1950

The Design of Reinforced Concrete Buildings to Resist Earthquake Forces

John A. Bonell

THE DESIGN

OF

REINFORCED CONCRETE BUILDINGS

TO

RESIST EARTHQUAKE FORCES

by

John A. Bonell, Jr.

Assistant Prof. of Civil Engineering

University of Nevada

Reno, Nevada

In Partial Fulfillment of the Requirements for the Professional

Degree of Civil Engineer

at

South Dakota State College

Brookings, South Dakota

April, 1950

The thesis of John Bonell, Jr., for the degree of

CIVIL ENGINEER

is hereby approved.

Emory E. Johnson, Head
Department of Civil Engineering

H. M. Crothers, Dean
Division of Engineering

Table of Contents

Part I.

EARTHQUAKE FORCES

There does not exist today, and perhaps there never will be, any known method of predicting the location and time at which an earthquake will occur. Neither can we predict the maximum intensity that earthquakes in any given locality will bring. Hence the practicing structural engineer is constantly faced with the problem of deciding how much, if any, seismic force to include when designing any particular structure, and from the economic viewpoint, how often will this force occur. At present, he will, of course be guided by the particular building code of the area in which he is working but these codes, because of the lack of any very conclusive information based on research or experience, are likely to skip over the subject very lightly or to omit it altogether.

Some areas are known to be subject to frequent earthquakes of varying intensity and others only infrequently if at all. Due to lack of adequate measuring instruments the intensities of quakes prior to 1933 can be only roughly computed based on the extent of known damage. The intensity of the earthquake at Long Beach, Calif. in March, 1933 was measured by instruments and these records constitute the first adequate measurement of the motion of the earth during a major earthquake in the U. S. Under the impetus of this disastrous earthquake much additional study and research has been undertaken and much has been learned since that time. However, present knowledge is still inadequate, and a great deal more must be learned before analysis of structures for seismic forces will become very precise.

The U. S. Coast and Geodetic Survey published a Seismic Probability Map of the U. S. (See Figure 1) in 1948 showing the areas in which earthquakes of varying proportions might be expected to occur. This map is based on records of past seismic activity and because of the sketchiness of these records prob-


U. S. COAST AND GEODETIC SURVEY
SEISMIC PROBABILITY MAP OF THE UNITED STATES
Compiled in January 1948 by the U. S. Coast and Geodetic Survey with the advice or assistance of J. P. Buwalda, Perry Byerly, B. Gutenberg, Andrew Lawson, L. D. Leet, D. J. Linehan, S. J., J. B. Macelwane, S. J., C. F. Richter, V. C. Stechschulte, S. J., and H. O. Wood.
Zone 0 - no damage
Zone 1 - minor damage
Zone 2 - moderate damage
Zone 3 - major damage
M-2536-4

ably no two persons would be able to agree on the same map. It indicates zones of probability only and does not infer that an earthquake of greater intensity than indicated might not occur in any zone. In general, zone 0 indicates that area which has had some seismic activity of intensity not greater than IV (referring to the Modified Mercalli Scale, See Appendix I). Zone 1 is the area where the intensity has been V or VI. Zone 2 is that area where the intensity of VII or VIII has been frequently measured and an intensity of IX has been occasionally measured. Zone 3 is that area where major destructive earthquakes of higher intensity have occurred. Seismic forces should be considered in the design of structures located in Zones 1, 2, and 3.

Earthquakes are believed to be caused by the movement of immense blocks of the earths crust relative to each other along irregular planes called faults. The movement may be horizontal, vertical, or a combination of both and varies greatly in magnitude. This movement causes horizontal and vertical vibrations of the earths surface, the horizontal usually being the most destructive and varying from 5 to 10 times the magnitude of the vertical vibrations. The point of the earths surface directly over the center of movement is called the epicenter. With modern methods it has been possible to calculate the faults on which movement has taken place and the location of the epicenters of the quakes. The location of many active faults is known to geologists.

It is not the purpose of this paper to discuss the subject of seismology at any great length. It is important, however, for an engineer to have some knowledge of the instruments used in the measurement of seismic forces and the interpretation of the records secured therefrom. The discussion of these instruments which follows is intended to illustrate their use in securing data which is valuable to the structural engineer.

A program for the study of ground movements by the use of strong-motion instruments was initiated by the Coast and Geodetic Survey in 1925 and re-

ceived much impetus after the Long Beach, California earthquake of March 10, 1933. "The main purpose of the 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 and displacement meters. Each consists of the instruments which respond directly to the earthquake, a starting device, the automatic recorder, the time-marking clock, the light and optical system, electric circuits, and batteries." (Quoted from "Earthquake Investigations in California, 1934-35, published by U. S. Department of Commerce.)

The primary purpose of the instrumental equipment is to measure the motion of the ground or structure. A seismograph is an automatic, continuously operating and recording instrument based on modifications of the principle of the pendulum. A complete installation consists of two seismometers set at right

angles to each other to measure horizontal motion, and a vertical seismometer to measure vertical motion. A stylus bears against a paper disk or cylinder mounted on a clock operated turntable or drum and produces the seismogram. The free period of vibration of the seismograph is larger than that of earthquakes.

When the support of the seismograph is forcibly set in motion by some force such as an earthquake the pendulum is set in motion and records the movement of the earth, but because the seismograph pendulum has a natural period different from that of the earth this record will not accurately represent the true movement or vibration of the earth. Obviously then, a large number of instruments of varying degree of sensitivity would be required to record the large range of ground motions. This is impractical. A system of magnifying levers has been used which greatly increases the instruments usefulness with respect to the number of shocks it is capable of recording satisfactorily.

The analysis of seismograms is based upon the assumption of simple harmonic motion and the Fourrier statement that any irregular curve may be broken up into a series of simple harmonic vibrations. The engineer is interested in the acceleration and displacement as measures of the forces in action and he is also interested in the period of vibration insofar as it may be synchronous with the structure he is concerned with. As long as the seismogram records periodic action similar to simple harmonic motion, the period may be read simply by inspection since the pendulum is forced to oscillate in the period of impressed vibration. The range of visible periods, however, will depend largely on the period of the pendulum. Short period pendulums accentuate short-period waves; long period pendulums accentuate long-period waves.

The differences between recorded and impressed accelerations and displacements or amplitudes may be very great because of the difference between periods of the ground and the pendulum. The well known formula for acceleration due to simple harmonic motion is:

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

in which a is acceleration

A is amplitude of displacement of ground

T_e is the period of the earth.

It can be shown that when the earth wave period is more than 3 or 4 times the pendulum period the acceleration can be computed from the seismogram as follows:

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

in which T_o is the period of seismograph pendulum

A_t is the trace amplitude

V is the lever magnification of the pendulum.

From this equation it is seen that the acceleration depends only on the trace amplitude and the instrument functions as an accelerometer and is called an <u>accelerograph</u>.

When the earth periods are less than about one-third the pendulum periods the seismograph functions as a <u>displacement meter</u>. Because of the lever magnification neither accelerographs nor displacement meters are operated continuously but are equipped with special starting devices which operate when jolted.

When the impressed periods are close to the pendulum period the seismograph records neither displacement nor acceleration directly. This represents the condition existing between the useful periods of the accelerograph and the displacement meter. Under this condition and when simple harmonic motion is not visible on records of accelerographs and displacement meters, the displacement may be computed by integration. This is based on the fundamental equation of motion of a damped pendulum when subjected to an external acceleration:

$$\frac{d^2x}{dt^2} = \frac{d^2y}{dt^2} - 2k\frac{dy}{dt} - p^2 y$$

in which x is the instantaneous displacement of the ground or support, y is the instantaneous displacement of the pendulum relative to the instantaneous position of the ground (measured directly from the record), k is a damping constant, and p^2 is an instrument period factor. t is the time. The integration of this expression is a very laborious computation.

From the records and analysis of seismograms as outlined above it is seen that the acceleration, displacement, and period of an earthquake can be determined by measurement and calculation. It would seem then that the forces acting on a structure could be determined by the application of Newtons second law of motion, $F = M a$, using values of acceleration which seem typical of past earthquakes in the region. Accelerations during the Long Beach earthquake of 1933 were measured between 0.1 and 0.23 gravity; records for other quakes in California have shown accelerations from 0 to 0.3 gravity. Indeed many of the present-day building codes use this formula with requirements of design acceleration from 0.02 to 0.2 gravity. The use of this formula neglects the effect of foundation material and the type of the structure itself which may either dampen or accelerate the movements of the ground.

In the Long Beach earthquake of 1933 it was found that masonry buildings located on well consolidated alluvial fill with a low water table suffered more damage than those located on more recently deposited alluvium with a water table from 2 to 10 feet below the surface. Experimental work on determining the periods of the ground has been done in Japan, Germany, and in Southern California. It has been found that earthquakes have induced periods in the ground at different localities not necessarily the same as those of the earthquake itself. In the experiments conducted in Southern California, for example, it was found that there was a predominance of periods of 0.5 to 0.6 seconds and 1.0 to 1.1 seconds as long as the earthquake was over 50 kilometers away. For local quakes the periods were different. Structures should not be built having periods the same as the prevailing periods of the ground on which they are to be built without considering the effect of such resonance as might result. The experiments on periods of the soil conducted to date are far from conclusive as far as application to any locality other than the one in which the experiments were conducted is concerned.

Using seismographs similar to those described above the U. S. Coast and

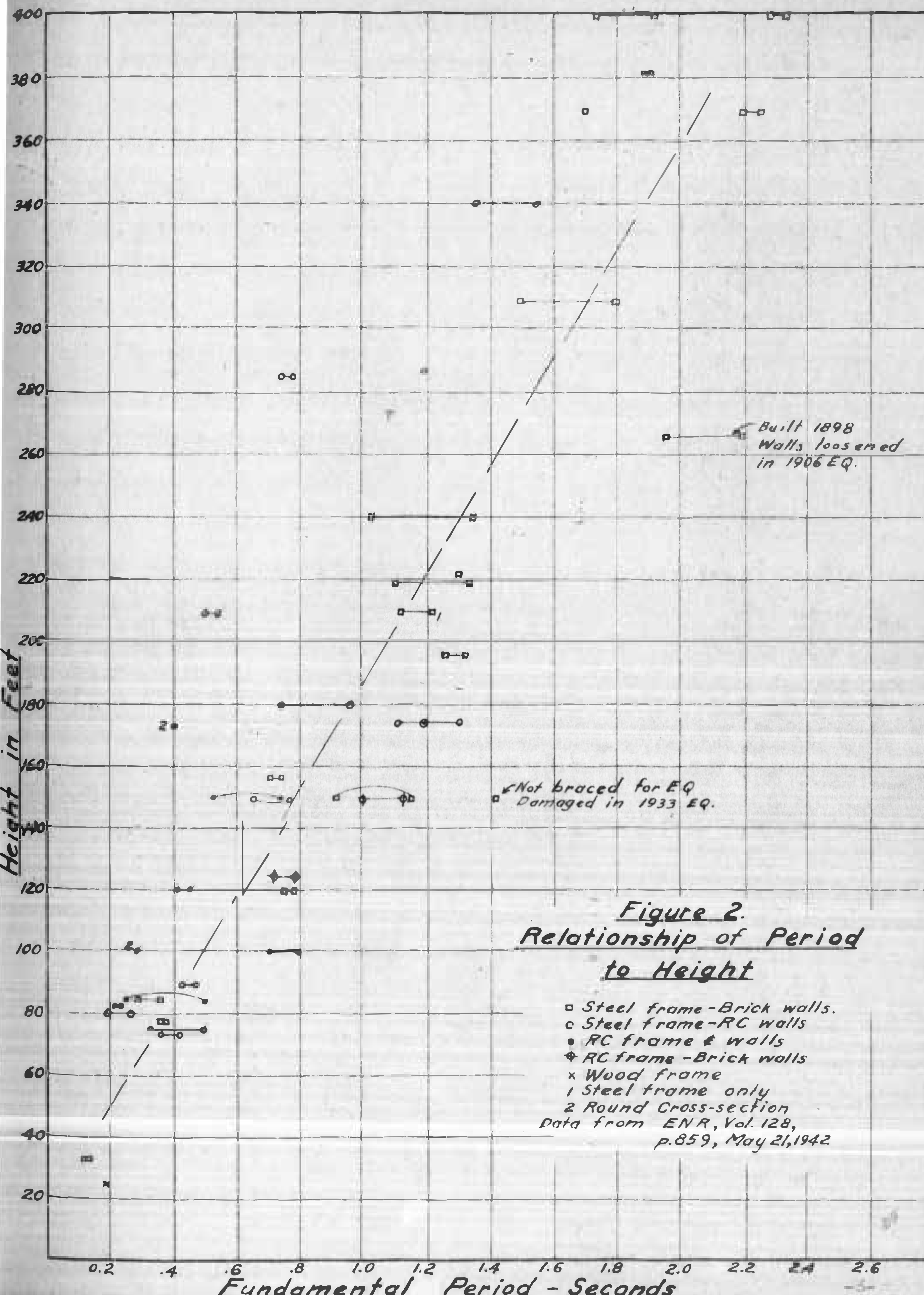


Figure 2.
Relationship of Period to Height

Geodetic Survey has conducted experiments on the vebration of buildings to determine their fundamental periods. In most of the structures measurements were taken with moderate or light winds blowing and the amplitudes were consequently small. In some cases vibrations were induced using a vibrating machine having eccentric revolving wheels of considerable mass. By varying the speed of rotation, points of resonance in the structure could be studied. It should be noted that in none of these studies was the amplitude of vibration large enough to break the bond between any of the principle parts of the structure. Should such a break occur the period of the structure would be vastly different than that noted and would probably approach that of the building frame itself. Those structures having the shorter periods are the more rigid but it should be remembered that if some part of the structure becomes inactive through failure the period may increase considerably.

Some of the results of the vibration tests of the U. S. Coast and Geodetic Survey were published by Mr. H. M. Engle in Engineering News Record, May 21, 1942. This data is reproduced graphically here as Figure 2. Data included are for all types of buildings as indicated and the graph shows a very rough linear relation between the period and the height. It also shows that those structures designed for a lateral force of approximately 0.1 gravity have a shorter period than the structures not so designed. (The structures designed for the gravity force have their periods marked in red.) It should be noted that variations in materials of construction and the stiffness of partitions effect the stiffness of the building and it is difficult to calculate their exact effect.

Results of some of the building vibration tests made by the Coast and Geodetic Survey are presented here in Table I. The structures selected for this Table are principally reinforced concrete (RC) or reinforced concrete walls and steel frame but a few examples are included of steel frame with brick walls and reinforced concrete frame with brick walls for comparative purposes. Examples were selected about which the available information was the greatest. The values

TABLE I. COMPARATIVE VIBRATION DATA ON REINFORCED CONCRETE BUILDINGS

Bk.	Brick walls are curtain walls unless otherwise noted.
Comb.	Combined
Conn	Connected horizontally (interconnected)
F	Facing
Fl	Floor
L	Wood lath
Mez	Mezzanine floor
ML	Metal lath
P	Plaster
par	Partition
*	Known to have been designed for a lateral force.
RC	Reinforced concrete
St	Structural steel
S	Stone
s	Street sides. Refers to facing on walls.
T	Tile
TC	Terra Cotta
W	Wood
X	Position of vibrator in building
◯	No. of stories--in column for "shape"

References:
Earthquake Investigations in California, 1934-5
U. S. Coast & Geodetic Survey
Engineering News Record, Vol. 128, p. 859, May 21, 1942.

No.	Building & City	Soil	Foundation Data	Frame	Walls	Floors	Ht. in Ft.	Shape (N, W, E, S)	Period N-S	Period E-W	Remarks
1.	Hollywood Strge. Hollywood, Calif.	Soft alluvium	RC Comb & col. footing on piles.	RC	RC	RC	141	217' × 51'	1.20	0.49	A few T par on first Fl only
2.	Medico-Dental Los Angeles Calif		RC Footing	RC	Bk	RC	156	100' × 60' (13)	1.22	1.06	T pars
3.	So. Calif. Gas Co., Los Angeles		RC Comb	RC	RC	RC	85	84' × 50' (7)	0.51	0.27	A few T par
4.	Architects Los Angeles		RC Footing	RC	RC	RC	150	157' × 57' (12)	0.75	0.53	T par. Negligible damage in Long Beach, 1933 earthquake.
5.	Associated Realty, Los Angeles		RC Footing	RC	Bk	RC	150	201' × 105' (12)	1.14	1.00	T par

TABLE I. COMPARATIVE VIBRATION DATA ON REINFORCED CONCRETE BUILDINGS--Continued.

