
101 to 200	10	26	32	93	**255**	**235**	86	38	17	25	1	4	2												
201 to 300		**4**	8	17	27	**55**	37	24	8	12	8														
301 to 400			12	8	**33**	**30**	25	22	8	4	6														
401 to 600			34	23	76	91	**103**	91	92	**112**	65	25	22	7	9	3									
601 to 800					2	3		1	2	2	3			2	3	2	**5**								
801 to 1,000			1		1	2		1	1	5	3		3	9	3	2	1	1							
1,001 to 1,500			1		5	7	15	11	9	**24**	18	8	8	4	10	7	10	5	1	1	6			12	5
1,501 to 2,000									1	2	4	2	2	2	3		2	1							
2,001 to 2,500							1	2	1	1	2	1	4		4	5	6	17	17	21	**40**	6			
Total	16	43	101	147	411	**432**	273	195	140	**187**	110	40	41	24	32	19	24	24	18	22	46	6		12	5

TABLE 44.—*Frequencies of different periods recorded at Santa Barbara during earthquakes originating at different ranges of distance*—Contd.

SANTA BARBARA M-PHASE

Distance (km)	Periods in seconds																								
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50					2	6	5	**4**	**11**	**10**	6														
51 to 100	3	6	4	2	5		6	3	4	3															
101 to 200		8	19	23	161	**182**	172	125	75	52	34	8	6	1	2	1									
201 to 300		2	10	16	**45**	**46**	21	25	20	17	9														
301 to 400		1	1	3	13	10	19	24	**26**	24	14	3	3												
401 to 600		1	22	10	34	43	61	**103**	79	**102**	**105**	59	46	17	21	7	3								
601 to 800															3	5	**7**	**7**							
801 to 1,000											1		1	6	3	4		1	1	1	**10**	8	**12**		
1,001 to 1,500							1	3	4	8	9	5	12	8	10	18	**23**	14	4	2	2	2	1		
1,501 to 2,000																2	8	4	2						
2,001 to 2,500																	4	8	1	1	6	28	26	**37**	2
Total	3	18	56	54	**260**	**287**	**285**	**287**	219	216	178	75	68	32	39	37	45	34	8	4	18	38	39	37	2

TABLE 45.—*Frequencies of different periods recorded at Haiwee during earthquakes originating at different ranges of distance*

HAIWEE P-PHASE

Distance (km)	Periods in seconds																							
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4
0 to 50[1]	20	**35**	26	6	4																			
51 to 100	2	6	6	**7**	**7**																			
101 to 200	5	23	36	**37**	**42**	19	2	2		2														
201 to 300		44	81	107	**328**	**285**	183	116	78	85	104	64	21		1	2								
301 to 400			1	4	**15**	**10**	6	10	4	7	10	1	1											
401 to 600			9	25	**70**	**43**	19	24	25	33	18	6	3											
601 to 800			1	6	22	24	29	22	13	**39**	**43**	12	7	5	3	1								
801 to 1,000									1	7	8	4	7	2	2	1	1	1						

1,001 to 1,500			1	1	6	3	2	2	3	15	23	16	8	3										
1,501 to 2,000											6	1	1	5	6	4	4	1						
2,001 to 2,500							3	2	3	8	3	3	2	1	2	4	9	6						
Total	7	73	132	187	490	394	244	178	127	196	215	104	50	16	14	12	14	8						

HAIWEE S-PHASE

0 to 50[1]	12	35	38	14	21	1	1																	
51 to 100	1	5	3	6	10	4																		
101 to 200		12	20	23	47	41	20	15	2	1														
201 to 300		11	47	70	271	268	199	159	118	101	124	63	36	8	2									
301 to 400		3	3	1	4	3	4	6	6	7	12	9	2	5										
401 to 600				11	53	34	18	17	23	35	37	12	12											
601 to 800					16	31	32	21	19	33	34	12	6	4	3	4	3							
801 to 1,000											4	3	5	1	3	3	1				2	1	3	1
1,001 to 1,500									3	9	13	5	17	10	6	4	1					2	4	
1,501 to 2,000															2		4	3	2	3	7	3	1	
2,001 to 2,500																	1	6	5	11	10	8	1	2
Total	1	31	73	111	401	381	273	218	171	186	224	104	78	28	16	11	10	9	7	14	19	14	9	3

HAIWEE M-PHASE

0 to 50[1]	2	11	19	16	24	2	3	1																
51 to 100		2	3	6	9	4	3	1	2															
101 to 200		2	20	14	42	36	24	7	7	10	5	3												
201 to 300	2	10	28	31	139	176	193	156	132	226	189	94	56	17	5	1								
301 to 400				1	4	6	12	8	3	2	4	1												
401 to 600				5	25	16	17	16	20	48	49	14	19	4	5	4	1							
601 to 800					6	6	12	15	9	33	35	16	27	9	5	3	5	3		8	17	6		
801 to 1,000										2		3	3	2	3		1		1	1	9	5		
1,001 to 1,500													6	6	10	4	3	1	1		11	14	13	2
1,501 to 2,000																					3	5	11	8
2,001 to 2,500																	2	4	1		3	7	13	11
Total	2	14	51	57	225	244	241	203	173	321	282	131	111	38	28	12	12	8	3	9	43	37	37	21

[1] From records of shocks not listed in table 29. These figures are not included in the total.

TABLE 46.—*Frequencies of different periods recorded at Tinemaha during earthquakes originating at different ranges of distance*

TINEMAHA P-PHASE

Distance (km)	Periods in seconds																							
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4
0 to 50[1]	23	**41**	32	17	12																			
51 to 100		16	**18**	14	15																			
101 to 200	5	48	58	68	**123**	84	48	23	15	7	3													
201 to 300	2	10	49	42	**113**	56	25	12	8	11	24	18	6	4										
301 to 400	4	38	81	101	**283**	165	89	29	8	11	3	3	6											
401 to 600		2	5	15	**64**	52	37	28	16	20	11													
601 to 800			3	6	22	**34**	**35**	22	16	**29**	22	7	6		1									
801 to 1,000				3	**4**	2	1	2		1			1		**4**	**3**	**4**							
1,001 to 1,500					1	4	3	1	3	11	**18**	**10**	**17**	4	4	4	4	5						
1,501 to 2,000							4			7	6	5	6	3	**10**	1	4	1						
2,501 to 3,000							1	5	2	7	2	1	3	3	5	4	**21**	13	10	1				
Total	11	114	214	249	**625**	397	243	122	66	**104**	89	44	45	14	24	12	33	19	10	1				

TINEMAHA S-PHASE

Distance (km)	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4
0 to 50[1]	5	21	34	44	**82**	7																		
51 to 100	2	6	7	12	**21**	15	4																	
101 to 200		25	22	51	**78**	**79**	57	41	20	31	24	8	6	4										
201 to 300	9	17	24	32	**77**	57	45	37	28	28	31	11	5											
301 to 400		2	23	36	**193**	**209**	164	64	24	23	13	9	3		2	1	2							
401 to 600				2	18	28	29	27	24	**56**	30	17	9	1	3		1							
601 to 800				5	**32**	**30**	23	16	11	**29**	17	7	10	5	4	1	3	3	2					
801 to 1,000								1	1				2		**6**	**5**	4	2						
1,001 to 1,500					1	2			1	8	1	3	**13**	**9**	**13**	**12**	10	5	3	3	1			
1,501 to 2,000													1	4	3	3	8	6	3	**9**	2		2	
2,501 to 3,000															4	1	**16**	**17**	15	9	**17**			
Total	11	50	76	138	**420**	**420**	322	186	109	**175**	116	55	49	23	35	23	44	33	23	21	20		2	

TINEMAHA M-PHASE

0 to 50[1]		2	7	19	**54**	34	7	4	1	2														
51 to 100		5	5	2	6	8	4	5	10	**16**	4	2												
101 to 200		8	23	24	54	**59**	50	44	40	43	**49**	22	26	11	10	1	7	9	3	1				
201 to 300		**4**	11	30	**70**	42	41	30	24	31	20	21	17	7	7	3	3		1					
301 to 400	2	8	20	20	109	168	**191**	94	59	39	18	11	6	1	7	4		4	1	2				
401 to 600			2	2	14	17	23	22	21	**36**	17	12	17	11	8	2	3	1	2					
601 to 800					6	6	12	14	7	**27**	19	21	24	13	9	10	10	10	4	4				
801 to 1,000													1		5	3	8			1				
1,001 to 1,500												1	2	3	**14**	**13**	10	10	3	3	8	9	8	
1,501 to 2,000														1	1	2	2	1	1		1	4	**15**	6
2,501 to 3,000																	4	6	2	4	**22**	**19**	8	8
Total	2	25	61	78	259	**300**	**321**	209	161	192	127	90	93	47	61	38	47	41	17	15	31	32	31	14

[1] From records of shocks not listed in table 29. These figures are not included in the total.