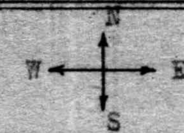
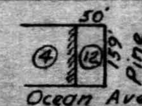
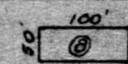
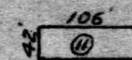
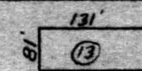
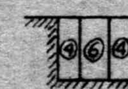
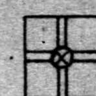
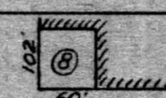
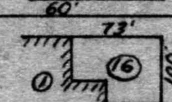
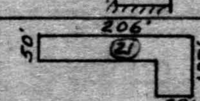
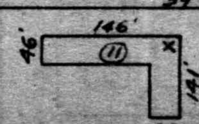
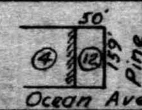
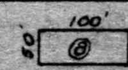
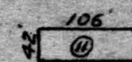
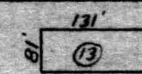
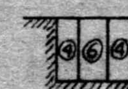
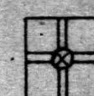
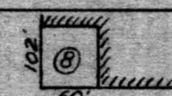
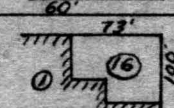
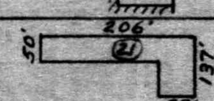
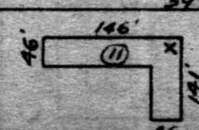
No.	Building & City	Soil	Foundation Data	Frame	Walls	Floors	Ht. in Ft.	Shape 	Period		Remarks
									N-S	E-W	
16	Heartwell Long Beach, Calif	Soft Alluvium	RC Footing 7700 lb/sq.ft.	RC	Bk	RC	130		0.76	1.10	T par. Not repaired since 1933 quake.
17	Medical Los Angeles		RC Footing	RC	RC	RC	90		0.50	0.41	S & P par
18	Medico-Dental San Jose	Blue clay	RC Conn Ftg. on piles	St	RC	RC	138		0.92	0.64	ML & P par
19	State Los Angeles	Gravel	RC Footing	St	RC Bk F	RC	162		0.91	0.63	T par
* 20				St	RC S F		90		0.43	0.48	Braced for .08 gravity
21	Court House	Sand & clay	RC interconnected footings	St	RC	RC	210		Complete 0.55 St. frame 1.22	0.51 1.13	Designed for 0.2 gravity
22		Tidal Flat	Piles	St	RC		75		0.33	0.50	Braced for 0.1 gravity. 5 RC par Undamaged in 1933 EQ
* 23	Oceanic San Francisco	Fill overlying marine clay	RC Footings on piles	RC	RC	RC	100		0.70	0.80	Quality of design & materials questionable. T, ML & P par
24	Financial Center Oakland, Calif	Sandy clay	RC Footings 8000 lb/sq.ft.	St	Bk & RC	RC	219		1.13	1.34	T par
25	Standard Oil Co. San Francisco	Sand & sandy clay	RC Mat. Piles under end walls 4000 lb/sq.ft.	St	Bk	RC	309		1.80	1.60	T par
26	Willmore Apts. Long Beach, Calif.	Soft alluvium	RC Footing	RC	RC	RC	132		0.62	0.62	T par. After repairs, 1933 EQ

TABLE I. COMPARATIVE VIBRATION DATA ON REINFORCED CONCRETE BUILDINGS--Continued.

No.	Building & City	Soil	Foundation Data	Frame	Walls	Floors	Ht. in Ft.	Shape N W E S	Period N-S	Period E-W	Remarks
16	Heartwell Long Beach, Calif	Soft Alluvium	RC Footing 7700 lb/sq.ft.	RC	Bk	RC	130		0.76	1.10	T par. Not repaired since 1933 quake.
17	Medical Los Angeles		RC Footing	RC	RC	RC	90		0.50	0.41	S & P par
18	Medico-Dental San Jose	Blue clay	RC Conn Ftg. on piles	St	RC	RC	138		0.92	0.64	ML & P par
19	State Los Angeles	Gravel	RC Footing	St	RC Bk F	RC	162		0.91	0.63	T par
* 20				St	RC S F		90		0.43	0.48	Braced for .08 gravity
21	Court House	Sand & clay	RC interconnected footings	St	RC	RC	210		Complete 0.55 St. frame 1.22	 0.51 1.13	Designed for 0.2 gravity
22		Tidal Flat	Piles	St	RC		75		0.33	0.50	Braced for 0.1 gravity. 5 RC par Undamaged in 1933 EQ
* 23	Oceanic San Francisco	Fill overlying marine clay	RC Footings on piles	RC	RC	RC	100		0.70	0.80	Quality of design & materials questionable. T, ML & P par
24	Financial Center Oakland, Calif	Sandy clay	RC Footings 8000 lb/sq.ft.	St	Bk & RC	RC	219		1.13	1.34	T par
25	Standard Oil Co. San Francisco	Sand & sandy clay	RC Mat. Piles under end walls 4000 lb/sq.ft.	St	Bk	RC	309		1.80	1.50	T par
26	Willmore Apts. Long Beach, Calif.	Soft alluvium	RC Footing	RC	RC	RC	132		0.62	0.62	T par. After repairs, 1933 EQ

98979

TABLE I. COMPARATIVE VIBRATION DATA ON REINFORCED CONCRETE BUILDINGS--Continued.

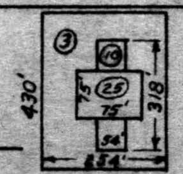
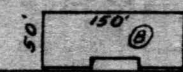
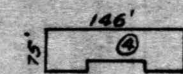
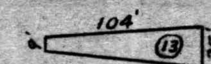
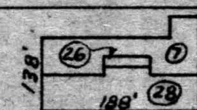
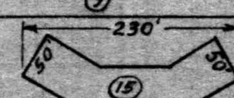
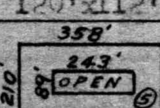
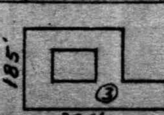
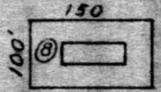
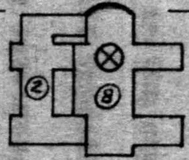
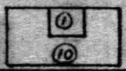
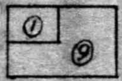
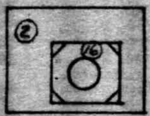
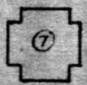
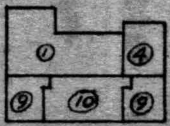
No.	Building & City	Soil	Foundation Data	Frame	Walls	Floors	Ht.	Shape (N, S, E, W)	Period N-S	Period E-W	Remarks
27	City Hall Los Angeles, Cal.	Blue shale	RC Footing mat under 25 floor portion	St	Bk T F	RC	400		2.10	2.25	T par
28	Kress Long Beach, Calif	Adobe over sand	RC Footing	RC	RC	RC	95		0.58	0.44	T par
28A	School Administration, Long Beach, Calif.	Soft alluvium	RC Footing, some comb. 5000 lb/sq.ft.	RC	RC	RC	48		0.37	0.31	T par. Damaged by 1933 quake
29	Federal Realty Oakland, Calif.	Sand & clay	RC Comb Footing	St	RC TC F	RC	177		1.70	1.06	ML & P par
30	450 Sutter San Francisco	Sand & clay	RC footing 8000 lb/sq.ft.	St	RC TC F	RC	340		1.36	1.55	T par
31	Horace Mann Pasadena J. C. Pasadena, Calif.	Gravel	RC Footing	RC	Bk	RC		Irregular (3)	with Bk. Walls 0.22 walls down 0.46	0.23 0.46	Being repaired
32	Municipal Auditorium, Long Beach	Fill	RC Piles	St & RC	RC	RC	105	Irregular 277'x172' (9)	0.33	0.33	T & RC par Large open spaces, balconies, etc.
33	Villa Riviera Long Beach, Calif	Soft alluvium	RC Footing 5500 lb/sq.ft	St	RC	RC	184		1.25	0.33	T par. Heavy surf running during test.
34	Womens City Club Berkeley, Calif	Alluvium	RC Footing	RC	RC	RC	100	Irregular 120'x112' (6)	0.23	0.23	ML & P par Attached wings
35	Federal Office San Francisco	Sand	RC Footing 6000 lb/sq.ft	St	Bk & S	RC	85		0.37	0.33	Frame braced for 15 lb wind. Heavy walls.
36	Public Library San Francisco	Sand	RC Conn Footing	St	Bk S F	RC	78		0.38	0.36	T par. High floors. Frame not braced.

TABLE I. COMPARATIVE VIBRATION DATA ON REINFORCED CONCRETE BUILDINGS--Continued

No.	Building & City	Soil	Foundation Data	Frame	Walls	Floors	Ht. in Ft.	Shape (N ↑, S ↓, W ←, E →)	Period N-S	Period E-W	Remarks
37	Sheldon San Francisco	Fill over marine clay	RC Footing on piles	RC	RC	RC	99		0.70	0.55	ML & F par. Adjacent bldgs. not known
*38	Dormitory	Sand & gravel		RC	RC	RC	82		0.25	0.23	Massive structure Wings do not complicate vibrations. Moderate wall opns.
39	Office Building	Alluvial		RC	RC & Br	RC	180		0.97	0.74	Framed for wind only
40				RC	Bk	RC	125		0.70	0.78	Not designed for wind or EQ. Continuous frame gives stiffness.
* 42	Library	Sand & gravel	Solid mat under tower	St	RC	RC	285		0.74	0.77	Steel frame braced for 20 lb wind only. Bldg as a whole braced for 0.1 grav.
43	Hospital	Clay & Rock		RC	RC	RC	85		0.26	0.55	Wall openings moderate. Rigid
* 44	Federal	Rock		St	RC S F	RC	80		0.27	0.20	Braced for 0.1 g. Rigid
*45	Tank Tower	Hardpan		RC	RC		76 (Fl) 100 to top of walls		0.29	0.29	Braced for 0.1 g. Period about same for tank full or empty. Solid walls to ground
*46	Hotel	Deep alluvium.	RC interconn. footings on RC piles	RC	RC	RC	120		0.45	0.41	Braced to resist shock.

of fundamental periods taken from Table I have been plotted against height in Figure 3 and against the ratio of height to depth in the direction of translation in Figure 4.

The period of a structure is a function of the mass, height, depth, and rigidity of the structure. In the analysis of any structure the mass can be approximately computed and the height and depth in direction of translation can be determined with relative precision. The rigidity, however, depends upon the shape, materials of construction, workmanship, and foundation and the calculation of this rigidity in the case of most relatively simple structures results in a very complex if not impossible mathematical solution. The effect of partitions and walls, for instance, exerts a varying influence on the rigidity depending on their location and type of construction.

An examination of Figures 3 and 4 will, however, show that the reinforced concrete structures and those structures designed for a lateral force have shorter periods than structures built of other materials. This is not to be taken as a criterion indicating that buildings having short periods are always earthquake resistant. Before a small period c an be assumed to indicate a high degree of earthquake resistance, it must be correlated with the design and quality of construction. Buildings 23, 33, and 40 have periods in the class of the earthquake resistant buildings but experience with this type of construction in past earthquakes has shown them to be highly susceptible to damage.

Generally a period of 0.7 second or less indicates structural strength and rigidity and a period of 1.0 or larger is indicative of lack of bracing and lateral rigidity. Whether the larger periods also indicate a flexibility capable of withstanding the earthquake shock is a matter which cannot be decided until the argument between the advocates of rigid construction and those of flexible construction if finally resolved. It may be observed in Figures 3 and 4 that there is a tendency toward a straight line relationship between the fundamental period and height or height/depth ratio but the variance of the points

from the straight line is too great for a conclusive result. Using the data shown in Table I the writer has attempted to develop a curve, or relationship, giving closer correlation of all points than those of Figures 3 and 4.

The first study was made on the relationship of the period per story to the height in stories, the height in feet and the height/depth ratio. One reference in the literature stated that the period of structures varied from one-tenth to one-twentieth of a second per story. The writer found the periods of the structures shown in Table I to range from 0.03 to 0.13 second per story, a range too great to be of any practical value. Furthermore the range of periods within each building classification was almost as great. The results were as follows:

Building Classification	No. Studied	Range of Periods per Story	Average Period per Story
RC. frame and walls	30	0.030-0.103	0.0639
St. frame and RC. walls	27	0.039-0.130	0.0737
RC. frame and Brick walls	8	0.063-0.095	0.0840
St. frame and Brick walls	6	0.066-0.130	0.0940

It should be noted that the more rigid structures have the lower average periods per story. The three relationships mentioned at the beginning of this paragraph were entirely inconclusive and their graphs are not reproduced here.

Other studies were made to show the correlation between the measured periods of the structures (Table I) and various ratios of height and depth as suggested by formulas devised by others as relatively simple means of obtaining the periods of structures mathematically. The formulas used were:

(a) For tall buildings $T = c\sqrt{\dfrac{w\,H^4}{g\,EI}}$ (Creskoff)

(b) For short buildings $T = \sqrt{\dfrac{c^2 w\,H^4}{g\,EI}\left(1+\dfrac{d^2}{4c_1H^2}\right)}$ (Creskoff)

(c) For all buildings $T = 2\pi\sqrt{\dfrac{w}{g}\dfrac{H^3}{3EI}}$

In these formulas T is the fundamental period of the building, w is the weight per unit of height, H is the height, d is the depth in direction of translation, E is the modulus of elasticity, I is the moment of inertia, g is the acceleration

due to gravity, c_1 is a coefficient depending on end conditions (0.13 for fixed base and 0.55 for hinged base) and c is a coefficient also depending on end conditions. Considering only the exterior walls as contributing the major portion of the moment of inertia these formulas may be reduced to:

(a) $T = N_1 \frac{H^2}{d^{3/2}}$ or $T = N_2 \frac{H^2}{d}$

(b) $T = N_1 \frac{H}{d^{3/2}} \sqrt{H^2 + 2d^2}$ for fixed base.

$T = N_1 \frac{H}{d^{3/2}} \sqrt{H^2 + 0.5d^2}$ for hinged base.

(c) $T = N_3 \sqrt{\left(\frac{H}{d}\right)^3 \frac{L}{1+3k}}$ in which k is the ratio of width of building to depth. The values of N are dependent upon physical constants.

Figures 5 and 6 show the values of the fundamental periods plotted against equations (a). Although the points are scattered these curves are perhaps the most satisfactory studied. Since this paper was started, a committee of engineers from San Francisco has recommended that seismic coefficients be computed from the curve shown in Figure 6. In view of the wide spread of points on both sides of the curve it is the writers opinion that the adoption of the curve in building codes as standard would, in some cases, be a dangerous procedure.

The results from equations (b) are shown in Figures 7 and 8 and the results from equation (c) are shown in Figure 9. Equations (b) and (c) are more cumbersome than equation (a) and the results are no more accurate.

It was concluded from these studies that there is probably no simple relationship for determining the period of a structure based upon its physical properties alone. This might well have been expected since the formulas do not take into account the damping or accelerating influence of the soils upon which the building stands. It would appear then that more information about the action of soils is necessary before the periods of individual structures in different localities may be predicted with any degree of accuracy. A study of the curves herein presented and the data on which they were based may, however, be of some assistance in estimating the periods of specific structures.

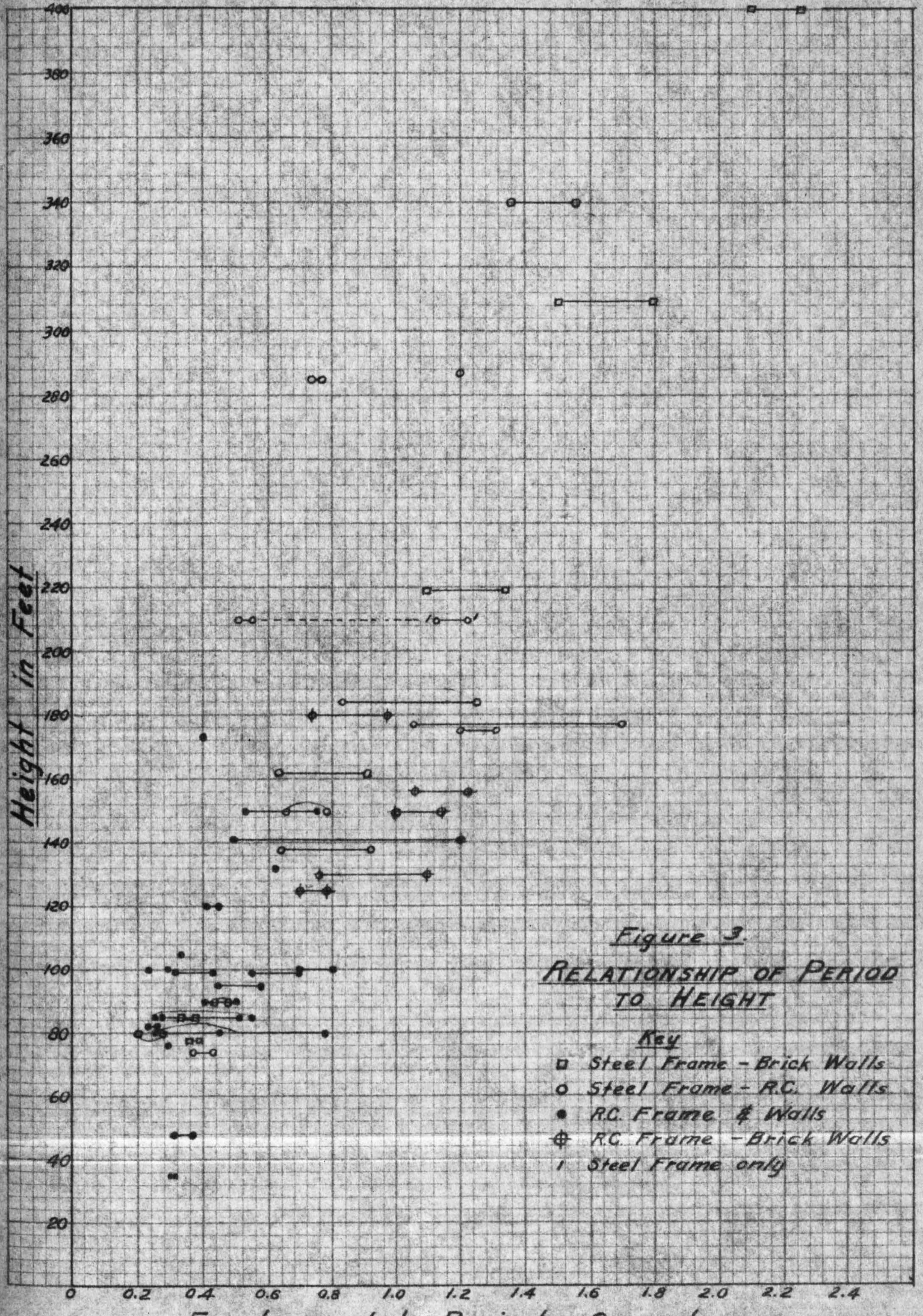


Figure 3.
RELATIONSHIP OF PERIOD TO HEIGHT
Key
Steel Frame - Brick Walls
Steel Frame - R.C. Walls
R.C. Frame & Walls
R.C. Frame - Brick Walls
Steel Frame only
Height in Feet
Fundamental Period - Seconds
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400

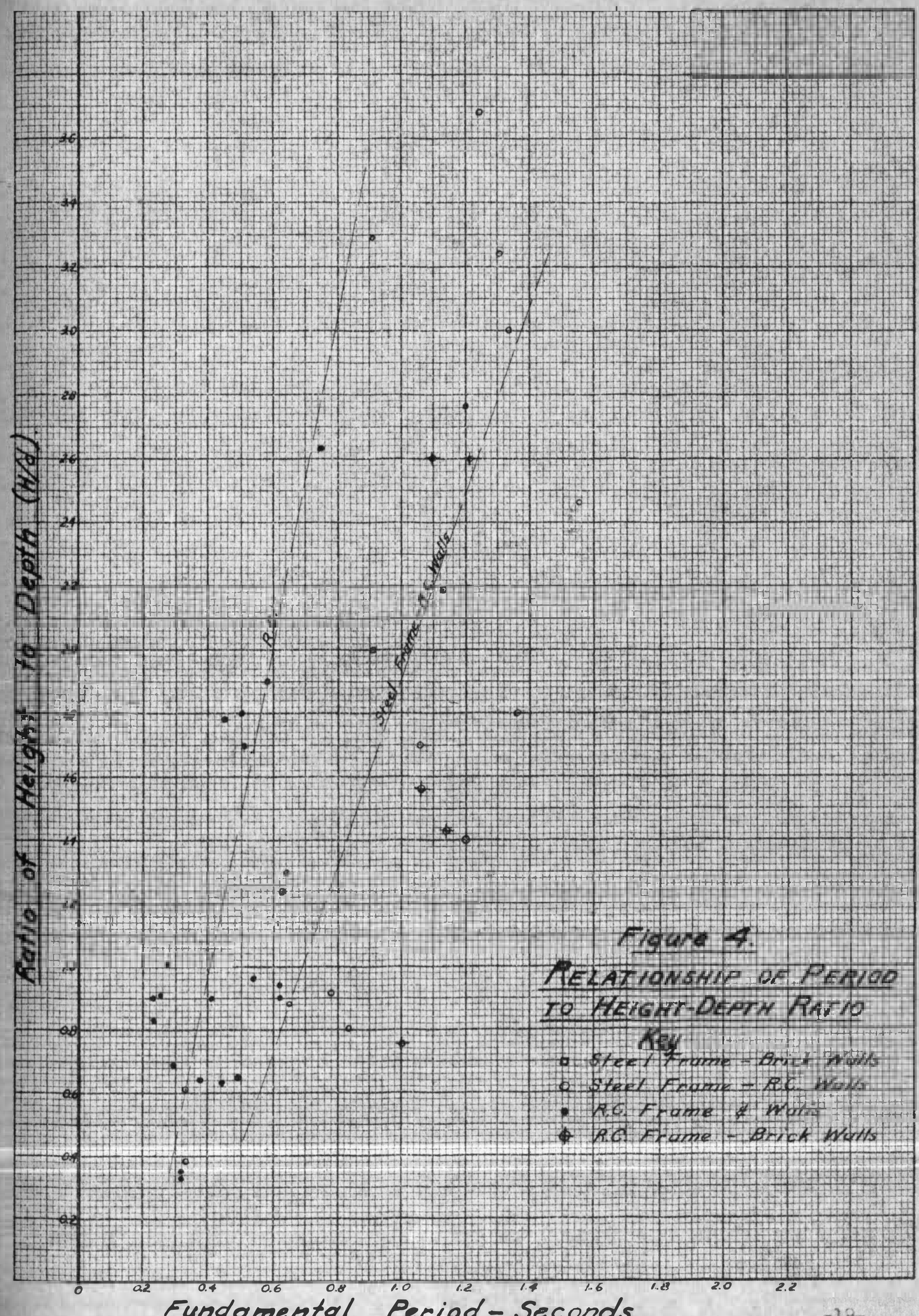


Figure 4.

RELATIONSHIP OF PERIOD TO HEIGHT-DEPTH RATIO

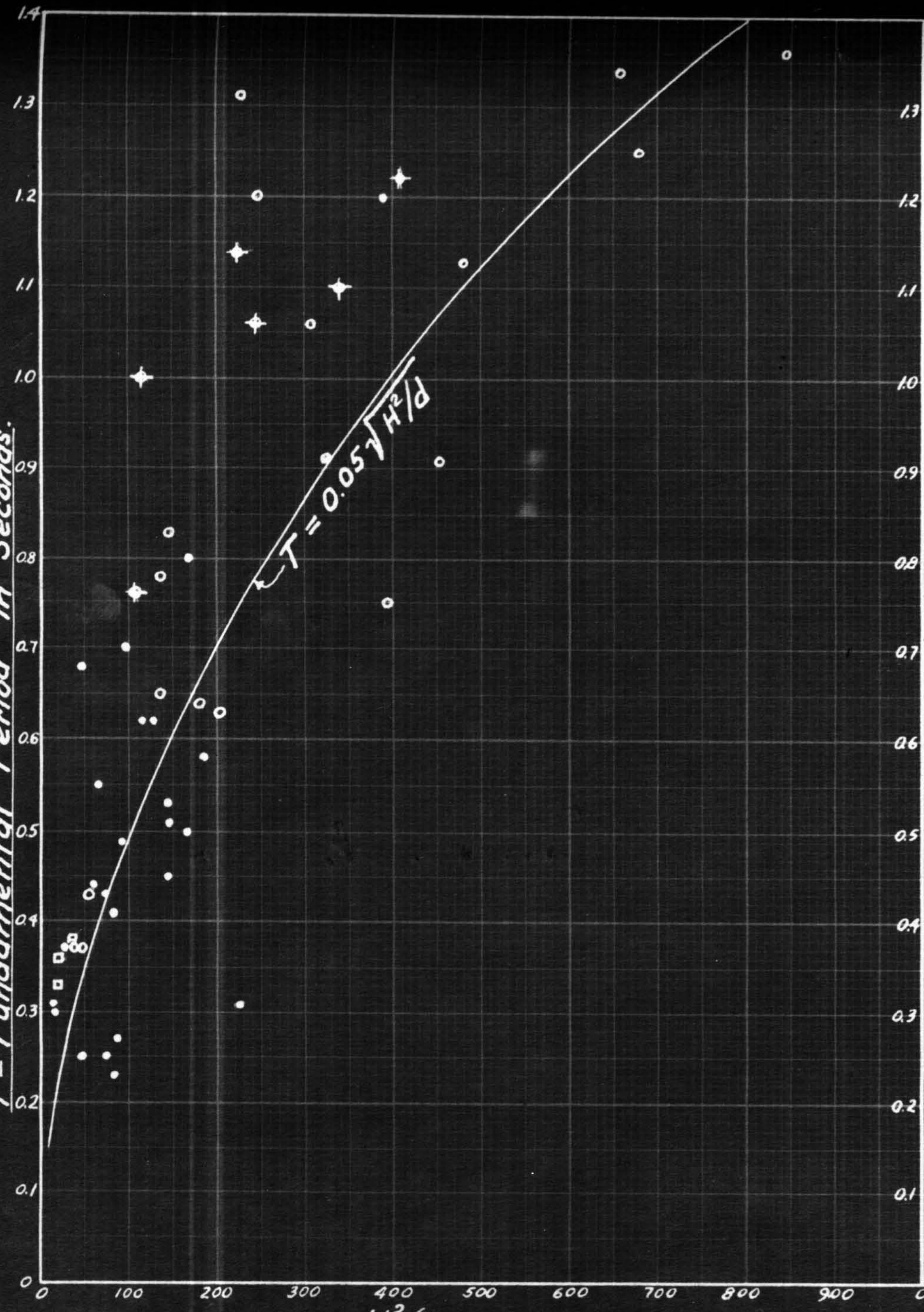


Figure 6 (Equation a)

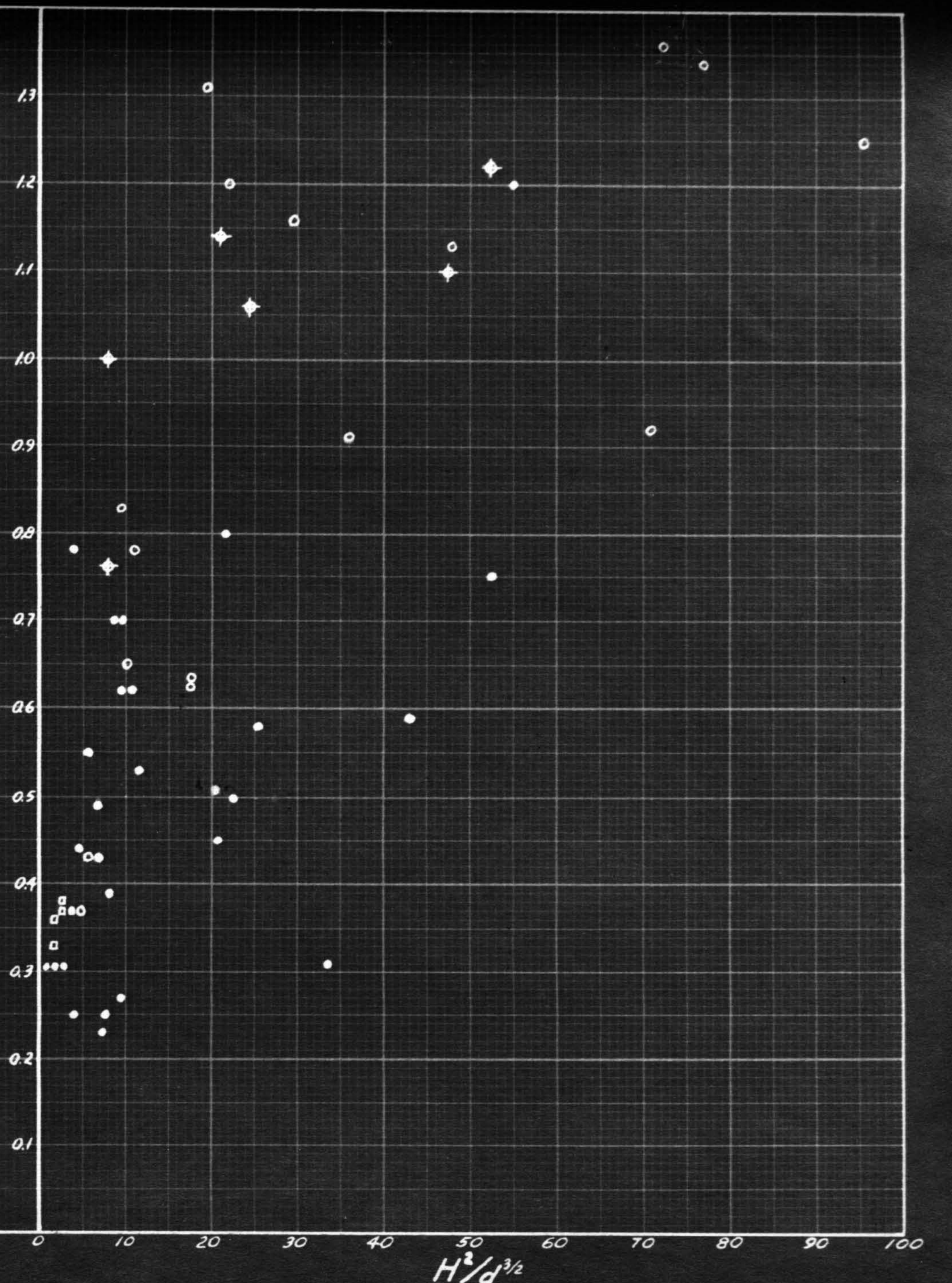


Figure 5 (Equation a)

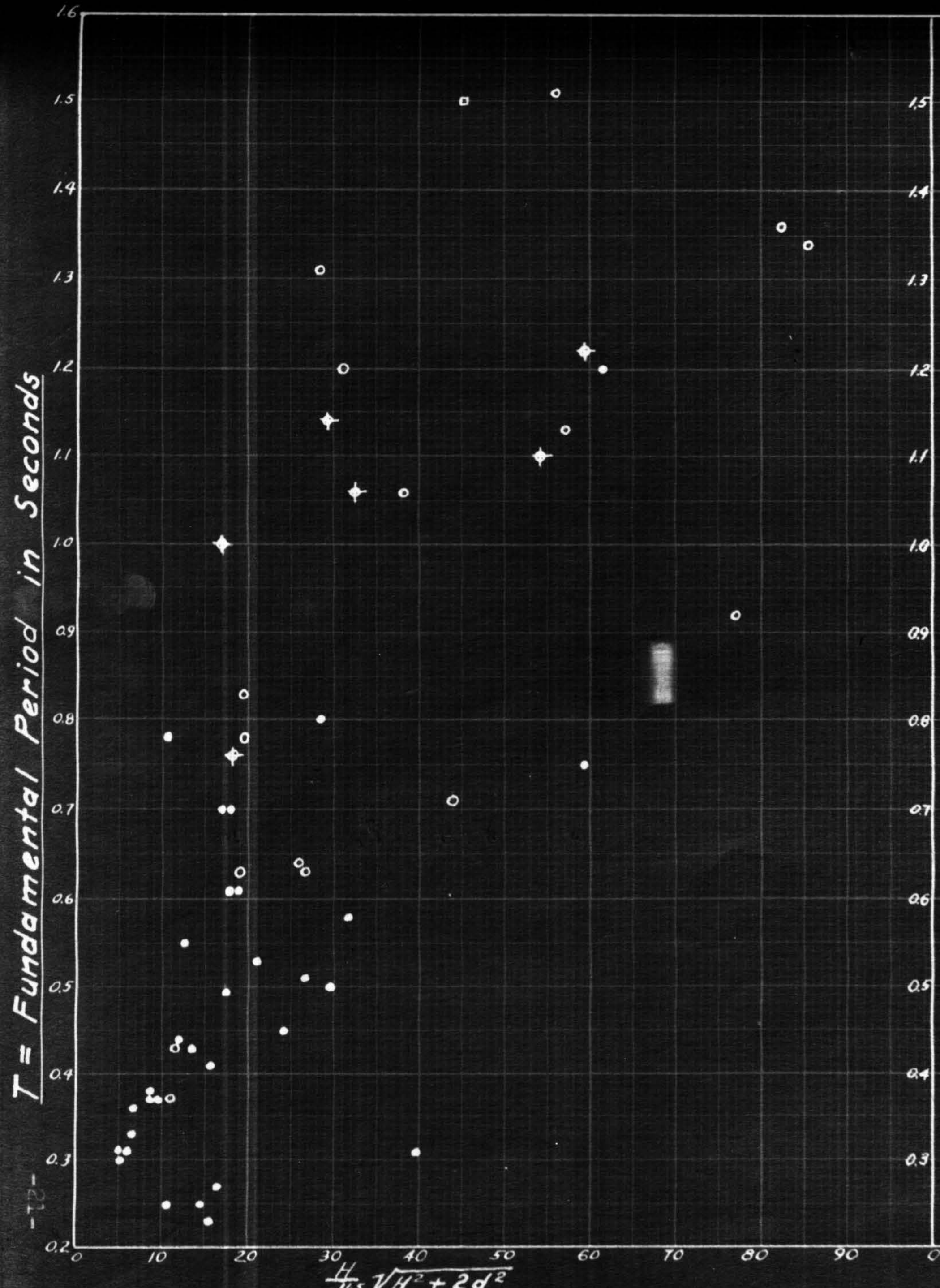


Figure 7. (Equation b).