TABLE 47.—*Frequencies of different periods recorded at La Jolla during earthquakes originating at different ranges of distance*

LA JOLLA P-PHASE

Distance (km)	Periods in seconds																								
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50[1]	8	**19**	**15**	3																					
51 to 100	4	**7**		8	6																				
101 to 200	40	153	**222**	**204**	128	40	11		1	5	7		1												
201 to 300		4	9	**13**	**20**	3	1																		
301 to 400	3	9	16	20	**36**	18	3	2	3	1															
401 to 600		8	39	30	**44**	12	9	8	10	12	11	3													
601 to 800		2	13	10	34	33	57	82	61	**131**	**153**	52	48	10	10	12	1	2	1						
801 to 1,000						1	2			**7**	**7**	4	3	2	1	1									
1,001 to 1,500				3		14	28	34	19	**44**	34	6	6	6	11	2	2	2	2						
2,001 to 2,500								3	2	11	5	9	16	6	15	**26**	17	13	2	4					
Total	47	183	**299**	**288**	**268**	121	111	129	96	**211**	**217**	74	73	24	37	41	20	17	5	4					

TABLE 47.—*Frequencies of different periods recorded at La Jolla during earthquakes originating at different ranges of distance*—Continued

LA JOLLA S-PHASE

Distance (km)	Periods in seconds																								
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50[1]	2	11	**17**	9	4																				
51 to 100			2	**9**	8	1																			
101 to 200	24	81	[illegible]	**172**	**182**	87	34	13	10	1	3	1													
201 to 300		3	7	9	**24**	12	3																		
301 to 400		1	1	16	**39**	26	8	4	1																
401 to 600		1	17	12	**30**	**26**	**27**	23	11	21	7	3	1	1	1										
601 to 800			3		11	27	42	50	55	94	**124**	82	103	31	26	19	12	1			3				
801 to 1,000											3		6	8	3	3	1					2			
1,001 to 1,500					1	3	13	13	7	35	**50**	13	15	9	16	13	20	5	3	1					
2,001 to 2,500									1	1	1		3	6	11	**20**	8	11	12	15	**22**	5	3		
Total	24	86	170	218	**295**	182	127	103	85	**152**	**188**	99	128	55	57	55	41	17	15	16	25	7	3		

LA JOLLA M-PHASE

Distance (km)	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	1.7 to 1.8	1.9 to 2.0	2.1 to 2.4	2.5 to 2.8	2.9 to 3.2	3.3 to 3.6	3.7 to 4.4	4.5 to 6.4	6.5 to 8.4	8.5 to 10.4	10.5 to 15.4	15.5 to 20.4
0 to 50[1]		8	**12**	9	6	3																			
51 to 100		3	3	**7**	6																				
101 to 200	9	14	43	86	**205**	155	118	85	35	22	7	3	2	2	1										
201 to 300		4	5	8	**14**	**12**	3	2	2	2															
301 to 400		1	3	3	**12**	6	5	10	10	**13**	5	7													
401 to 600			6	4	9	15	19	18	14	**30**	13	6	4	4	10	5	5	1							
601 to 800	1	6	1	2	10	16	24	34	29	62	**127**	65	95	53	53	54	41	16	9	4	4				
801 to 1,000											3	2	**5**	**4**	**4**						3	4	2		
1,001 to 1,500							3	1	4	**19**	16	8	18	10	19	24	**28**	8	1	2	20	6	10	4	2
2,000 to 2,500													1	1	3		7			2	9	27	13	**34**	8
Total	10	28	61	110	**256**	204	172	150	94	**148**	**171**	91	125	74	90	83	81	25	10	8	36	37	25	38	10

[1] From records of shocks not listed in table 29. These figures are not included in the total.

TABLE 48.—*Prevailing periods*

Station	Distance in kilometers										
	0 to 50	51 to 100	101 to 200	201 to 300	301 to 400	401 to 600	601 to 800	801 to 1,000	1,001 to 1,500	1,501 to 2,500	≧2,501
	Seconds	*Seconds*	*Seconds*	*Seconds*	*Seconds*	*Seconds*	*Seconds*	*Seconds*	*Seconds*	*Seconds*	*Seconds*
P-PHASE											
Pasadena, long	0.2-0.4	0.4	0.6	0.6	0.6	0.8-1.0	1.0	1.0	2.0	----------	2.5-5.0
Pasadena, short	0.2-0.3	0.4-0.5	0-4-0.5	0.5-0.6	0.5-0.6	0.5-0.6	1.0-1.2	1.0-1.2	1.1	----------	2.2
Mount Wilson	0.2	0.2	0.3	0.3-0.5	0.5	0.5	----------	1.0-1.1	1.5	----------	3
Riverside	0.1	0.1-0.2	0.3-0.5	0.4-0.5	0.5	0.5	1.1	1.0-1.1	1.0	2	2
La Jolla	[1] 0.2-0.3	0.2; 0.4	0.3-0.4	0.4-0.5	0.5	0.5	1.0-1.2	1.0-1.2	1.0-1.1	----------	2.2
Santa Barbara	0.2	0.2	0.5	0.5	0.5	0.6	0.5	1.0	1.0	1.0	0.9; 2.8
Tinemaha	[1] 0.2-0.3	0.2-0.5	0.5	0.5	0.5	0.5	0.6-0.7; 1.0	(0.5); 2-2.5	1-1.5	2	3
Haiwee	[1] 0.2	0.2-0.5	0.3-0.5	0.5-0.6	0.5-0.6	0.5-0.6	0.7; 1.0-1.2	1.0-1.2	1.1	(2)	1-2.8
S-PHASE											
Pasadena, long	0.4	0.6	0.6	0.6	1.0	2.0	1.5	2.0	4.0	----------	4.0
Pasadena, short	0.3	0.5	0.5	0.5-0.6	0.5	0.6; 1.0-1.1	1.1	1.1; 1.5	1.5	----------	3
Mount Wilson	0.2-0.3	0.2-0.3	0.3-0.5	0.5	0.5	0.6	----------	1.1	2	----------	5
Riverside	0.1-0.3	0.2	0.5	0.5-0.6	0.5-0.6	0.5-1.2	1.2	1-2	2.5	3	5
La Jolla	[1] 0.3	0.4-0.5	0.4-0.5	0.5	0.5	0.5-0.7	1.1	1.7	1.1	----------	2.2; 5
Santa Barbara	0.5	0.2-0.3	0.5-0.6	0.6	0.5-0.6	0.7; 1.0	2.8	1.8	1.0	(1.1)	5
Tinemaha	[1] 0.5	0.5	0.5-0.6	0.5	0.5-0.6	(0.6-0.7); 1.0	0.5-0.6; 1.0	2	1.5-2	(2.8; 4)	3; 5
Haiwee	[1] 0.2-0.3	0.4-0.5	0.5-0.6	0.5-0.6	1.1	0.5; 1.0-1.2	0.6-0.7; 1.1-1.2	1.5; (10)	1.6	6	4-6
M-PHASE											
Pasadena, long	0.6	1.0	0.6	0.6; 1.0	1.0	1.0; 2.0; 4.0	3.0	5.0	7; 12	----------	12
Pasadena, short	0.5-0.6	0.6	0.5-0.6	0.8; 1.0	0.4-0.5	1.0	5	5	(12)	----------	10
Mount Wilson	0.2-0.3	0.3	0.5	0.5-0.6	0.6	0.6-1.2	----------	1-2; 5-8	12	----------	7-10
Riverside	0.1-0.2	0.2	0.2; 0.5-0.6	0.5-0.6	1.0	1.0	1.0-1.5	2	2; 8	8	8
La Jolla	[1] 0.3	0.4	0.5	0.5-0.6	0.5; 1.0	1.0	1.0	1.5-2	2-3	----------	12
Santa Barbara	0.2; 0.6; 0.9	(0.2); (0.7)	0.6-0.7	0.5-0.6	0.9	0.8; 1.0-1.2	2.5-3	4.5-10	(2.5)	2.8	9-15
Tinemaha	[1] 0.5	(0.6); 1.0	0.6; 1.1	0.5	0.6-0.7	(0.7-0.9); 1.0	1.0; (1.5)	(2.8)	2	10	5-8
Haiwee	[1] 0.3; 0.5	0.5	0.5-0.6	0.6-0.7; 1.0	0.7	1.0-1.2	1.0-1.2	6	5-10	10-12	10-12

[1] From records of shocks not listed in table 29.