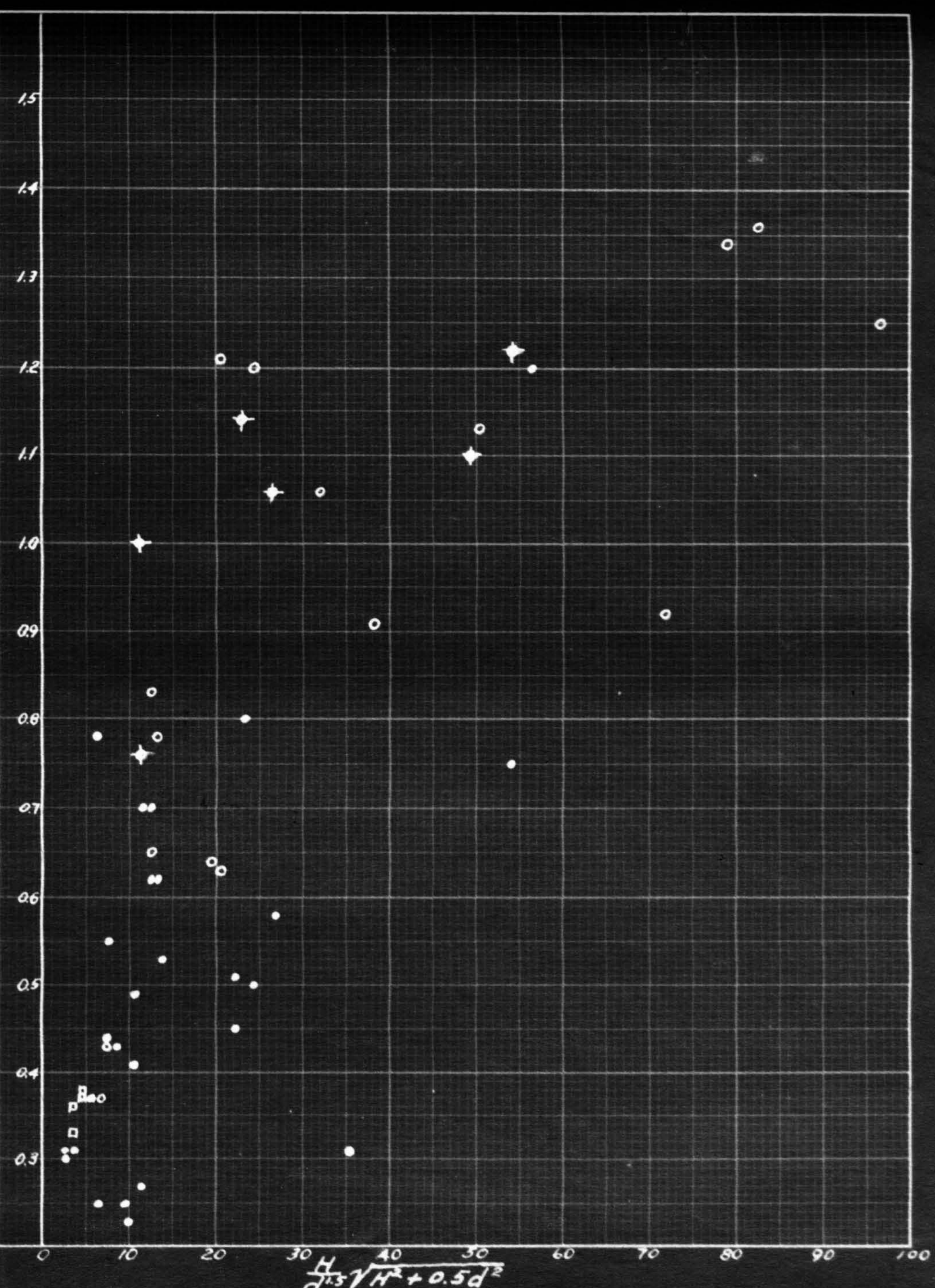


Figure 8 (Equation b)

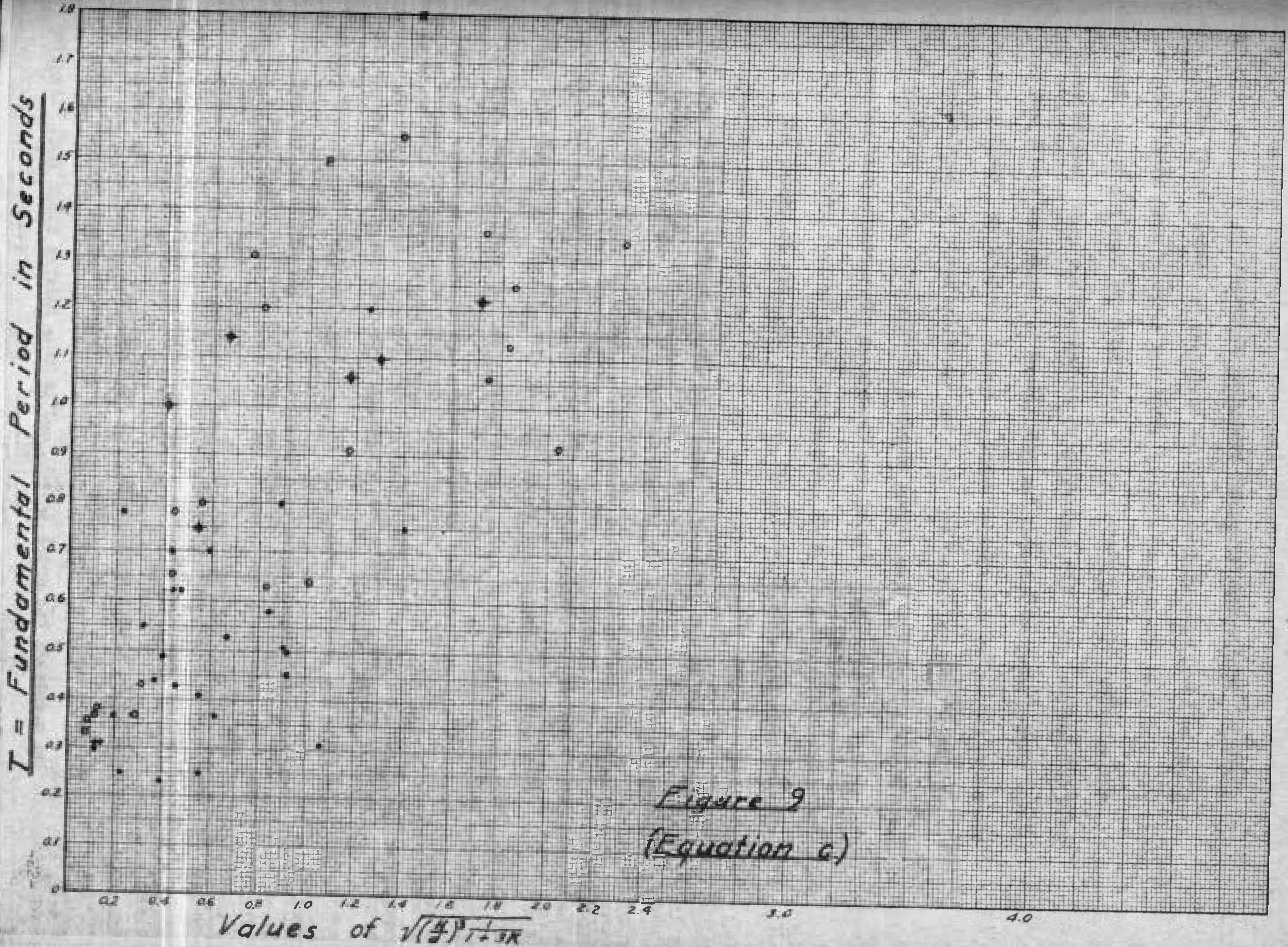


Figure 9
(Equation c)

DAMAGE TO BUILDINGS CAUSED BY EARTHQUAKES

The first earthquake resistant building designs were based on analysis of the damage done to other buildings during earthquakes. Such analysis is still one of our most valuable tools in determining how to design against these forces. Much was learned about earthquake resistant construction from the Long Beach earthquake of March, 1933 and from the Imperial Valley earthquake of 1940. Most of the damage in these earthquakes was in the improperly designed brick or stone buildings and from this lesson we have learned that all parts of a structure must be designed to act as a unit. In minor quakes there has been considerable damage and some loss of life from the falling off of parapets, cornices, or other appendages. In both of the two quakes mentioned the damage to buildings designed for a lateral force of approximately 0.1 gravity was negligible.

Two theories of design have been used, a flexible theory and a rigid theory. The flexible theory is based on making the structure flexible enough to vibrate with the earthquake without damage to the structure. In practice, however, it is very difficult to make the whole structure so flexible that this will occur and consequently those parts of the structure having some rigidity, such as light walls, riveted joints, etc., have suffered extensive damage or complete failure. The rigid theory is based on making the structure so rigid that it will dampen out the vibrations caused by the earthquake without damage to the structure. The design of structures based on the flexible theory has almost completely given way to the design based on the rigid theory.

Typical damage to reinforced concrete structures during the Long Beach earthquake was:

(1). A considerable number of columns cracked in flexure at the floor levels while the adjacent floor system did not. This indicated that the column was not adequately designed to take the moments caused by continuity.

(2). Walls, acting as lateral bracing, relieved the stresses on the columns. That walls carried most of the lateral force was indicated by

X-cracks in a number of walls. Cracks running out from the corners of windows were noted. Cracks occurred at some construction joints in walls.

(3). Spandrel beams continuous over several spans were cracked in their exterior bays. This indicated that the piers or columns evidently carried shear in proportion to their rigidities since the moment in interior piers would be distributed to two beams, while the moment in exterior piers would be distributed to only one spandrel beam.

(4). Columns supporting wood or steel roof systems failed in flexure at their bases in cases where their tops were not connected by concrete tie beams.

(5). Any effort at the interpretation of structural damage in terms of the acceleration alone brought the usual contradictions because of the equal importance of the period of acceleration and the stiffness of the structure. Accelerations in the Long Beach earthquake were measured as high as 0.23 gravity.

Inspection of a structure after an earthquake to determine the extent of structural damage is a difficult task. Usually the frame and resisting walls are covered with plaster or other ornamentation to such an extent that visual examination of the structural elements is an impossibility. Where walls are out of plumb or where cracks in the structural frame or walls are visible, serious damage has occurred. New cracks in plaster at juncture of walls or columns with beams, movement of roof trusses or rafters relative to bearing walls, separation of floors from bearing walls, and cracks in structural members indicate some damage to the structure. Minor damage such as the cracking of partitions and loosening of veneer may usually be easily discovered.

A method of determining damage to the structural frame has been suggested by Mr. J. J. Creskoff. His method is to measure the period of vibration of the structure before and after the earthquake. If the period has increased it is probable that damage to the structural elements has occurred.

BUILDING CODE REQUIREMENTS

Many building codes in the areas subject to earthquakes have adopted provisions for safeguarding structures against the forces imparted by earthquakes. These provisions are not proposed to make the structure earthquake proof but to make it earthquake resistant, within economical limits, against the magnitude of earthquake forces known to have occurred in the particular locality during recent history.

These codes base their provisions on the application of static horizontal loads proportional to the mass of the structure in a manner believed to be somewhat analogous to the forces resulting from an earthquake shock. There is considerable difference between the provisions of the various codes and they are presented here to illustrate this variance, which only emphasizes the fact that we have so much to learn about earthquake resistant design. Data is presented from the following codes:

- Uniform Building Code (1946 Edition) adopted by the Pacific Coast Building Officials Conference. This code is in quite general use in the smaller cities of the Pacific Coast.
- Los Angeles, Calif., Building Code (1948)
- State of California Administrative Code, Title 21, Public Works. This code governs the design of all public schools in California.
- Building Regulations for Reinforced Concrete, 1941, of the American Concrete Institute.

General Requirements. Every building or structure and every portion thereof shall be designed and constructed to resist the horizontal forces of wind and earthquake as such are specifically designated in the respective codes, provided however, that wind and earthquake forces need not be combined. The effect of continuity in construction shall be provided for in the design of all joints and members connecting thereto.

Increase in Allowable Stresses. As earthquakes occur infrequently all of the codes mentioned herein permit an increase in the allowable design stresses of 33 1/3 per cent. Allowable design stresses permitted by the various codes are shown in Table II. The ACI Code does not contain specific earthquake provisions but its allowable stresses are shown to illustrate that the other codes are essentially in complete agreement on these items.

Earthquake Force Formula. All of the codes express the design load by the formula

$$F = C\,(W_D + kW_L)$$

in which F = Lateral or horizontal force of the earthquake, applied in any direction.

W_D = Total weight (vertical dead load) of the portion considered.

W_L = Total live load (vertical) on the portion considered. See Table III

C = Coefficient depending on the structural element or on foundation conditions. See Table IV

k = Coefficient depending on the code being used and varying with the type of load. See Table V

The force "F" is applied through the center of gravity of the structure, or in the case of buildings it is applied as a series of loads acting at the various floor levels. Thus the term W_D is composed of the weight of the floor and contributing walls, columns, and partitions.

Distribution of Horizontal Shear. The total horizontal shear at any level shall be distributed to the various resisting units at that level in proportion to their rigidities, giving due consideration to the distortion of the horizontal distributing elements. The resisting units are considered to be the walls, piers, columns or other bracing in a vertical plane.

TABLE II. ALLOWABLE WORKING STRESSES FOR REINFORCED CONCRETE

Item	Uniform Code	Los Angeles Code	California Code	ACI Code
Compression in bending	$0.45f'_c$	$0.45f'_c$	$0.375f'_c$	$0.45f'_c$
Compression in bending adjacent to supports	$0.45f'_c$	$0.45f'_c$	$0.45f'_c$	$0.45f'_c$
Tension reinforcing				
—Intermediate Grade	20,000#/sq. in.	20,000#/sq. in.	18,000#/sq. in.	20,000#/sq. in.
—Structural Grade	18,000#/sq. in.	18,000#/sq. in.	18,000#/sq. in.	18,000#/sq. in.
—Web	20,000#/sq. in.	16,000#/sq. in.	16,000#/sq. in.	20,000#/sq. in.
Shear				
—no web reinforcing	$0.02f'_c$	$0.02f'_c$	$0.02f'_c$	$0.02f'_c$
—with web reinforcing	$0.06f'_c$	$0.06f'_c$	$0.06f'_c$	$0.06f'_c$
—with web reinforcing & special anchorage	$0.12f'_c$	$0.12f'_c$	———	$0.12f'_c$
Bond				
—plain bars	$0.04f'_c$	$0.04f'_c$	$0.04f'_c$	$0.04f'_c$
—deformed bars	$0.05f'_c$	$0.05f'_c$	$0.05f'_c$	$0.05f'_c$
Columns with vertical steel and ties	$P = A_g(.18f'_c + .8f_s p_g)$	$P = .18f'_c A_g + .8f_s A_s$	$P = .225f'_c A_g(1 + (n-1)p)$	$P = A_g(.18f'_c + .8f_s p)$
Columns with vertical steel and spirals	$P = A_g(.225f'_c + f_s p_g)$	$P = .225f'_c A_g + f_s A_s$	$P = A_g f_c(1+(n-1)p)$ $f_c = 300 + (.10 + 4p)f'_c$	$P = A_g(.225f'_c + f_s p)$
Compression reinforcing	18,000#/sq. in.	16,000#/sq. in.	nf_c	40% of minimum yield point

* $0.03f'_c$ with special anchorage

** Intermediate grade

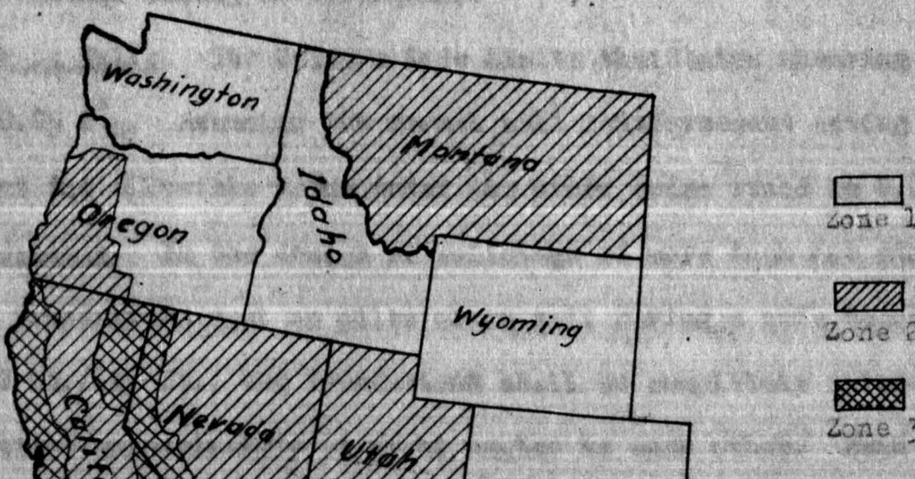
Combined bending and axial stress for all codes $f_c = \frac{t + De}{t + CDe}(f_a)$

TABLE III MINIMUM ALLOWABLE LIVE LOADS (IN POUNDS PER SQ. FT.)

TYPE OF LOAD	UNIFORM CODE	L. A. CODE	CALIF. CODE
Apartments, dwellings	40	40	
Auditoriums, movable seats	100	100	100
Dance floors	100	100	100
Garages	100	100	100
Gymnasiums	100	100	100
Libraries, reading rooms	60	50	60
Libraries, corridors	100	100	50
Libraries, stack rooms	125		125
Manufacturing, light	75	100	125
Manufacturing, heavy	125		
Marquees	60	60	50
Offices	50	50	50
Roofs	20	20	20
Schools, class rooms	40	40	50
Schools, corridors	100	100	50
Stairways	100	100	75
Storage, light	125	100	125
Storage, heavy	250	200	
Stores, retail	75	100	
Stores, wholesale	100	100	
Wind, above 60' elev.	20	20	20
Wind, below 60' elev.	15	15	15

TABLE IV. VALUES OF THE COEFFICIENT "C".

Portion of the Structure	Uniform Code	L. A. Code	Calif. Code	
Building as a Whole	$\frac{60}{N+4.5}$ **			
Allowable soil pressure-- Less than 2000 lb./sq. ft. 2000 lb./sq. ft. or more	 .04 * .02 *			
4000 lb./sq. ft. or less 4000 lb./sq. ft-8000 lb./sq. ft. More than 8000 lb./sq. ft.			.10 .08 .06	
Piles Caissons			.10 .08	
Bearing walls, non-bearing walls, partitions, curtain walls, enclosure walls, panel walls.	.05* with min. of 5#/sq.ft.	.20		
Cantilever parapet & other cantilever walls	.25 *	1.00	1.00	
Exterior & interior ornamentations & appendages	.25 *	1.00	1.00	
Towers, tanks, towers & tanks plus contents, chimneys, smokestacks, and penthouses when connected to or part of a building	.05 *	.20	.20	



* Coefficient shown in Table for Zone 1.
For Zone 2, multiply by 2
For Zone 3, multiply by 4

** N is the number of stories above the story under consideration.

Map of the 11 Western States
Showing Zones of Approximately Equal
Seismic Probability
For use in determining the value of "C" in the formula $F = C(W_D + W_L)$ in the Uniform Code

TABLE V VALUES OF THE COEFFICIENT "k"

Type of Structure	Uniform Code	L. A. Code	Calif. Code
Warehouse	1.0	1.0	0.75
Tanks	1.0	1.0	0.75
Other Storage	1.0	1.0	0.75
Others	0.5	0.0	0.60

Horizontal Torsional Moments. The vertical structural units of the building which resist the force of the earthquake shall be so arranged that, in any horizontal plane, the centroid of such resisting structural units is coincident with the center of gravity of the weight of the building, or else proper provision shall be made for the resulting torsional moment in the building. The junctures between wings and the main portion of a building may be designed for these rotational moments, or the juncture may be made by means of sliding fragile joints having a width sufficient to prevent the two parts of the building from knocking together during an earthquake.

Shear in Walls. The Uniform Code limits the design shearing stress in walls to $0.05\ f'_c$. Assuming the normal wall reinforcement acting as web reinforcement the allowable value under the other codes would be $0.06\ f'_c$.

Foundations. In the design of buildings of more than one story for which the foundations rest on piles or on soil having a safe bearing value of less than 2,000 lb./sq. ft., the foundations shall be completely inter-connected in two directions approximately at right angles to each other. Each such interconnecting member shall be capable of transmitting by both tension and compression at least 10 per cent of the total vertical load carried by the heavier only of the footings or foundations connected. The minimum size of such a reinforced tie shall be 12" by 12" and it shall be reinforced with not less than four 5/8 inch round bars and tied with at least ¼" round ties spaced not greater than 12". The foundation ties shall be at the level of the pile caps.

Overturning Moment. In no case shall the calculated overturning moment of any building or structure due to the earthquake forces exceed two thirds of the moment of stability of such building or structure. The Uniform and Los Angeles Codes permit the moment of stability to be calculated using the same vertical loads that were used in determining the overturning moment while the California Code requires the moment of stability to be calculated from the dead load only. Soil pressures may be increased 33 1/3 per cent when resisting horizontal forces.

PART II

PROCEDURE FOR THE DESIGN OF A REINFORCED CONCRETE BUILDING FOR SEISMIC FORCES.

The design of reinforced concrete structures will be discussed in ten steps as follows and an example will be shown in Part III.

1. The building, or structure, will first be designed for all static loads, i.e. dead load, live load, and wind forces. The principle structural elements of the building should be made as symmetrical about both horizontal and vertical axes as possible since this will eliminate torsional vibrations and stresses and greatly increase the stability of the building against earthquake forces.

2. Determine the coefficients for seismic loading. These coefficients should be based on proximity of the location to active faults, dominant periods of the soil in the area, seismic probability, probable fundamental period of the structure, and building code requirements. The location of active faults and the dominant periods of soils in some areas may be obtained from the U. S. Coast & Geodetic Survey. In general if the soil on which the foundations rest is very hard and compact with a high elastic value, its damping value will be poor and it will have a low amplitude. However if the soil is softer, non-uniform in character, or contains considerable moisture, it will be less elastic, have a higher damping value and will have a higher amplitude of vibration.

The seismic probability of the area may be obtained from the Seismic Probability Map of the U. S., published by the U. S. Coast & Geodetic Survey (See Figure 1). The fundamental period of the structure may be estimated very roughly from Figure 6. This fundamental period is dependent upon so many variables such as size, shape, mass, rigidity and damping characteristics that its computation is usually too time-consuming to be included in practical design procedures. To avoid resonant vibration, the fundamental period of the building should not be the same as the dominant period of the soil.

The seismic force is based upon the deal load of the floor and tributary walls and the live load carried by the floor; its magnitude is equal to the dead load plus a portion of the live load depending on the type of occupancy, both multiplied by the seismic coefficient. The seismic load is applied as a horizontal force acting at each floor level or as a uniform horizontal load acting on such vertical members as walls, columns, or piers. Where the governing building codes specify these seismic coefficients, the specified coefficients will be the minimum allowable and need be increased only in such instances as a study of seismic probability and magnitude may so indicate. It should be noted that different parts of the structure are subject to different coefficients. (See Table IV.)

From Newtons second law $F = M\,a = W\,a/g$. Assuming harmonic motion

$$a = \frac{V^2}{R}\cos(\alpha t) = \frac{4\pi^2 R}{T^2}\cos\left(\frac{2\pi}{T}t\right)$$

then $$F = \frac{W}{g}a = \frac{W}{g}\frac{4\pi^2 R}{T^2}\cos\left(\frac{2\pi}{T}t\right) = \frac{4W\pi^2 R}{gT^2} \qquad (Max.)$$

in which F is the seismic force, M is the mass, a is the acceleration, g is the acceleration due to gravity, v is the horizontal velocity, R is half the amplitude, T is the period, t is the time, and is the angular velocity.

During the 1906 earthquake in San Francisco it was reported that the period was 1 second and the amplitude about 4 inches. Then:

$$F = CW = \frac{4\pi^2 R}{gT^2}W = \frac{4\pi^2 2}{32.2 \times 12 \times 1^2}W = 0.205\,W$$

In 1923 it was assumed that the normal period of the Tokyo earthquake was 1.5 seconds, while the amplitude was assumed to be 4 inches. Therefore:

$$C = \frac{4\pi^2 2}{32.2 \times 12 \times (1.5)^2} = .091$$

These values represent typical coefficients for two of the most destructive earthquakes on record.

3. Check the walls for transverse loads. In the ordinary case a wall which is adequate to withstand wind stresses will also be adequate to withstand the transverse horizontal seismic load. However some cases demand special attention.

Parapet walls. These walls have a very poor record of performance during past earthquakes. The trouble has usually been that they have been inadequately tied to the main portion of the structure resulting in their complete collapse and falling to the street in many cases. During an earthquake a parapet wall acts as a vertical cantilever wall subjected to a uniform horizontal force. In zones 3 of the Earthquake Probability Map most building codes require that the parapet walls be designed for a lateral force equal to its own weight or

$$F = C\ w\ H \qquad \text{in which } C = 1.00$$

At the junction of the parapet wall and roof the moment will equal

$$M = \tfrac{1}{2}\ C\ w\ H^2$$

which may cause tenion on either face depending on the direction of the force. Reinforcement should be placed in the amount required by this moment and for temperature change requirements. Typical details for reinforcing in one or two layers are shown in Figure 10.

Curtain walls. These walls should be designed for the transverse seismic load which may act on them and in the case of exterior curtain walls they should also be designed for wind stresses. Although a wall is ordinarily supported on four sides it is customary to design these walls to carry the full horizontal load over the shortest span, which in buildings is usually in the vertical direction. If the wall is built integrally with floors and columns it could be considered to be at least partially fixed against rotation at its supports but since the seismic force may act in either direction, it is on the safe side to consider the wall as a simple beam and reinforce both faces equally. In that case the moment becomes:

$$M = \pm\ C\ w\ H^2/8$$

in which C is the seismic coefficient specified for walls. In most cases, reinforcement specified for temperature change will also be adequate for seismic loads.

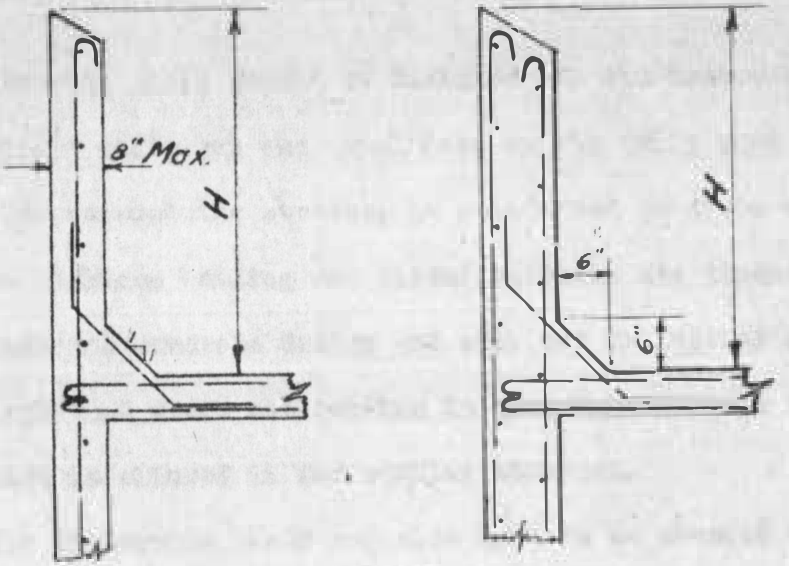


Figure 10. Parapet Wall Details.

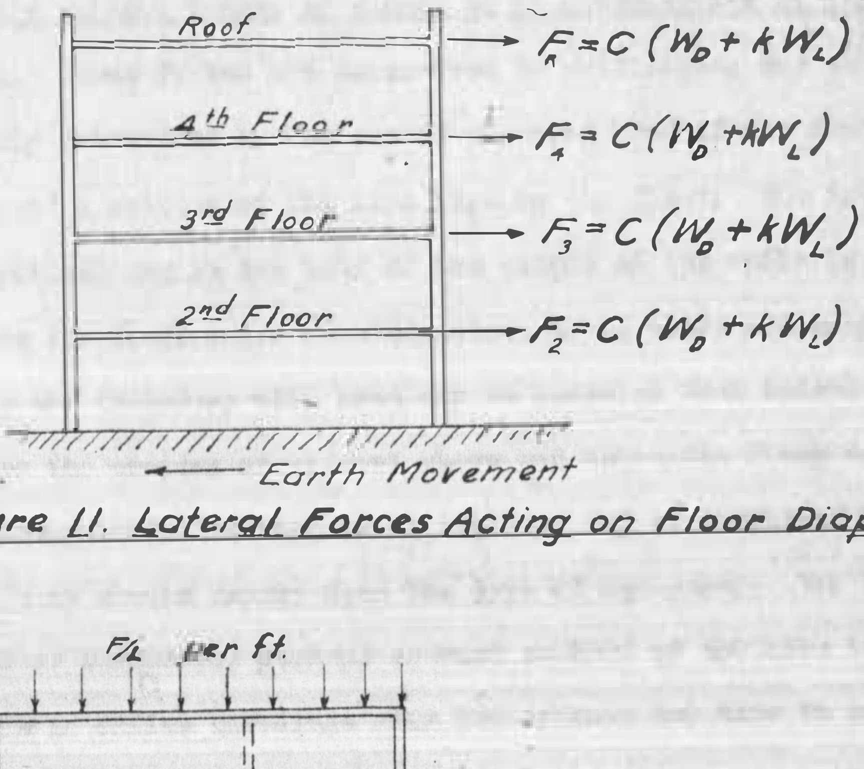


Figure 11. Lateral Forces Acting on Floor Diaphragm.

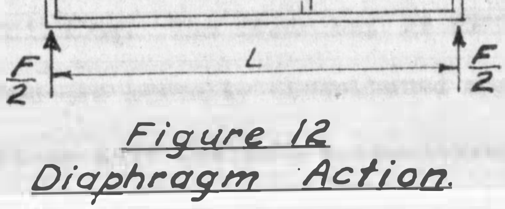


Figure 12
Diaphragm Action.

Bearing walls. Bearing walls should be designed for the transverse seismic loading as for curtain walls but the axial load on the walls must also be considered. Methods for determining stresses in reinforced concrete walls and columns subject to combined bending and direct stresses are discussed in all text books on reinforced concrete design and will not be further discussed here except to point out that where the bending is caused by seismic forces an increase of one-third is allowed in the working stresses.

Walls designed for transverse loads may also have to be checked for shearing stresses in their longitudinal direction but will not be subject to both transverse and longitudinal seismic forces at the same time.

4. *Calculate the seismic forces or shears to be concentrated at the various floor levels.* These forces are determined by multiplying the seismic coefficient previously determined by the sum of the dead load of the floor and tributary walls and a portion of the live load on the floor. The tributary walls to be included may be one half of the weight of the walls in the story above and below the floor under consideration, or if there are many windows in the walls the tributary wall load may be taken as that weight of wall included between the centers of windows above and below the floor under consideration. The amount of live load to be included may be specified by the code but in any case should depend upon the type of occupancy. For instance in the design of warehouses perhaps as much as 100% of the live load may be included while in office buildings only 50% or less may have to be included. The amount of live load to be included should be dependent upon the percentage of time that full live load may reasonably be expected to be present in the building. The force may be formulated as: $F = C(W_D + k\,W_L)$. Since these forces are actually distributed approximately uniformly over the floor area the floor must act as a transmitting diaphragm to carry the force to the vertical resisting elements.