We may therefore conclude either that these free periods are the fundamental period and its first harmonic (or two higher harmonics) of free vibrations of a layer which extends throughout the whole region under consideration, or that we have everywhere two single layers of different thickness, the period of 0.5 second indicating free vibrations of the thinner layer and the period of 1.1 seconds the free vibrations of the thicker layer.

It may be noticed, finally, that for periods less than 1.5 seconds there is no great difference between the tabulation for the short period instruments and the long period instruments at Pasadena.

THE EFFECT OF THE STATION

There is a distinct difference between the short periods prevailing at distances less than 50 kilometers at different stations. This should be attributed, therefore, to free vibrations of thinner layers which differ from station to station.

THE EFFECT OF THE EPICENTER ON THE RECORDED PERIODS

As may be seen from tables 32 to 38 there can be no large differences due to effects at the epicenter or the wave paths, as the periods measured from different shocks do not differ very widely if the distances are about the same. There are several possibilities for investigating the effect of the epicenter or wave path on the recorded periods. One would be to compare the numbers of periods measured in shocks from about the same distance but from epicenters in different directions. Another would be to compare the periods measured from a group of shocks originating in the same region with the average of all shocks originating at about that distance. The latter procedure must be used very carefully as the average of all shocks for a distance contains, of course, the shocks under consideration, and if they form a large percentage of these shocks the average will be affected considerably by the shocks under consideration.

Both methods were used in the following investigation. Ten regional groups of earthquakes were selected from those listed in table 30, as follows:

Group nos.	*General region*	*Earthquake nos.*
1	Mojave Desert	4 to 8, inclusive.
2	Parkfield	13, 22 to 25.
3	Santa Monica	10, 21, 28, 29.
4	Brawley	30 to 38.
5	Long Beach	56 to 75, 89.
6	Whittier	76 to 86.
7	Nevada, nearby	88 to 96, 108.
8	Nevada, distant	97 to 102.
9	San Bernardino Mountains	1 to 3.
10	Santa Barbara	14, 15, 107.

Comparisons of periods (1) between individual groups, and (2) between single groups and average values are shown in table 49. The more frequent periods are shown in bold face type. The following comments are made on the various tabulated comparisons, of which table 49 gives some examples:

TABLE 49.—*Frequencies observed at various stations for selected regional groupings of earthquakes*

[The tabulated frequencies are expressed in percentage of the total number of periods measured in each horizontal group. Read text for comment on comparisons. The most frequent periods are shown in boldface type]

Comparison no.	Group no.	Observing stations and epicentral regions	Average or range of epicentral distances	Phase	Periods in seconds													
					0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	≧1.7
		RIVERSIDE	*km*															
1	1	Mojave Desert	75–146	M	8	**22**	23	**24**	16	6	1	0	0	0	0	0	0	0
		Various epicenters	101–200	M	4	12	10	10	**16**	**14**	11	7	4	3	2	1	2	4
		PASADENA																
2	1	Mojave Desert	117–173	M	1	18	19	19	**24**	13	3	3	1	0	0	0	0	0
		Various epicenters	101–200	M	5	11	10	13	**23**	15	8	6	2	3	2	0	0	0
		SANTA BARBARA																
3	1	Mojave Desert	245–300	M	0	1	10	12	**29**	23	10	7	6	1	0	0	0	0
		Various epicenters	201–300	M	0	1	5	8	21	**22**	10	**12**	10	8	4	0	0	0
		TINEMAHA																
4	1	Mojave Desert	235–300	M	0	2	3	13	**40**	14	14	8	4	4	0	0	0	0
		Various epicenters	201–300	M	0	1	3	8	**19**	12	11	8	7	9	6	6	5	4
		SANTA BARBARA																
5	2	Parkfield	168–179	S	0	2	9	9	**24**	15	12	11	4	12	0	2	0	0
		Various epicenters	101–200	S	1	3	4	11	**31**	**29**	11	4	2	3	0	1	0	0
6	2	Parkfield	168–179	M	0	0	2	3	**15**	12	12	12	10	**15**	9	3	4	3
		Various epicenters	101–200	M	0	1	2	3	**19**	**21**	**20**	14	8	6	4	1	1	0
		TINEMAHA																
7	2	Parkfield	235–350	M	0	1	3	5	**11**	7	7	9	9	**15**	11	7	6	9
		Various epicenters	201–300	M	0	1	3	8	**19**	12	11	8	7	**9**	6	6	5	5
		PASADENA																
8	3	Santa Monica	50	M	8	**12**	6	9	**26**	13	9	9	7	1	0	0	0	0
	5	Long Beach	60	M	2	2	1	5	17	**24**	14	17	10	7	2	0	0	0

TABLE 49.—*Frequencies observed at various stations for selected regional groupings of earthquakes*—Continued

[The tabulated frequencies are expressed in percentage of the total number of periods measured in each horizontal group. Read text for comment on comparisons. The most frequent periods are shown in boldface type]

Comparison no.	Group no.	Observing stations and epicentral regions	Average or range of epicentral distances	Phase	Periods in seconds: 0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6	≧1.7
		MOUNT WILSON	*km*															
9	3	Santa Monica	55	M	4	9	14	**27**	**26**	6	7	4	1	1	0	0	0	0
	5	Long Beach	70	M	4	**24**	**30**	13	10	5	3	1	5	3	2	0	0	0
		SANTA BARBARA																
10	2	Parkfield	176	M	0	0	2	3	**15**	12	12	12	10	**15**	9	3	4	3
	5	Long Beach	182	M	0	1	0	0	13	**23**	**25**	20	11	5	3	0	0	0
		TINEMAHA																
11	3	Santa Monica	369	P	0	1	4	13	**44**	24	8	4	3	0	0	0	0	0
	5	Long Beach	379	P	0	5	11	12	**31**	21	11	4	0	2	1	1	1	0
12	3	Santa Monica	369	M	0	0	9	4	21	**27**	17	8	1	1	4	6	0	1
	5	Long Beach	379	M	0	2	1	3	13	22	**28**	13	9	5	2	0	1	1
		LA JOLLA																
13	3	Santa Monica	144	P	3	19	**24**	**22**	**30**	3	0	0	0	0	0	0	0	0
	6	Whittier	137	P	0	19	**24**	**20**	**22**	9	5	0	0	0	0	0	0	0
14	3	Santa Monica	144	M	8	4	10	18	**26**	16	13	2	1	2	0	0	0	0
	6	Whittier	137	M	0	3	17	15	**22**	15	15	5	5	4	0	0	0	0
		HAIWEE																
15	2	Parkfield	231	S	0	1	5	4	**25**	**22**	10	11	3	5	8	4	2	0
	3	Santa Monica	266	S	0	2	6	11	19	**48**	12	0	2	0	0	0	0	0
	5	Long Beach	275	S	0	1	3	6	**25**	**19**	14	9	8	7	6	2	1	0
	7	Nevada, nearby	250	S	0	1	2	4	**12**	**15**	**13**	**12**	9	8	11	8	4	1
	10	Santa Barbara	255	S	0	7	7	0	9	9	**23**	18	18	7	2	0	0	0
16	2	Parkfield	231	M	0	0	2	2	8	7	11	11	8	**23**	14	6	6	3
	3	Santa Monica	266	M	0	0	2	8	**23**	15	4	9	**15**	**15**	9	0	0	0
	5	Long Beach	275	M	0	0	1	2	13	**17**	14	11	9	**14**	10	5	2	1
	7	Nevada, nearby	250	M	0	0	1	1	7	10	**13**	12	9	**14**	16	8	5	3
	10	Santa Barbara	255	M	0	0	0	0	10	13	15	**21**	18	**21**	3	0	0	0

		TINEMAHA																
17	2	Parkfield	239	M	0	1	3	5	**11**	7	7	9	9	**15**	11	7	6	9
	8	Nevada, distant	203	M	0	0	0	0	4	**10**	7	7	9	12	**17**	15	12	8
		HAIWEE																
18	8	Nevada, distant	308	M	0	0	0	0	2	3	**11**	6	6	22	**27**	5	8	0
		Various epicenters	201–400	M	0	0	0	2	10	15	**29**	20	7	5	10	2	0	0
		RIVERSIDE																
19	5	Long Beach	70	M	11	**21**	10	7	8	8	5	4	6	5	4	4	4	3
	6	Whittier	58	M	13	**44**	25	9	6	2	1	0	0	0	0	0	0	0
	9	San Bernardino Mountains	50	M	11	**20**	7	12	**21**	10	0	7	6	8	0	0	0	0
		LA JOLLA																
20	9	San Bernardino Mountains	157	S	0	0	0	18	**42**	**32**	3	3	2	0	0	0	0	0
		Various epicenters	101–200	S	3	11	19	**23**	**24**	13	5	2	1	0	0	0	0	0

Group 1—(Comparisons 1 to 4).—Mojave Desert region. At most stations the periods measured are below average for all phases. Long periods are often missing.

Group 2—(Comparisons 5 to 7).—Parkfield region. The observed periods are in general slightly larger than the average corresponding to this distance. At Santa Barbara noticeably more periods of about 1.0 second are observed than in the average case, but fewer periods of 0.5 second. This illustrates well the fact, which we had found in investigating the effect of distance on the recorded periods, that usually the dominant periods change with the distance not gradually but in jumps, and that periods of about 0.5 and 1.0 second prevail. Results at Tinemaha show a similar effect.

Groups 3 and 4.—Santa Monica and Brawley. Earthquakes in the Santa Monica and Brawley areas have periods which everywhere correspond to about the average for the corresponding distances.

Group 5—(Comparisons 8 to 12).—The aftershocks of the Long Beach earthquake were so numerous that the average periods of all shocks, including those at other corresponding distances, are largely controlled by the periods of the aftershocks. No large differences can therefore be expected. In order to avoid spurious conclusions we compare in this case the shocks of one group with the shocks of another group originating at about the same distance from the station under consideration. The comparisons do not show any large differences.

Group 6—(Comparisons 13 and 14).—Whittier region. The observed periods correspond fairly closely to the average.

Group 7—(Comparisons 15 and 16).—Nearby shocks in Nevada. In general the periods agree about with the averages although the tabulations indicate slight differences.

Group 8—(Comparisons 17 and 18).—Distant shocks in Nevada. The Tinemaha records show about the same distribution of periods as the Parkfield shocks. Periods of about 1.1 seconds have been found very much more frequently at Haiwee than is usual at distances of 308 kilometers where periods of about 0.7 second prevail. This indicates again a jump in period as previously mentioned.

Group 9—(Comparisons 19 and 20).—San Bernardino Mountains. The periods are in general slightly longer than the average.

Group 10.—Santa Barbara region. The observed periods correspond to about the average.

As a whole we see that in most cases the effect of epicenter or wave path, which usually can hardly be separated, is relatively small. The differences may be explainable partly by differences in intensity in some cases. The periods from epicenters of a specific group may really differ somewhat from the average. The most interesting fact which we found is a confirmation of our previous result, that the periods prefer certain values and that the dominant periods jump if the distance passes a certain critical value. The periods observed less frequently correspond to the forced vibrations of the ground.

THE EFFECT OF MAGNITUDE OF THE SHOCK

In addition to the normal periods longer periods are observed usually in shocks where the original movement extends over a large area. Unfortunately, very little data are available in our material with which to investigate the effect of magnitude. The only region from which we have data concerning close-by shocks with different magnitudes is the Long Beach area. The main Long Beach shock was recorded clearly in detail in all phases at Pasadena only by the strong-motion instrument. The record shows, principally, waves with periods between 0.2 and 0.4 second riding on waves with longer periods—4 to 5 seconds and 8 to 10 seconds. There is no doubt that the short-period waves carried very much greater energy than the long-period waves so that they were really responsible for the damage. No waves with periods between 1.1 and a few seconds stand out in this record. Generally in the strong-motion records of the aftershocks only periods of a few seconds are recorded clearly, but on the records of the other instruments waves with periods of about 0.5 second are recorded frequently in the P- and S-phases while in the M-phase waves of about 1 second prevail. A comparison of the main shock with the aftershocks indicates, therefore, that both groups had about the same short periods, but that in the main shock there was, besides, a larger number of waves with periods of a few seconds which were very much less clear or missing in the aftershocks.

At La Jolla, Riverside, and Santa Barbara the records of the main shock do not show any details. At Haiwee (distance 276 kilometers) the periods observed during the main shock correspond approximately to those measured in the aftershocks except that the number of periods over 0.5 second which have been measured is relatively larger. The same is true concerning Tinemaha (distance 380 kilometers) where in the M-phase of the main shock, particularly, only periods of 1 second or more have been measured.

Similar phenomena, but less clear, have been found in other cases, so that it seems that longer periods beside the normal short periods are produced in strong shocks.

THE PERIODS OBSERVED IN BLASTS

The investigation of periods observed in blasts is of interest as it is possible to investigate the free periods in a certain region by blasting provided the periods so found correspond to those measured in earthquakes.

A list of blasts has been given in table 30. The distances in most cases have been taken from studies on the records of blasts in southern California by H. O. Wood and C. F. Richter.[1] The periods which have been measured in the records of the blasts are to be found in table 50. No blasts have been recorded at Tinemaha and Haiwee. Table 50 gives results similar to those found in the study of earthquakes except that the longer periods are less frequent. This agrees with our previous result that usually the longer periods are the more outstanding the larger the energy involved in the source. To produce waves with periods over 0.1 second, which are of interest to the engineer, a certain minimum of energy is needed, and, therefore, in most cases an explosion of e. g. 100 pounds of dynamite will not give an energy strong enough to investigate the frequencies of periods in the desired range. The short periods of 0.2 and 0.3 second are clearly prevailing at all stations, and the periods of 0.5 second and sometimes also 0.6 second show a clear maximum, especially in the S- and M-

[1] H. O. Wood and C. F. Richter, A Study of Blasting Recorded in Southern California (Bulletin of the Seismological Society of America, vol. 21, 1931, p. 28).
A Second Study of Blasting Recorded in Southern California (Bulletin of the Seismological Society of America, vol. 23, 1933, p. 95).

phases at large distances. A larger number of periods of about 1 second is indicated in the M-phase at Pasadena and Riverside. The long-period instruments at Pasadena show in each phase, in more than 70 percent of all cases, periods between 0.4 and 0.6 second. Only in the M-phase there is, besides, a slight maximum for periods around 1.0 second.