5. <u>Design the floor diaphragms to carry the horizontal seismic floor load to the resisting columns and walls.</u> All seismic shears must be carried to the ground through the resistance of walls, piers, and columns. The design of floor diaphragms, therefore, must be considered in relation to the capacity, of the walls, piers, and columns supporting the floor, to resist the shears. The simplest case is that of a floor with no openings, or symmetrical openings, supported on end walls of equal rigidity. (See Figure 12) In this case the reaction of each wall (a and b) is equal to F/2 and the floor must be designed to resist the resulting shearing and flexural stresses. If walls a and b are not of equal rigidity, or if another wall is introduced at c, the flexibility or rigidity of the diaphragm must be considered.

<u>Flexible diaphragms</u>. In Figure 12, if the floor diaphragm has less rigidity than the wall c (i.e., its deflection between a and b is greater than that of wall c), the force F should be distributed to the supporting walls in proportion to the floor areas adjacent to the respective walls. In this case the shear in the floor adjacent to walls a and c would be the greatest and would be one-half of the portion of F acting between a and c. Correspondingly, the total shear taken by wall c is F/2 with the remaining shear divided between walls a and b.

<u>Rigid Diaphragms</u>. In Figure 12, if the floor diaphragm has greater rigidity than the wall c (i.e., its deflection between a and b is less than that of wall c), the force F should be distributed to the supporting walls in proportion to the relative rigidity of the walls themselves. Most concrete floors, without an abnormal number of openings, are of the rigid type.

In case there is doubt whether the floor is more rigid than the walls, the floors may be quickly checked for shearing and flexural stresses for both conditions of distribution.

6. <u>Distribute the seismic shears to the vertical resisting members (i.e., walls, piers, and columns)</u>. As previously stated, the seismic shears

must be carried to the ground through the shear and moment resistance of walls, piers, and columns. Where cross walls are available they offer a great deal more rigidity than columns and in those cases where both walls and columns are available practically all seismic shear will be taken by the walls. If there are no walls rigid frames may be used to transmit the shears; this must be done in open front buildings, for example.

In cases where the floor diaphragms are more flexible than the supporting walls, each wall must be designed to carry one-half the horizontal floor shear between itself and adjacent walls. The wall will then be designed as shown in step 8.

In cases where the floor diaphragms are more rigid than the supporting walls or rigid frames, each wall or rigid frame must be designed to carry a portion of the horizontal floor shear in proportion to its relative rigidity. The walls may have openings of various sizes which divide the wall into a series of piers of different sizes. The problem, then, is to determine the relative deflections at the floor level of the different walls, since rigidity if defined as $1/\Delta$, where Δ is the total deflection due to a unit force.

The applicable formulas for deflection will depend upon the conditions of end restraint of the walls or piers. Since the height to depth ratio is likely to be small in many cases (sometimes less than one) the deflection caused by shear must be considered as well as that caused by moment. Assuming the foundations to rest on unyielding soil, or assuming the footings so designed as to produce equal settlement, the bottom of the walls may be considered fixed. Three walls of equal story height and width are shown in Figure 13. In Figure 13 all of the piers in (a) may obviously be considered fixed at both ends. In Figure 13 (b) the shallow beam H has insufficient rigidity to fix the top of piers J and L while pier K may probably be considered fixed at both ends. In Figure 13 (c) the wall may be considered fixed at both top and

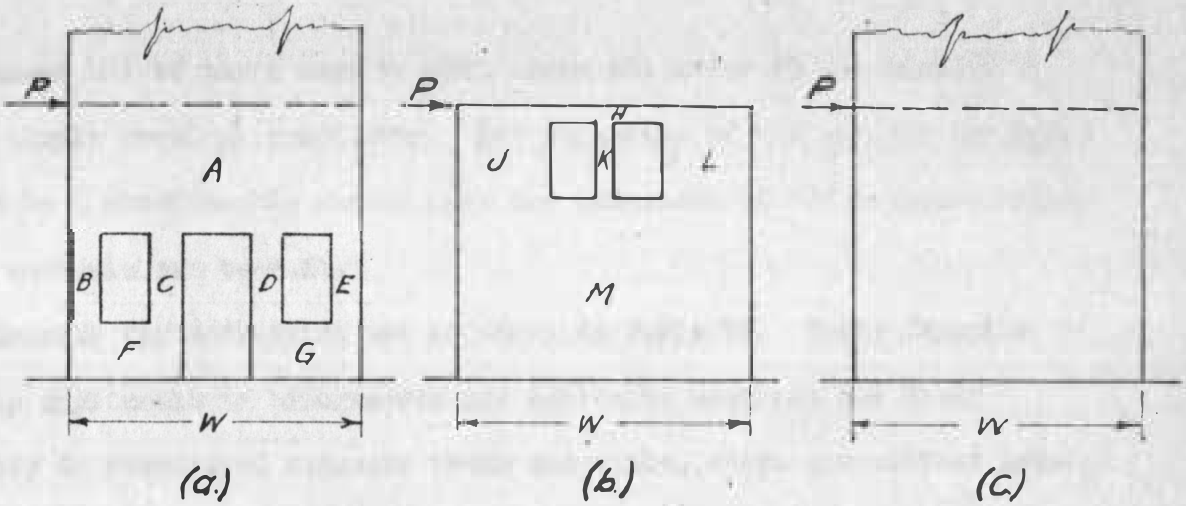
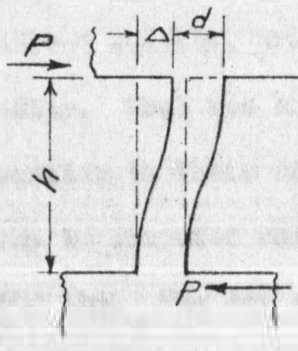
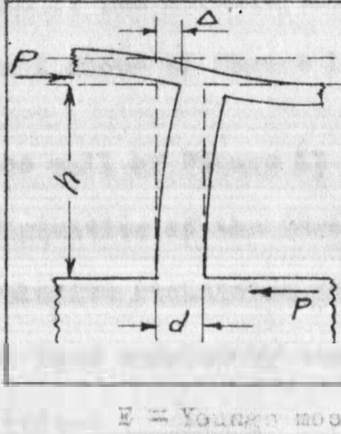


Figure 13

Table VI. Deflection of Piers			
	Moment Δ	Shear Δ	Δ
	$\Delta_m = \frac{Ph^3}{12EI}$	$\Delta_s = \frac{6}{5}\frac{Ph}{E_s A}$ for rectangular sections $\Delta = \frac{10}{9}\frac{Ph}{E_s A}$ for circular sections	$\Delta = \Delta_m + \Delta_s$
	$\Delta_m = \frac{Ph^3}{3EI}$	$\Delta_s = \frac{6}{5}\frac{Ph}{E_s A}$ for rectangular sections $\Delta_s = \frac{10}{9}\frac{Ph}{E_s A}$ for circular sections	$\Delta = \Delta_m + \Delta_s$

E = Youngs modulus of elasticity
E_s = Modulus of elasticity in shear, usually considered equal to 0.4 E
I = Cross-sectional moment of inertia in the direction of bending (for rectangular section = $bd^3/12$)
A = Cross-sectional area (for rectangular section = bd)

bottom as shown but if there were no wall above the floor at the load P, it would be a simple vertical cantilever. The selection of the correct condition will depend to a considerable extent upon the judgement of the designer unless a rigorous analysis can be made.

The formulas for deflection are as shown in Table VI. These formulas are strictly applicable to homogeneous and isotropic sections and their applicability to reinforced concrete beams and walls, which are neither homogeneous nor isotropic, and to which Hookes Law does not fully apply, may be questioned. However we are concerned only with relative deflection and considering also that the values of E, E_s, and I can be only approximate at best, further refinement would probably be unnecessary.

If the floor diaphragm is very rigid, the supporting walls and frames will deflect equally, providing the center of mass coincides with the center of rigidity. Thus the horizontal forces will be distributed to the cross walls in proportion to their rigidities. The deflection of each of the supporting walls may be computed under the action of equal horizontal forces, say P = 1,000,000 lbs. The deflection of a wall consisting of a series of piers is the sum of the deflections of the piers on each level. Consider, for example, the wall shown in Figure 13 (a):

$$\Delta_a = \Delta_A + \frac{1}{\frac{1}{\Delta_B}+\frac{1}{\Delta_C}+\frac{1}{\Delta_D}+\frac{1}{\Delta_E}} + \frac{1}{\frac{1}{\Delta_F}+\frac{1}{\Delta_G}}$$

For the wall of Figure 13 (b):

$$\Delta_b = \Delta_H + \Delta_M + \frac{1}{\frac{1}{\Delta_J}+\frac{1}{\Delta_K}+\frac{1}{\Delta_L}}$$

The rigidities of the three walls of Figure 13 are then: $\frac{1}{\Delta_a}$; $\frac{1}{\Delta_b}$; $\frac{1}{\Delta_c}$

The relative rigidities of the walls and hence the proportion of total horizontal load carried by each wall are:

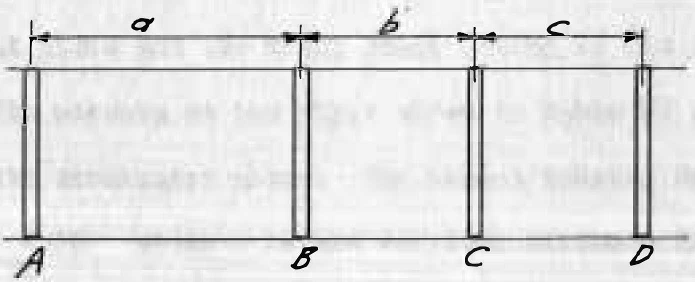
(a)--- $\dfrac{\frac{1}{\Delta_a}}{\Sigma \frac{1}{\Delta}}$ (b)--- $\dfrac{\frac{1}{\Delta_b}}{\Sigma \frac{1}{\Delta}}$ (c)--- $\dfrac{\frac{1}{\Delta_c}}{\Sigma \frac{1}{\Delta}}$

It should be pointed out here that the total horizontal shear to be applied at each floor may not be the same for the design of the floor diaphragm as for the shear walls. If dynamic forces are considered the seismic coefficients will not be the same for successive floors and may even be negative at some

elevations. Furthermore, the walls must be designed for the summation of all horizontal forces above the wall itself while the floor is designed for the horizontal force of inertia of its own weight. Variations of coefficients according to the laws of dynamics are considered in the Los Angeles Code.

7. <u>Correct distribution of shears to walls, etc., for torsional effect.</u> In step 6 the horizontal seismic shears were distributed to the supporting walls in proportion to their rigidities. If the center of mass of the floor load coincides with the center of rigidity of the supporting walls this distribution will be correct. If not, a correction must be made to allow for the torsional moment developed by the amount of eccentricity.

The center of mass will be located at the center of the weight of the floor. The center of rigidity may be found by taking static moments about the center line of one of the end walls, using the relative rigidities of the walls as weights. Thus-



From A--Center of mass $= \frac{a + b + c}{2}$

From A--Center of rigidity $= R_B(a) + R_C(a + b) + R_D(a + b + c)$

where R if the relative rigidity of the wall indicated in the subscript and a, b, and c are the distances between center lines of walls. The torsional moment is the product of the horizontal force and the distance between the centers of mass and rigidity.

In resisting this torsional moment the walls will be subject to additional deflections over those found in Step 6. Those walls on the same side of the center of rigidity as the center of mass will be subjected to an additive deflection while those walls on the opposite side will be subjected to a subtractive or relieving deflection. It is customary to design the walls with

additive deflections for the total deflection thus found but those walls with subtractive deflections are designed for the deflections of step 6 without the subtraction being made. An example of these computations is shown in Part III.

The walls and piers are then designed for the maximum shears applied to them.

8. Design the shear walls for combined horizontal and vertical loads. The walls must be designed for a) shearing stresses caused by the horizontal seismic shear, b) flexural stresses, tension and compression, caused by beam action and overturning, and c) compression due to gravity loads.

a). The shearing stress equals P/A and if it exceeds the value allowable for concrete alone it must be provided for by wall reinforcement.

b). The moments causing flexural stress will be caused by the shears taken by individual piers and the total shear acting on the wall and tending to overturn it. The moments on the piers shown in Table VI are Ph/2 and Ph respectively for the conditions shown. The moment causing overturning may be represented by ΣPH where H is the vertical distance from the total horizontal shear ΣP to the plane on which the stresses are being computed. As an approximation the flexural formula Mc/I may be used to compute stresses, or more exact methods taking into account the correct position of the neutral axis may be used. Any net tension must be taken by the steel reinforcement.

c). Gravity loads on the wall from the roof or floors above it will cause compression stresses in the wall.

Methods for the design of walls subject to combined axial and bending stresses are available in standard textbooks on reinforced concrete and will not be repeated here.

9. Check overturning pressures on footings.

The effect of the overturning moment of the horizontal forces is to increase the vertical footing pressure on the side of the building in the dir-

ection of the force. Most building codes allow an increase of one-third in soil pressures for combined loading with the requirement that the moment of stability of the vertical loads be one and one-half times the overturning moment. The foundations designed for vertical loads must then be checked for these requirements.

An adequate, rigid foundation greatly increases the resistance of a building to earthquake forces. Individual footings should preferably be tied together with reinforced concrete ties or struts capable of transmitting at least 10% of the maximum load of the heaviest footing in either tension or compression. This is especially important if the building rests on a soft soil or if the footings are at substantially different elevations or are very far below the ground floor. This interconnection is easily accomplished and usually at little added expense.

10. Calculate the fundamental period of the building and check for resonance. The calculation of fundamental periods of multistory structures involves the solution of "n" differential equations where "n" is the number of stories. Several shortcuts in the solution have been proposed. Mr. John E Goldberg, in the Proceedings of the American Concrete Institute, Sept. 1939, developed the formula: $T = 2\pi\sqrt{\frac{\Sigma(m\Delta^2)}{\Sigma(F\Delta)}}$
in which "F" are the forces at each floor, developed by a method of trial and error in the ratio to the deflection and the masses "m", and Δ equals the deflection at the respective floor.

Another approximation is given by Mr. M. V. Pregnoff in the same publication in 1940 as follows: $T = 1.1\sqrt{\frac{\Sigma(W\Delta^2)}{\Sigma(W\Delta)}}$
in which "W" equals the weight of the structure at each floor in pounds and Δ equals the movement of each floor relative to the ground in feet by lateral loads equal to its own weight.

Should resonance with the natural period of the soil occur, it may be necessary to increase the seismic coefficients.

PART III

DESIGN OF A TWO STORY REINFORCED CONCRETE SCHOOL BUILDING FOR CLASSROOM USE

The design procedure outlined in Part II will now be applied to an actual example, the design of a two story reinforced concrete school classroom building. This design was governed by the provisions of both the Los Angeles Building Code and the State of California Administrative Code, Title 21. (See Part I.)

The elevations and roof and floor plans are shown in Figures 14 and 15. The gabled roof was of joist construction with a $2\frac{1}{2}$ inch slab and 6 inches wide by 12 inches deep joists 3 feet on centers. The joists were supported on four beams, two exterior and two interior, running the long dimension of the building. The second floor and a portion of the first floor were also of joist construction consisting of a 2 3/4 inch slab on joists 3 feet on centers and formed with metal pans 30 inches wide and 14 inches deep. The beams were similar to those of the roof. The remainder of the first floor was a 4 inch slab resting on fill. The interior columns were 14 x 14 inches resting on individual spread footings and the side walls were 12 inches thick with the piers between windows being designed as columns. The end walls were 10 inches thick and interior shear walls were of varying thickness as noted later. Footings under all walls were continuous wall footings and the bottom of all footings, column and wall, was $7\frac{1}{2}$ feet below the elevation of the first floor.