TABLE 50.—*Periods observed in blast records*

Station	Phase	Distance	Periods in seconds												
			0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4	1.5 to 1.6
		km													
Pasadena, short period	P	17–49	8	**63**	**53**	16	37	15	7	1	----	----	----	----	----
	P	67–98	----	23	**31**	15	15	5	1	----	----	----	----	----	----
	P	112	----	**10**	2	2	4	1	----	----	----	----	----	----	----
	S	17–49	3	36	**46**	30	**48**	34	14	8	5	5	4	----	----
	S	67–98	1	17	34	15	16	4	1	----	----	----	----	----	----
	S	112	----	6	4	1	2	1	4	----	----	----	----	----	----
	M	17–49	2	26	**32**	17	32	**48**	30	16	6	**22**	13	3	2
	M	67–98	2	6	16	14	**18**	9	4	2	----	----	----	----	----
	M	112	----	----	----	----	4	4	6	4	1	3	1	----	----
Mount Wilson	P	20–47	29	**40**	26	4	3	3	1	----	----	----	----	----	----
	P	85	7	**21**	12	1	----	----	----	----	----	----	----	----	----
	S	20–47	16	**34**	17	8	**14**	**15**	4	3	2	1	1	----	----
	S	85	----	10	**20**	5	6	----	----	----	----	----	----	----	----
	M	20–47	11	**29**	9	8	**19**	15	11	8	3	3	1	2	----
	M	85	----	5	**14**	6	7	2	----	----	----	----	----	----	----
Riverside	P	1–9	1	**14**	4	----	----	----	----	----	----	----	----	----	----
	P	44–85	18	**66**	45	17	16	7	1	----	----	----	----	----	----
	P	158	----	----	2	----	**9**	4	2	2	1	1	1	----	----
	S	1–9	3	**11**	8	3	**7**	3	4	----	----	----	----	----	----
	S	44–85	6	**38**	**38**	28	**36**	19	4	2	1	2	----	----	----
	S	158	----	----	----	1	**6**	4	4	2	4	----	----	----	----
	M	1–9	----	1	4	----	4	4	2	2	1	----	----	----	----
	M	44–85	5	**33**	**32**	16	**37**	25	9	2	----	3	----	----	----
	M	158	----	----	----	----	3	1	3	3	1	**6**	2	----	----
Santa Barbara	P	56	5	**17**	8	5	7	----	----	----	----	----	----	----	----
	P	145–176	----	7	**20**	**15**	**18**	8	5	----	----	----	----	----	----
	S	56	----	2	**17**	6	9	2	----	----	----	----	----	----	----
	S	145–176	----	1	7	6	**22**	**20**	10	4	3	2	----	----	----
	M	56	----	9	8	6	**12**	3	3	2	----	----	----	----	----
	M	145–176	2	2	3	6	**14**	**17**	**13**	7	2	1	----	----	----
La Jolla	P	162–195	----	5	**17**	5	9	5	3	----	----	----	----	----	----
	S	162–195	----	3	6	7	**13**	8	1	**4**	1	----	----	----	----
	All	1	4	**21**	11	6	4	----	----	----	----	----	----	----	----
	M	162–195	----	----	2	5	**11**	**12**	7	5	----	1	----	----	----

THE PERIODS RECORDED DURING MICROSEISMS

The measurements of the microseisms during the days which are listed in table 31 showed results which are listed in table 51. Again the same periods prevail as in earthquakes and blasts except that in this case the number of the longer periods is still more reduced than in the records of the blasts.

The records of the long-period instruments as Pasadena showed the following frequencies:

Periods in seconds	*Number*
0.1, 0.2	2
0.3	30
0.4	58
0.5	**189**
0.6	53
0.7–0.9	11
1.0–1.2	9
1.3–2.0	13
2.1–4.4	7
4.5–6.4	49
6.5–8.4	**125**
8.5–10	3

This tabulation shows again that periods of 0.4 to 0.6 second prevail by far. A second strong maximum occurs for periods between 6.5 and 8.5 seconds. The most remarkable result is that there are so few periods around 1 second recorded, although waves with this period are somewhat more frequently recorded by the short-period instruments at Pasadena (see table 51). It seems as if there are no vibrations with periods in the neighborhood of 1 second arriving in microseismic movements at the region near Pasadena.

TABLE 51.—*Microseisms*

NUMBERS OF OBSERVED PERIODS

Station	Periods in seconds											
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4
Pasadena, short period	1	32	**114**	48	**80**	49	30	8	6	**14**	10	4
Mount Wilson	67	**131**	50	17	**4**	------	------	------	------	------	------	------
Riverside	8	45	**162**	97	29	1	------	------	------	------	------	------
Santa Barbara	11	40	**26**	87	38	6	------	------	------	------	------	------
La Jolla	43	**109**	**179**	32	9	2	1	------	------	------	------	------
Tinemaha	3	97	**135**	59	39	16	9	4	------	3	------	------
Haiwee	9	22	**51**	40	**65**	**44**	22	9	7	3	2	1

PERCENTAGE OF OBSERVED PERIODS

Station	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4
Pasadena, short period	0.3	8	**29**	12	**20**	12	7.5	2	1.5	3.5	2.5	1
Mount Wilson	25	**49**	18.5	6	1.5	------	------	------	------	------	------	------
Riverside	2	13	**47**	28	8.5	0.5	------	------	------	------	------	------
Santa Barbara	4	13	**41**	28	12	2	------	------	------	------	------	------
La Jolla	16	**40**	29	12	3	0.5	0.5	------	------	------	------	------
Tinemaha	1	27	**37**	16	11	4	2.5	1	------	1	------	------
Haiwee	3	8	**19**	15	**24**	16	8	3	2.5	1	0.5	0.5

THEORY

When an earthquake occurs, waves approaching more or less sinusoidal type with periods of between a very small fraction of 1 second and ½ minute or even more are produced. The very long waves, according to B. Gutenberg and C. F. Richter, apparently are connected with the movements of large blocks in the earth's crust. They do not depend to a noticeable extent on the depth of the focus, as earthquakes originating at the same place in regard to location as well as to depth in some cases show very long periods in all phases together with short periods and in other cases only short-period waves. In the first case there is usually some indication of block movement extending over a very much larger area than in the second case. The waves on their path very probably change their periods [2] due to the viscosity of the material in which they are propagated and to scattering. Another factor which may be involved is the different absorption of the energy for waves with different periods. It is a general belief that short waves undergo considerably stronger absorption than long waves, but it seems that this difference has been deduced merely from the observed fact that at large distances shorter earthquake waves disappear faster than long-period waves. Therefore, if one tries to explain the later fact by presuming larger absorption for the short-

[2] The word "period" in this section is defined as twice the time between 2 succeeding passings of the rest line regardless of the form of the wave.

period waves, a vicious circle may result. On the other hand, it seems very likely that the periods of all types of waves increase with distance due to the viscosity of the material in which they are propagated. The first theoretical investigations were published by K. Sezawa.[3] Sezawa developed his theory for certain cases of surface waves. In order to apply his formulas, the wave form must be known. Using some cases where single waves or half waves form the original disturbance, B. Gutenberg[4] found that in the case of the surface waves investigated by Sezawa a wave at the distance D from the source to a first approximation should have the period

$$T=\sqrt{T_0^2+\frac{aD}{V^3}}$$

where T is the period at the distance D, T_0 the period at the origin, V the velocity of the wave and a a constant depending on the viscosity of the material. This formula supposes a single wave and plane surface waves. The investigation of this problem has not been extended to body waves. On the other hand, it is not correct to use it in the case of a wave train. If, therefore, the given formula for T is applied to such cases, it is more or less an empirical formula. In the case of microseisms and surface waves, and in seismic field work where the periods are very much smaller than in earthquakes the change of periods with distance agrees very well with the formula. Besides, the coefficient of viscosity for the crustal part of the earth, which has been derived by using observed periods in earthquake surface waves, agrees well with values found in other ways.[5] The factor a in the latter investigation has been found to be about 1 for surface waves in earthquakes if all quantities are measured in seconds and kilometers.

Supposing that the formula is an approximation for body waves too, we find that it agrees fairly well with our observations of periods in the local shocks. There is some uncertainty as to what values should be used for the velocity, for the longitudinal as well as the transverse waves pass through layers where the velocity changes quite rapidly with depth. Assuming a velocity of longitudinal waves of 6 kilometers per second for short distances and of 8 kilometers per second for distances of 500 kilometers or more, we find that the periods of longitudinal waves should increase about in the following way if they are very short at the epicenter:

Distance in kilometers	50	100	500	1,000	2,000
Period in seconds	1/3	1/2	1	1½	2

In a similar way we find, theoretically, for transverse waves the following increase in period:

Distance in kilometers	50	100	500	1,000	2,000
Period in seconds	1	2	2½	3	5

The increase in period of the surface waves should be still faster.

The calculated results correspond well to the general trend of the observed periods in all three cases. The caculations indicate that a faster increase in period should be expected in the surface waves than

[3] K. Sezawa, On the decay of the waves in visco-elastic solid bodies (Bulletin of the Earthquake Research Institute, Tokyo, vol. III, 43, 1927).
[4] B. Gutenberg, Handbuch der Geophysik, vol. IV, p. 22.
[5] B. Gutenberg and H. Schlechtweg, Viskosität und innere Reibung fester Körper (Physikalische Zeitschrift, vol. 31, 1930, p. 745).

in the transverse waves and in the transverse waves than in the longitudinal waves. Both conclusions correspond to the observed facts. The increase in period seems to be slightly smaller than it has been calculated. Better agreement between observations and calculations may be found by using a slightly smaller value of a. The problem of increase in period at still greater distances needs further investigation.

The striking difference between the observations and the calculations is the fact that the theoretical method gives a gradual increase in period with distance, while the observations very clearly show abrupt changes in period at certain distances. If we plot the observed and the calculated curves, we find that the observed values give a step-like curve for which the calculated curve is a continuous approximation. The simplest explanation for this observed phenomenon may be that the ground vibrates mainly with the free vibration or its harmonic which is closest to the forced vibration given by the theory. It is very difficult to explain theoretically these free vibrations, as there are different possibilities not only due to the various harmonics of free vibration which may be observed but also due to the mode of vibration. Some theoretical investigations on free vibrations of the ground under different assumptions have been published by Sezawa and others. (See references on p. 163.) If we have free vibrations of the surface of a pendulum-like type, the relation between the free period and the thickness d of the layer vibrating in this way is very complicated and depends on the vibrating mass; the period is proportional to the square root of d. If we have free vibrations of a pipe-organ type, the observed wave lengths should be $\frac{1}{4}$, $\frac{3}{4}$, $\frac{5}{4}$ and so on of the thickness of the vibrating layer and, consequently, the periods should be proportional to d. In a homogeneous crust no free periods should be observable.

Considering the difficulties just mentioned we will make no attempt to calculate thicknesses of layers which correspond to the observed vibrations or to correlate them with the thickness of the layers which have been found in the region under investigation. Considering the relatively large number of known layers and the possibility of using harmonics of different order, it is not difficult to find such correlations, but there is always a good chance for misinterpretation of such results, the physical meaning of which is at least very doubtful.

According to the theory of Dr. H. Benioff, the strain seismograph should respond to free pendular vibrations of the ground only if the crustal block on which the instrument records is not a part of the system in free vibration. The records of the short-period strain seismograph at Pasadena show in all cases the same periods prevailing as on the other instruments:

TABLE 52.—*Relative frequencies, in percent, of periods under 1.5 seconds recorded on the short-period strain seismograph at Pasadena*

Phase	Periods in seconds											
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1 to 1.2	1.3 to 1.4
P	3	**16**	9	4	**16**	**18**	8	3	1	4	5	3
S	2	8	4	6	**13**	**15**	7	5	4	4	6	6
M	1	**7**	1	1	4	**7**	5	6	9	**17**	10	6

The long-period strain seismograph records show less clearly the periods prevailing than do the records of other long-period instruments. In the P-phase about 20 percent of the recorded periods have values of about 1.5 seconds, while periods of 0.5 second are almost missing. In the S-phase more than one-third of all waves measured on the long-period strain records have periods between 4 and 8 seconds, and in the M-phase the proportion is still higher.

CONCLUSIONS

If an earthquake occurs, waves are produced at the epicenter showing certain "periods." These periods are mostly rather short, only a very small fraction of a second, but long periods occur too, especially in strong shocks. The larger the parts of the earth's crust are, which are affected by the earthquake, the longer the periods which occur together with short-period vibrations. Usually the largest acceleration occurs in waves having periods less than 0.5 second, while the largest displacement is very often observed in waves with periods over 1 second. In rare cases periods as long as 0.5 minute may occur in the epicentral region.

The effect of the structure at the epicenter on the periods seems, in general, to be small.

During their propagation the waves get longer and longer. Apparently to a first approximation the period is proportional to $\sqrt{\frac{D}{V^3}}$, where D is the distance in kilometers from the epicenter and V is the velocity in kilometers per second of the wave under consideration. The periods therefore increase with distance faster in waves with small velocities than in those with high velocities, faster for the maxima than for the S-phase and faster for the S-phase than for the P-phase. These results are not valid for the body waves at distances greater than 2,000 kilometers.

The theory of wave propagation in a homogeneous layer indicates a gradual change in period with distance. The studies show clearly that all observations in southern California are in agreement with observations in the other regions which have been discussed in the first section of this paper: Certain periods prevail and these dominant periods do not increase gradually with increasing distance but in jumps. The most probable explanation for this result is the assumption that the crust is layered and that we observe in most cases free oscillations of the ground. The periods in the observed movements in earthquakes as well as in blasts and microseisms show clearly a predominance of certain values. In southern California at each station there is an individual period between 0.1 and 0.4 second which occurs very much more frequently than others in this range of periods, as may be seen from the following tabulation:

TABLE 53.—*Dominant periods observed at the various stations for local earthquakes, blasts, and microseisms*

Station	Earthquakes less than 100 km away			Blasts less than 100 km away			Microseisms
	P	*S*	*M*	*P*	*S*	*M*	
Pasadena, short	0.2–0.3	0.3	0.2, 0.5–0.6	0.2–0.3	0.3	0.3	0.3
Mount Wilson	0.2	0.2–0.3	0.2–0.3	0.2	0.2–0.3	0.2–0.3, 0.5	0.2
Riverside	0.1	0.1–0.3	0.1–0.2	0.2	0.2–0.3	0.2–0.3	0.3
La Jolla	0.2–0.3	0.3	0.2–0.4	----------	0.2	----------	0.2
Santa Barbara	0.2	0.2, 0.3, 0.5	0.2, 0.6, 0.9–1.0	0.2	0.3	(0.2–0.3)	0.3
Tinemaha	0.2–0.3	0.5	0.5–0.6	----------	----------	----------	0.3
Haiwee	0.2	0.2–0.3	0.3, 0.5	----------	----------	----------	0.3

NOTE.—P = longitudinal wave, S = transverse wave, M = surface wave. In the neighborhood of the epicenter it is very difficult to discriminate between S and M, which usually together are responsible for most of the damage. Both phases consist mainly of vibrations perpendicular to the wave path.

At all our stations periods of 0.5 to 0.6 and of 1.0 to 1.1 seconds are observed very much more frequently than other periods near those values. The periods around 1 second occur infrequently as a rule at distances close to the epicenter, but it is not impossible that in very strong shocks these periods, too, may play an important role at short distances.

The dominant periods of less than 1 second are more outstanding in the vertical than in the horizontal components.

Chapter 10.—PRELIMINARY REPORT ON 4-UNIT PORTABLE SEISMOGRAPH

H. Benioff

The design and construction of a new form of electromagnetic seismograph is being carried on at the Seismological Laboratory of the Carnegie Institution of Washington in cooperation with the California Seismological Program of the United States Coast and Geodetic Survey.

The assembly consists of four portable electromagnetic sesimometers arranged with galvanometers to record simultaneously on a

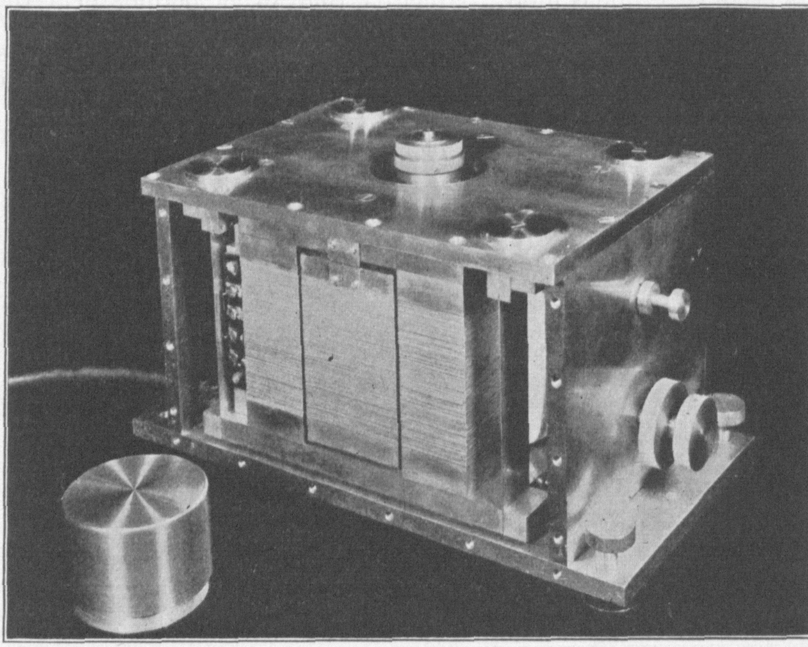


FIGURE 120.—Benioff electromagnetic seismometer. Top cap is removed showing period adjusting screw

single moving film. It is hoped that with such an instrument observations can be made on the simultaneous movements of the various floors and parts of buildings during aftershocks. A further use, not contemplated originally, is the simultaneous recording of ground movements at four points separated by distances up to several kilometers. Such an array would behave like a geophysical seismograph and could be used for the determination of the direction of propagation of seismic waves in earthquakes as well as in microseisms.