The building was designed to use a concrete having an ultimate compressive strength of 3000 lb. per square inch and the California Code limited the working compressive stress in the concrete to 1125 lb. per square inch ($K =$ 189). The California Code also limited the reinforcing steel stress to 18,000 lb. per square inch. The foundation material was tannish gray coarse sand to coarse gravel having an allowable bearing power of 6,000 lb. per sq. ft. for vertical load only or 8,000 lb. per sq. ft. for combined load. Design loads pertinent to the seismic analysis follow:

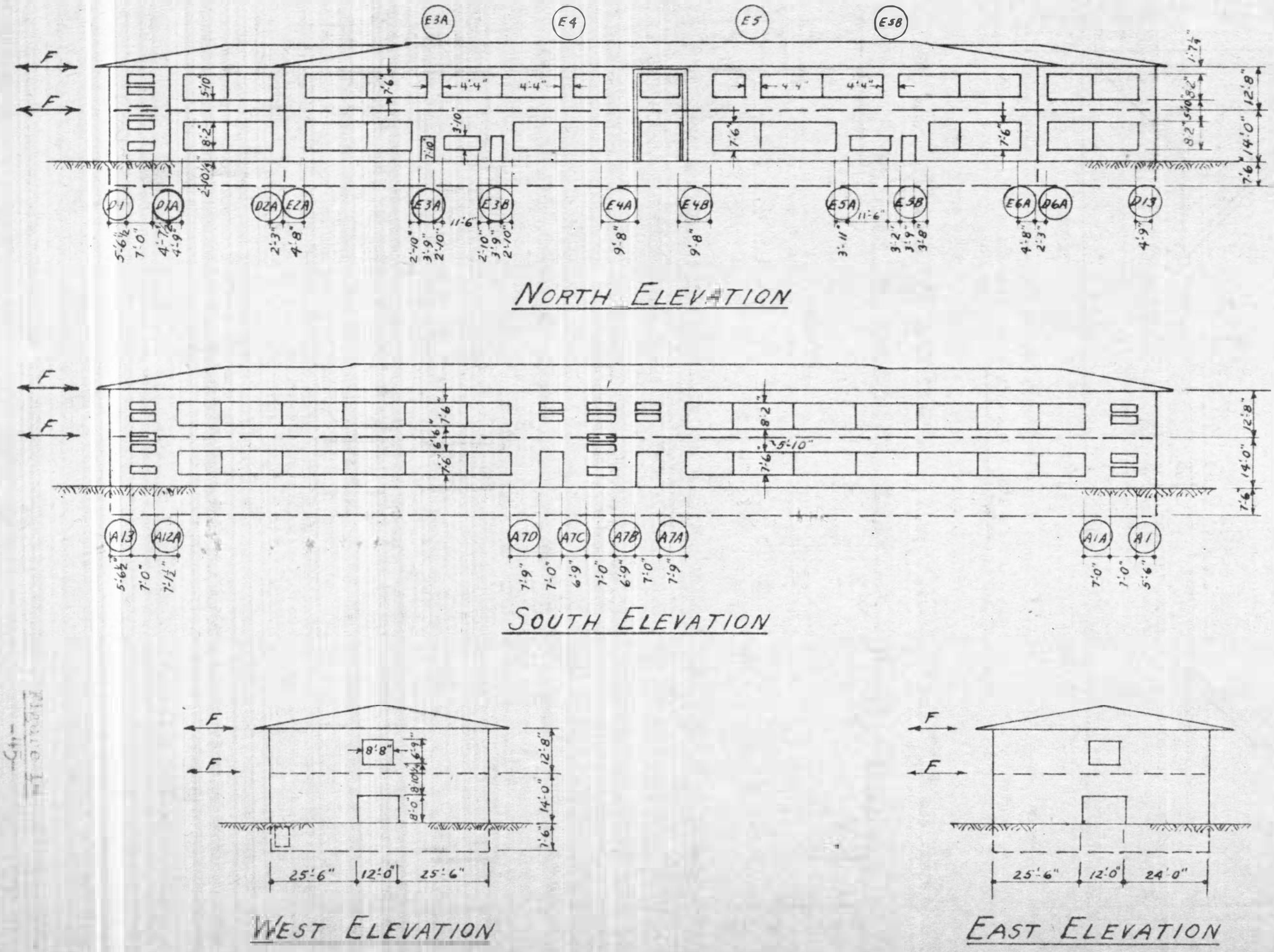

NORTH ELEVATION
SOUTH ELEVATION
WEST ELEVATION
EAST ELEVATION

Figure 14

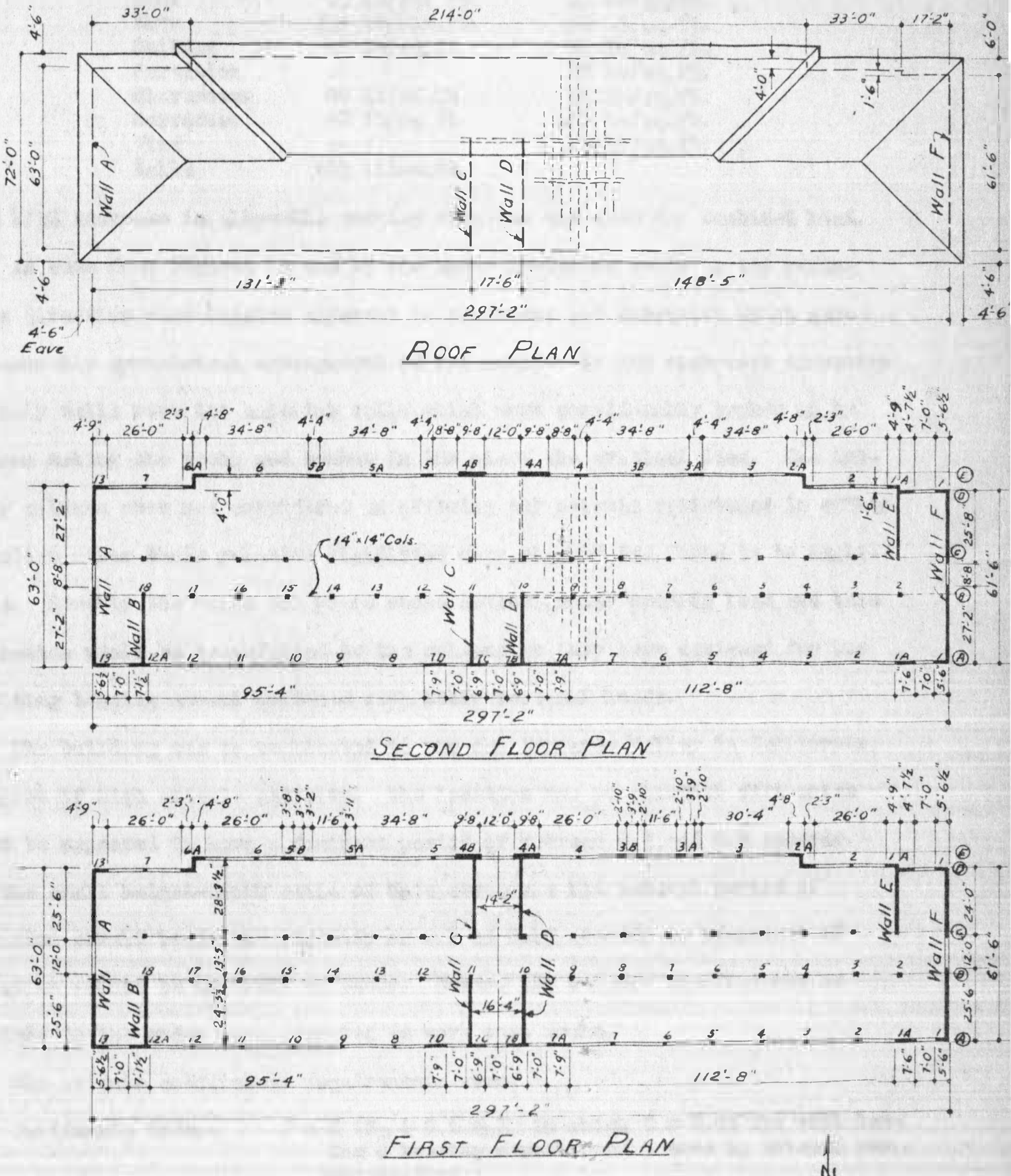

ROOF PLAN
Wall A
Wall C
Wall D
Wall F
Eave
14"x14" Cols.
SECOND FLOOR PLAN
Wall A
Wall B
Wall C
Wall D
Wall E
Wall F
FIRST FLOOR PLAN
Wall A
Wall B
Wall C
Wall D
Wall E
Wall F
N

Figure 15

Loads	Dead load	Live load
Roof	75 lb/sq. ft.	20 lb/sq.ft.
Eave	750 lb/lin.ft.	20 lb/sq.ft.
Ceiling	10 lb/sq.ft.	10 lb/sq.ft.
Partition	--	20 lb/sq.ft.
Classrooms	80 lb/sq.ft.	50 lb/sq.ft.
Corridors	80 lb/sq.ft.	100 lb/sq.ft.
Wind	--	15 lb/sq.ft.
Walls	105 lb/cu.ft.	

A 33 1/3% increase in allowable working stresses was used for combined load.

As seen from Figures 14 and 15 the shear resisting walls in the north-south direction were located adjacent to stairways and entrances which gave a reasonably symmetrical arrangement of the walls. In the east-west direction the only walls were the exterior walls which were considerably broken up by windows making the shear and moment in the piers the critical item. The interior columns were not considered as offering any seismic resistance in either direction after their relative rigidities were checked and found to be negligible. However the walls and piers would deflect under seismic load and this deflection would be transferred to the columns so they were designed for the resulting bending moment combined with their vertical loads.

The building was to be located in the San Fernando Valley in California, a region of high seismic activity. The location was on alluvial fill which might be expected to have a dominant period of between 0.3 and 0.5 seconds. For the small height-depth ratio of this structure its natural period of vibration should be in the vicinity of 0.1 to 0.15 seconds so resonance of vibration would be unlikely to occur. Hence, the seismic coefficients as required in the codes were accepted as more than ample.

The seismic coefficient requirements were:

California Code-- $F = C(W_D + 0.6 W_L)$ in which $C = 0.08$ for soil having a bearing capacity of 3 tons or greater per square foot.

Los Angeles Code-- $F = C\,W_D$

in which C = 0.133 for the roof
0.133 for the second floor
0.109 for the second story
0.093 for the first story

The seismic shears were computed for both California and Los Angeles coefficients and the larger values were used. The total loads on the roof, second floor, and for each wall were first computed as follows:

Roof area--	63' x 297.17'	= 18,700
	4' x 214.0'	= 860
		19,560 sq. ft.
Eaves-	4.5(2x297.17 + 2x4 + 2x63)	= 3,280
	Total	22,840 sq. ft.
Second floor area---	Total	19,560 sq. ft.
Total roof dead load---	19,560 x 85	= 1,660,000
	780 x 728	= 570,000
	Total	2,230,000 lb.
Total roof live load---	22,840 x 30	= 685,000 lb.
Total second floor dead load---	19,560 x 90	= 1,760,000 lb.
Total second floor live load---		
Corridor and classrooms	19560 x 80	= 1,570,000
plus corridor	13 x 297 x 30	= 120,000
	Total	1,690,000 lb.

(Live loads based on 50 + 20 + 10 = 80 in classrooms and 100 + 10 = 110 in corridors)

Second story walls and columns---	Side walls	688,000
	East end wall	98,500
	West end wall	100,500
	Two interior walls	93,000
	34 interior columns	114,000
	Total	1,094,000 lb.

(One-half of this shear contributes to the roof and one-half to the second floor.)

First story walls and columns---	Side walls	828,000
	East end wall	104,000
	West end wall	106,000
	6 interior walls	273,000
	38 interior columns	94,000
	Total	1,405,000 lb.

(One-half of this shear contributes to the second floor and one-half to the first floor.)

The next step was the computation of horizontal seismic shears for the roof and second floor levels as shown on the following page:

North-south direction: total seismic shears.

California Code

Roof   0.08(2230 + ½x688 + ½x114 + .6x685) = 243 kips
2nd Floor   0.08(1760 + ½x688 + ½x114 + ½x828 + ½x94 + .6x1690) = 291 kips
2nd Story   0.08(2230 + ½x688 + ½x114 + .6x685) = 243 kips
1st Story   0.08(2230 + 1760 + .6x685 + .6x1690 + 688 + 114 + ½x828 + ½x94) = <u>525 kips</u>

Los Angeles Code

Roof   0.133(2230 + ½x688 + ½x114) = <u>350 kips</u>
2nd Floor   0.133(1760 + ½x688 + ½x114 + ½x828 + ½x94) = <u>349 kips</u>
2nd Story   0.109(2230 + ½x688 + ½x114) = <u>287 kips</u>
1st Story   0.093(2230 + 1760 + 688 + 114 + ½x828 + ½x94) = 480 kips

Note: Weights of north-south walls are not included in this table because these weights are carried by the walls themselves and should not be proportioned.
For the design of walls the larger, or underlined, values were used.

East-west direction: total seismic shears.

California Code

Roof   0.08(2230 + .6x685 + ½x98.5 + ½x100.5 + ½x93 + ½x114) = 228 kips
2nd Floor   0.08(1760 + .6x1690 + 688 + ½x98.5 + ½x100.5 + ½x93 + ½x114) = 262 kips
2nd Story   0.08(2230 + .6x685 + 688 + ½x98.5 + ½x100.5 + ½x93 + ½x114) = 282 kips
1st Story   0.08(2230 + 1760 + .6x685 + .6x1690 + 1094 + ½x1405 + ½x828) = <u>610 kips</u>

Los Angeles Code

Roof   0.133(2230 + ½x98.5 + ½x100.5 + ½x93 + ½x114) = <u>310 kips</u>
2nd Floor   0.133(1760 + ½x98.5 + ½x100.5 + ½x93 + ½x114 + ½x104 + ½x106.5 + ½x273 + ½x94) = <u>300 kips</u>
2nd Story   0.109(2230 + ½x98.5 + ½x100.5 + ½x93 + ½x114 + 688) = <u>340 kips</u>
1st Story   0.093(2230 + 1760 + 1094 + ½x1405 + ½x828) = 576 kips

Note: For the design of walls the larger, or underlined, values were used.

The total seismic shears on roof and floor levels were then <u>proportioned among the resisting walls as follows:</u>

<u>North-south direction.</u> The roof and second floor diaphragms were considered more flexible than the supporting walls. The seismic force, F, was distributed to the shear walls A, B, C, D, E, & F as follows:

	Proportions		Forces in kips		
Wall	Roof	2nd Floor	Roof	2nd Floor	½ Wall
A	1/3	2/9	61.0	64.7	21.5
B	---	1/9	---	32.3	6.2
C	1/4	1/4	61.0	73.0	20.9
D	1/4	1/4	61.0	73.0	20.9
E	---	1/9	---	32.3	6.2
F	1/3	2/9	61.0	64.7	20.8

Note The shears shown in this table in the "Wall" column are caused by the seismic force on the wall itself.

East-west direction. The roof and the second floor diaphragms were considered infinitely rigid in this direction. Therefore the seismic force was distributed to the side walls in proportion to their relative rigidities except that each wall was designed to take at least one-half of the total shear.
The proportioning of the seismic force and the design of the piers and walls to resist it are so closely inter-related that they will be taken up together in a later paragraph.

Considering the seismic force in the north-south direction the diaphragms and walls were next designed to resist the forces shown on page 49.

The roof diaphragm was subjected to a maximum shear of 350 kips and the thickness of the diaphragm was $2\frac{1}{4}$ inches. The unit shearing stresses are shown in the following table:

At	Total Shear in kips	Length	Cross-sectional Area (sq. in.)	Unit shear stress in lb./sq. in.
Wall A	350/3 = 117	63'0"	1990	62
Wall C	350/4 = 88	38'$2\frac{1}{2}$"	1146	76.7
Wall D	350/4 = 88	38'$2\frac{1}{2}$"	1146	76.7
Wall F	350/3 = 117	61'6"	1845	64.5

The maximum allowable shearing stress in the concrete alone was 1.33x60 = 80 lb. per sq. in. In order to keep the shearing stress in the diaphragm at Walls C and D within the allowable as shown it was necessary to extend the wall across the corridor as shown in the sketch.

The second floor diaphragm, 2 3/4 inches thick, was subjected to a maximum shear of 349 kips. The following shearing stresses resulted:

At	Total Shear in kips	Cross-sectional Area (sq. in.)	Unit shear stress in lb./sq. in.
Walls A & F	349x2/9 = 77.5	1261	61.5
Walls B & E	349x1/9 = 38.8	818	47.4
Walls C & D	349x1/4 = 87.3	2079	42.0

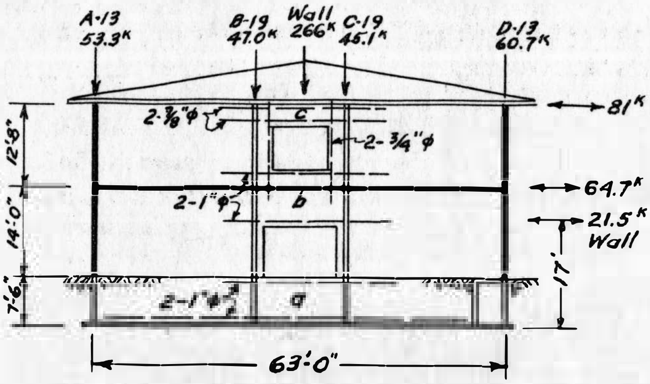
These stresses are also well within the allowable for concrete alone.