The 4-component galvanometer and 2 seismometers have been completed to date. The seismometers are of the general type designed by the writer for routine operation, but are changed considerably in detail. They are completely enclosed in a weather-proof aluminum housing and weigh approximately 50 pounds each (see fig. 120).

The inertia reactor is made up of the moving elements of the transducer only. It is suspended by four heavy phosphor-bronze ribbons. The period is normally adjusted to 1.5 seconds but can be varied over a range of approximately 0.5 second by means of an auxiliary tension member. The damping force is derived from the reaction of the output currents and is adjusted to a value $h=0.707$. The galvanometer assembly is shown in figure 121. It is made up of four independent

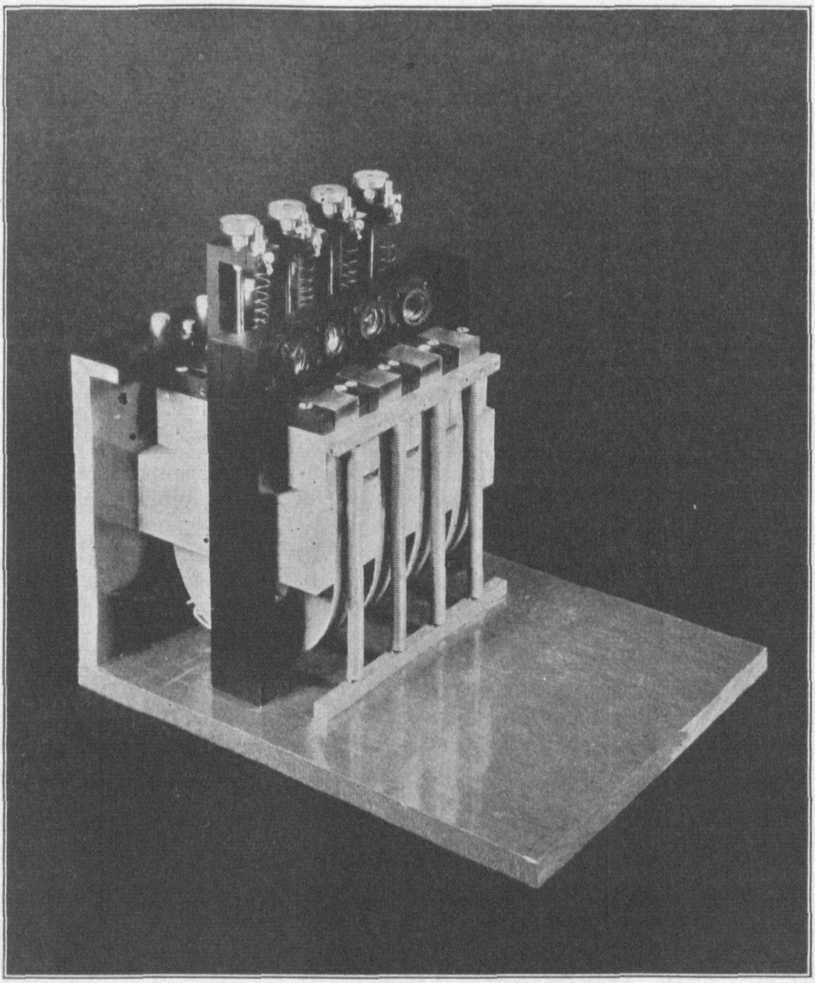


FIGURE 121.—Four-unit galvanometer assembly.

units. The period of each element is 1.5 seconds and the damping constant is $h=20$. The galvanometers thus behave as fluxmeters so that over the period range $1/10$ to 4 seconds the response of the seismograph is substantially that of the pendulum units. For the projected problem, therefore, the assembly provides the characteristics of simple pendulum seismographs with the flexibility and advantages of electromagnetic instruments. The magnification is adjustable over a range from 30,000 to 100 approximately.

Chapter 11.—GEODETIC WORK IN EARTHQUAKE REGIONS IN CALIFORNIA

W. F. REYNOLDS AND H. S. RAPPLEYE

TRIANGULATION

Three reports have been issued by the Coast and Geodetic Survey on the testing by triangulation of earth movements in California. The first one was entitled "The Earth Movements in the California Earthquake of 1906", and appeared as Appendix 3 of 1907 just after the earthquake of 1906. This publication considered in detail the amount and nature of the displacement of portions of the earth's crust due to this earthquake and also to earlier movements.

The old triangulation fixing the positions of the points before the earthquake of April 18, 1906, was done in many years, extending from 1851 to 1899, as a part of the regular work of the Coast and Geodetic Survey. The triangulation done during the interval July 12, 1906, to July 2, 1907, extends continuously from Mount Toro in Monterey County and Santa Ana Mountain in San Beñito County to Ross Mountain and the vicinity of Fort Ross in Sonoma County. It extends over an area 270 kilometers (168 miles) long and 80 kilometers (50 miles) wide, at its widest part.

The second report entitled "Earth Movements in California", Special Publication No. 106, was printed in 1924. Since the conclusions arrived at in this publication were based on insufficient evidence it was superseded by the third report.

The third report entitled "Comparison of Old and New Triangulation in California", Special Publication No. 151, was printed in 1928. In this publication a comparison was made of the positions of triangulation stations determined from the observations made previous to 1900 with the positions determined from the observations resulting from the reoccupation of the stations during the interval 1922 to 1925. The work discussed involves 64 stations.

In 1925, 1926, 1927, and 1928 no triangulation was executed in earthquake regions. In 1929 an arc extending from Newport Beach to the thirty-fifth parallel and consisting of 20 first-order and 86 second-order stations was executed. In 1930 two arcs were executed, the Point Reyes-Napa arc consisting of 9 first- and 36 second-order stations and the Monterey Bay-Mariposa Peak arc consisting of 10 first- and 24 second-order stations. In 1931 no triangulation was executed in earthquake regions. In 1932 three arcs were executed: 1, San Luis Obispo northeastward arc consisting of 18 first- and 57 second-order stations; 2, San Fernando-Bakersfield arc consisting of 16 first- and 76 second-order stations (see fig. 123); 3, vicinity of Taft arc consisting of 7 first- and 20 second-order stations.

During the year 1933 one arc of triangulation was completed for the investigation of earth movement in California and involved 8 first- and 57 second-order stations. This work was a reoccupation of the stations of an arc originally executed in 1928–29 from Newport

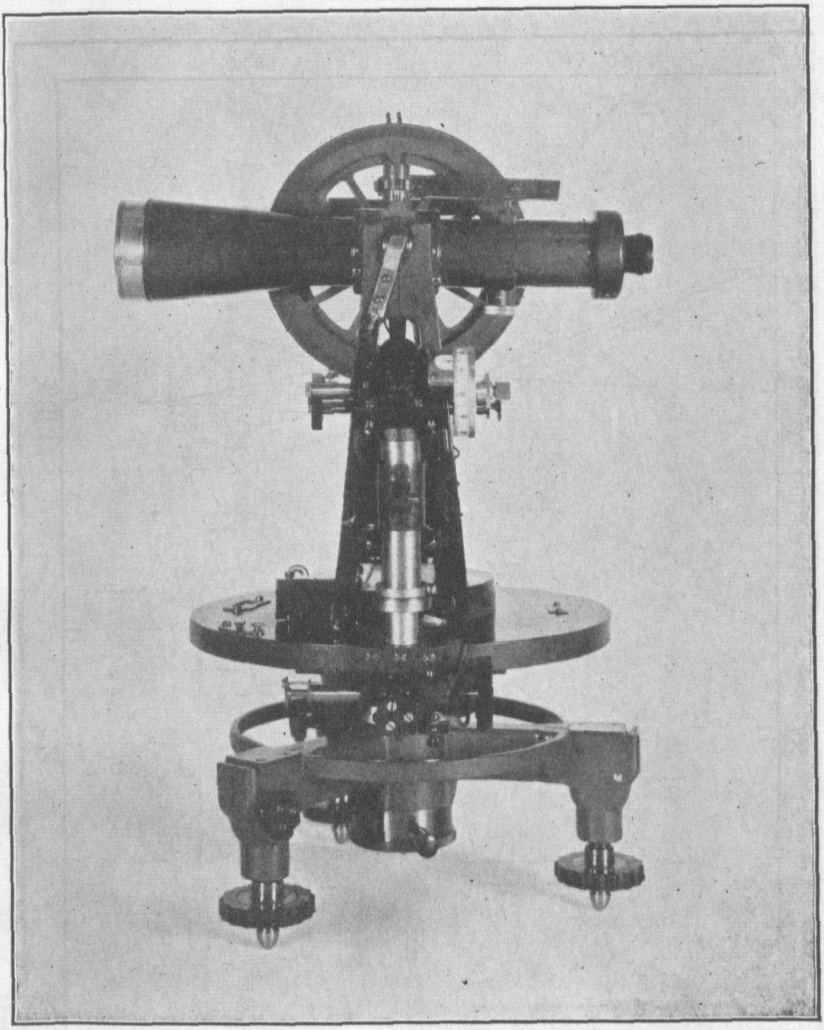


FIGURE 122.—Parkhurst 9-inch theodolite, used in first-order triangulation.

Beach to Riverside, Calif. The differences between the original observed angles and those observed in 1933 were too small to indicate that any movement of the stations had occurred. No triangulation was executed in earthquake regions in 1934.

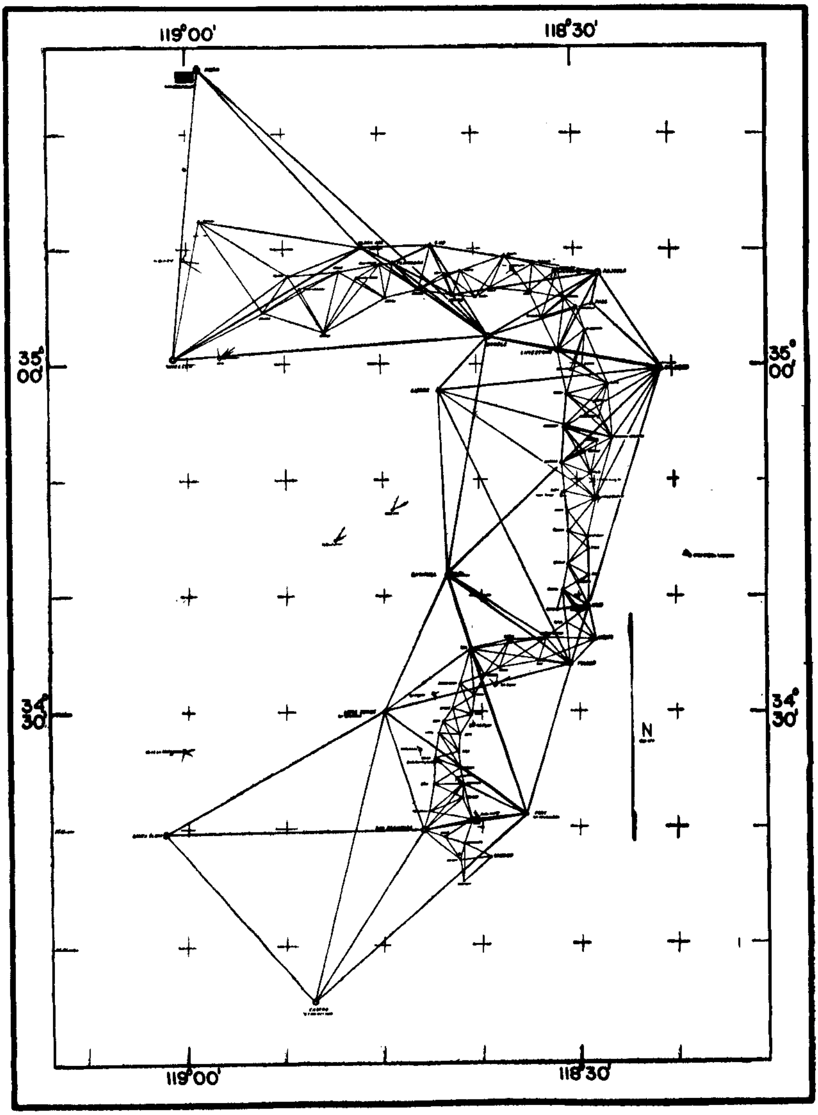


FIGURE 123.—Triangulation, San Fernando to Bakersfield, 1932. The large first-order scheme is used to control the detailed second-order work, which in turn serves as a basis for detecting earth movements.

LEVELING

The following lines of levels were run during the calendar year 1933 for the purpose of investigating earthquakes or detecting earth movements:

1. Dumbarton Bridge, via Palo Alto, to Skyline Boulevard, Calif.
2. Santa Ana to San Diego and Fall Brook, Calif. (Releveled.)
3. San Jose to Santa Margarita, Calif. (Releveled.)
4. San Francisco to Niles to Oakland, Calif. (Releveled.)
5. Mina to Battle Mountain, Nev.
6. Cairo-Hoxie area (Ark., Ky., and Tenn.).
7. Harbor City to Redondo Beach, Calif. (In progress at the end of the year.)
8. Long Beach area, Calif. (In progress at the end of the year.) (Releveled.)

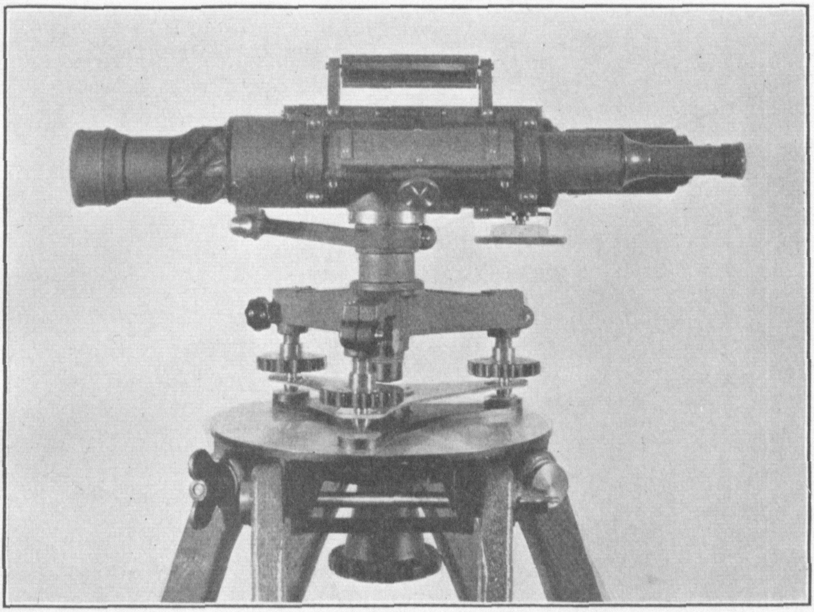


FIGURE 124.—Latest type of level used in first-order leveling.

The lines from San Jose to Santa Margarita, Calif., and from San Francisco to Niles to Oakland, Calif., were rerun because of abnormal settlement at San Jose, which may not be attributable to earthquakes.

During the year 1934, the following lines of levels were run for the purpose of earthquake investigation or detection of earth movements:

1. Releveling, Long Beach area, Calif. (In progress at the beginning of the year.)
2. Harbor City to Redondo Beach, Calif. (In progress at the beginning of the year.)
3. Playa del Rey to Los Angeles, Calif.
4. Azusa to Coldbrook Camp, Calif.
5. Oakland to Martinez, Calif.
6. Vicinity of Goleta, Calif.
7. Settlement investigation, vicinity of San Jose, Calif., spring, 1934.
8. Settlement investigation, vicinity of San Jose, Calif., fall, 1934.
9. Redlands to Victorville, Calif.
10. Releveling, vicinity of Kosmo, Utah.

In addition to the arcs of triangulation and lines of leveling considered above, other triangulation and leveling have been established in the State of California which will be of value in future investigations of earth movements.

CUT ON THIS LINE

PUBLICATION NOTICES

To make immediately available the results of its various activities to those interested, the Coast and Geodetic Survey maintains mailing lists of persons and firms desiring to receive notice of the issuance of charts, Coast Pilots, maps, and other publications.

Should you desire to receive such notices, you may use the form given below, checking the lists covering the subjects in which you are interested.

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DEAR SIR: I desire that my name be placed in the mailing lists indicated by check below, to receive notification of the issuance of publications referring to the subjects indicated:

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