The shearing stresses in the walls were a maximum in the first story just above the first floor for the shear of 525 kips. All the walls were 10 inches thick in the first story.

Shearing Stresses in Walls

Wall	Total Shear in kips	Length	Cross-sectional Area in sq. in.	Unit shear stress in lb./sq. in.
A	525x2/9 = 117	51'0"	6120	19.1
B & E	525x1/9 = 58.5	24'9½"	2976	19.7
C & D	525x1/4 = 132	49'7"	5950	22.2
F	525x2/9 = 117	49'6"	5940	19.7

The code requirements for resistance against overturning will be met if the moment of stability of the vertical dead loads only is at least 1½ times the overturning moment and if the maximum combined load soil pressure does not exceed 8,000 lb./sq. ft. Each wall was checked for those conditions but since the calculations are all similar only those for Wall A will be shown here.



Overturning Moment

(Roof)	81x34.17	= 2770
(2nd Fl)	64.7x21.5	= 1390
(Wall)	21.5x17.0	= 365
	M_o	= 4525 ft.-kips

Moment of Stability

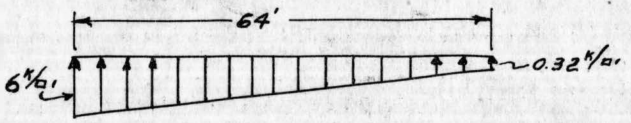
(A13)	53.3x62.0	= 3290
(B19)	47.0x38.2	= 1840
(C19)	45.1x23.8	= 1075
(Wall)	266.0x31.0	= 8250
	M_s	= 14455 ft.-kips

Factor of Safety = 14455/4525 = 3.2 which is greater than 1.5.

Wall A

Soil pressure. M = 4525 ft.kips. P = 266 + 53 + 47 + 45.1 = 471.1 kips.

$$p = \frac{471.1}{2.33x64} \pm \frac{4525x6}{2.33x(64)^2} = 3.16 \pm 2.84 = 6.00 \text{ or } 0.32 \text{ kips/sq. ft.}$$



In combination with live load the maximum combined load soil pressure was 6.44 kips/sq. ft.

The unbalanced soil pressure caused shears and moments in the wall beams a, b, and c. The moments and shears were computed and additional reinforcement was addet in the walls as shown in the above sketch.

For seismic forces in the east-west direction the forces were proportioned between the two longitudinal walls on the basis of relative rigidities. These relative rigidities depend upon the total deflections of the two walls which were calculated using the methods shown in Part II and which are shown in tabular form on the following pages. The first ten columns of the tables give the information from which the relative rigidities of the walls were calculated.

1 Pier	2 Width (")	3 Thickness (")	4 Area (sq.")	5 I (in.)^4	6 H (")	7 Δ_m $H^3/9I$	8 Δ_s H/A	9 Δ (")	10 $\frac{1}{\Delta}$	11 Prop. of Shear	12 Shear (k)	13 v = V/A #/(")^2	14 M = HV Ft-k.
				NORTH WALL---SECOND STORY									
D-1	66.5	12	798	294,080	66	0.108	0.083	0.191	5.23	0.093	15.9	20.0	87.5
D-1A	112.5	12	1350	1,430,000	98	0.073	0.073	0.146	6.85	0.122	20.9	15.5	171.0
D-2A	27.0	12	324	19,683	98	5.310	0.302	5.612	0.18	0.003	0.5	1.5	4.1
E-2A	56.0	12	672	175,616	90	0.461	0.134	0.595	1.68	0.030	5.1	7.6	38.3
E-3A	52.0	12	624	140,608	90	0.576	0.144	0.720	1.39	0.025	4.3	6.9	33.0
E-4	52.0	12	624	140,608	90	0.576	0.144	0.720	1.39	0.025	4.3	6.9	33.0
E-4A	116.0	23.5	2720	3,055,000	90	0.027	0.033	0.060	16.65	0.297	50.8	18.7	381.0
E-4B	116.0	23.5	2720	3,055,000	90	0.027	0.033	0.060	16.65	0.297	50.8	18.7	381.0
E-5	52.0	12	624	140,608	90	0.576	0.144	0.720	1.39	0.025	4.3	6.9	33.0
E-5B	52.0	12	624	140,608	90	0.576	0.144	0.720	1.39	0.025	4.3	6.9	33.0
E-6A	56.0	12	672	175,616	90	0.461	0.134	0.595	1.68	0.030	5.1	7.6	38.3
D-6A	27.0	12	324	19,683	98	5.310	0.302	5.612	0.180	0.003	0.5	1.5	4.1
D-13	57.0	12	684	185,193	98	0.564	0.143	0.707	1.41	0.025	4.3	6.9	35.1
									56.07				
				NORTH WALL---FIRST STORY		$(H^3/36I)$							(HV/2)
D-1	66.5	12	798	294,080	66	0.027	0.083	0.110	9.10	0.076	23.3	29.2	64.0
D-1A	112.5	12	1350	1,430,000	98	0.018	0.073	0.091	11.00	0.092	28.2	21.0	115.0
D-2A	27.0	12	324	19,683	98	1.330	0.302	1.632	0.61	0.005	1.5	5.0	6.1
E-2A	56.0	12	672	175,616	98	0.149	0.146	0.395	2.53	0.021	6.5	9.7	26.5
E-3A	34.0	12	408	39,304	60	0.153	0.147	0.300	3.33	0.028	8.6	21.0	21.5
	34.0	12	408	39,304	46	0.069	0.113	0.182	5.50	0.046	14.1	34.6	27.0
E-3B	34.0	12	408	39,304	46	0.069	0.112	0.182	5.50	0.046	14.1	34.6	27.0
	34.0	12	408	39,304	60	0.153	0.147	0.300	3.33	0.028	8.6	21.0	21.5
E-4A	116.0	23.5	2720	3,055,000	98	0.0086	0.036	0.045	22.20	0.187	57.4	21.1	234.0
E-4B	116.0	23.5	2720	3,055,000	90	0.0066	0.033	0.040	25.00	0.210	64.5	23.7	242.0
E-5A	47.0	12	564	103,823	46	0.026	0.082	0.108	9.25	0.078	23.9	42.5	45.7
E-5B	38.0	12	456	54,872	46	0.049	0.101	0.150	6.67	0.056	17.2	37.7	33.0
	44.0	12	528	85,184	52	0.046	0.098	0.144	6.95	0.058	17.8	33.7	38.5
E-6A	56.0	12	672	175,616	90	0.115	0.134	0.249	4.02	0.034	10.4	15.5	39.0
D-6A	27.0	12	324	19,683	98	1.330	0.302	1.632	0.61	0.005	1.5	5.0	6.1
D-13	57.0	12	684	185,193	98	0.141	0.143	0.284	3.52	0.030	9.2	13.5	37.6
									119.12				

Note: Deflections for P = 1,000,000. Stresses for P = 307,000 lb.

1 Pier	2 Width (")	3 Thickness (")	4 Area $(")^2$	5 I $(in.)^4$	6 H (")	7 Δ_m $H^3/9I$	8 Δ_s H/A	9 Δ (")	10 $\frac{1}{\Delta}$	11 Prop. of Shear	12 Shear (k)	13 $v = V/A$ $\#/(")^2$	14 $M = HV$ Ft.-k.
						SOUTH WALL---SECOND STORY							
A-1	66	12	792	287,500	89	0.272	0.112	0.384	2.60	0.047	8.3	10.5	61.5
A-1A	84	12	1008	592,700	89	0.132	0.088	0.220	4.55	0.080	14.2	14.1	105.0
A-7A	93	12	1116	804,400	66	0.040	0.059	0.099	10.10	0.177	31.4	28.1	173.0
A-7B	81	12	972	531,400	66	0.060	0.068	0.128	7.81	0.137	24.2	25.0	133.0
A-7C	81	12	972	531,400	66	0.060	0.068	0.128	7.81	0.137	24.2	25.0	133.0
A-7D	93	12	1116	804,400	66	0.040	0.059	0.099	10.10	0.177	31.4	28.1	173.0
A-12A	85.5	12	1026	625,000	66	0.051	0.064	0.115	8.70	0.153	27.1	26.4	149.0
A-13	66.5	12	798	295,000	66	0.108	0.083	0.191	5.23	0.092	16.3	20.4	89.6
									56.90				
						SOUTH WALL---FIRST STORY							
						$(H^3/36I)$							(HV/2)
A-1	66	12	792	287,500	89	0.068	0.112	0.180	5.55	0.035	11.1	14.0	41.1
A-1A	84	12	1008	592,700	89	0.033	0.088	0.121	8.25	0.050	15.9	15.8	59.0
A-7A	93	12	1116	804,400	90	0.025	0.081	0.106	9.44	0.057	18.1	16.2	67.1
A-7B	81	12	972	531,400	27	0.001	0.028	0.029	34.50	0.207	65.8	67.8	74.0
A-7C	81	12	972	531,400	27	0.001	0.028	0.029	34.50	0.207	65.8	67.8	74.0
A-7D	93	12	1116	804,400	90	0.025	0.081	0.106	9.44	0.057	18.1	16.2	67.1
A-12A	85.5	12	1026	625,000	27	0.001	0.026	0.027	37.10	0.221	70.3	68.5	79.0
A-13	66.5	12	798	295,000	27	0.002	0.034	0.036	27.80	0.166	52.8	66.3	59.4
									166.58				

Note: Deflections for P = 1,000,000. Stresses for P = 177 kips in second story and 318 kips in first story.

Columns 1-6 give the physical properties of the piers. From Part II, page 39. $\Delta = \Delta_m + \Delta_s$. For the east-west walls the piers of the second story are considered to act as cantilevers, hence $\Delta = \Delta_m + \Delta_s = \frac{PH^3}{3EI} + \frac{6}{5}\frac{PH}{AE_s}$. For $P = 1{,}000{,}000$ lb., $E = 3{,}000{,}000$ lb./sq.in. and $E_s = 0.4E = 1{,}200{,}000$ lb./sq.in., this formula reduces to $\Delta = H^3/9I + H/A$. The piers of the first story are considered fixed at both ends, hence $\Delta = \frac{PH^3}{12EI} + \frac{6}{5}\frac{PH}{AE_s}$ which reduces to $\Delta = H^3/36I + H/A$. Columns 7, 8, and 9 show the application of these formulas to the respective piers. Column 10 shows the relative rigidity of each pier.

Using the formulas shown on page 40 the deflection of the <u>north</u> wall becomes--

$$\Delta = \Delta_{\text{roof beam}} + \Delta_{\text{2nd Fl. bm.}} + \Delta_{\text{1st Fl. bm.}} + \frac{1}{\sum 1/\Delta_{\text{2nd story piers}}} + \frac{1}{\sum 1/\Delta_{\text{1st story piers}}}.$$

$$\Delta_{\text{roof beam}} = \frac{(3.17)^3 x12}{36x12(297.17)^3} + \frac{3.17}{297.17x12} = 0.0009 \text{ inches}$$

$$\Delta_{\text{2nd Fl. bm.}} = \frac{(6.5)^3 x12}{36x12(285.0)^3} + \frac{6.5}{285.0x12} = 0.0019 \text{ inches}$$

$$\Delta_{\text{1st Fl. bm.}} = \frac{1}{2x\left(\frac{(3)^3 x12}{(134)^3 x12x36} + \frac{3}{134x12}\right)} = 0.00093 \text{ inches}$$

$\sum 1/\Delta$ (from Column 10, page 52) is 56.07 for the second story and 119.12 for the first story. Then--

$$\Delta = 0.0009 + 0.0019 + 0.00093 + \frac{1}{56.07} + \frac{1}{119.12} = 0.03 \text{ inches}$$

Similarly for the <u>south</u> wall--

$$\Delta = 0.0009 + 0.0019 + 0.00093 + \frac{1}{56.90} + \frac{1}{166.58} = 0.027 \text{ inches}$$

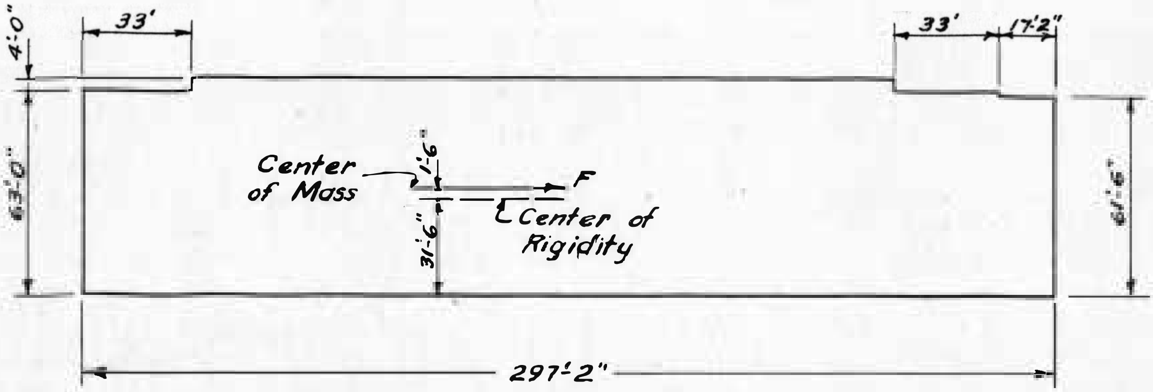
Then the relative rigidities are:

	Δ	$1/\Delta$	Relative Rigidity
North wall	0.0300"	33.33	0.476
South wall	0.0273	36.60	0.524

<u>Investigation of torsional moment</u>. Since the two walls do not have equal rigidities there would be additional stresses due to rotation. The center of mass was 33'0" from the outside face of the south wall. The center of rigidity,

y feet from the south wall was—

$1\,y = 0.476(63 + 0.72x4) = 31.5$ ft. from the outside face of the south wall. Therefore the eccentricity was $33.0 - 31.5 = 1.5$ ft. and the rotational moment was equal to $1.5\,F$.



Since there are only two resisting walls the rotational shear in each must be equal in magnitude. Taking moments about the center of rigidity:

$$31.5\,P + (31.5 + 0.72x4)P = 1.5\,F$$

$$P = \pm 0.23\,F$$

Then the final shears in each wall become:

	Second Story F = 340 kips	First Story F = 610 kips
North Wall	0.476x340 + 0.023x340 = 171 kips	0.476x610 + 0.023x610 = 307 kips
South Wall	0.524x340 = 177 kips	0.524x610 = 318 kips

These shears were then proportioned to the piers as shown in Columns 11 and 12, pages 52 and 53. The shearing stresses are shown in Column 13 and the moments in Column 14. The piers were then designed for combined axial and bending stresses, the work not being shown here.

The fundamental period of the building was checked approximately using the formula shown on page 43 (Pregnoff) and determined to be 0.17 second.

$$T = 1.1\sqrt{\frac{2.23(.03)^2 + 1.76(.0093)^2}{2.23x.03 + 1.76x.0093}} = 0.17$$

APPENDIX I

MODIFIED MERCALLI INTENSITY SCALE OF 1931

I. Not felt except by a very few under especially foverable circumstances.

II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.

III. Felt quite noticeably indoors, expecially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing of truck. Duration estimated.

IV. During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls made creaking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.

V. Felt by nearly everyone; many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop.

VI. Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.

VII. Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars.

VIII. Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimney, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Disturbed persons driving motor cars.

IX. Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.

X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Land slides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.

XI. Few, if any (masonry) structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipe lines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.

XII. Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into the air.

Bibliography

(1) "Earthquake Investigations in California, 1934-35", U. S. Department of Commerce, Special Publication No. 201.

(2) "Dynamics of Earthquake Resistant Structures" by Jacob J. Creskoff. Published by McGraw Hill Book Co.

(3) "Analysis of Small Reinforced Concrete Buildings for Earthquake Forces", Portland Cement Association.

(4) "Earthquake Resisting Structures" by Edward R. Dye. Bull. No. I of Montana State College Engineering Experiment Station.

(5) "Uniform Building Code", (1946), Pacific Coast Building Officials Conference.

(6) Los Angeles, Calif. Building Code, 1947-48

(7) State of California Administrative Code, Title 21.

(8) "Lateral Force Code", Joint Committee of the San Francisco Section, A. S. C. E., and Structural Engineers Association of Northern California.

(9) Engineering News Record, May 21, 1942, "Vibration Data" by H. M. Engle.

(10) Building Regulations for Reinforced Concrete, 1941, of the American Concrete Institute.

(11) Recommendations of the Board of Fire Underwriters of the Pacific for Earthquake Resistant Design.