WATERFRONT STRUCTURES
IN
EARTHQUAKE AREAS

by

L. D. Lawson

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PRINCETON UNIVERSITY

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A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in Engineering from Princeton University, 1962.

ACKNOWLEDGEMENTS

The author desires to express his sincere appreciation to the following:

Professor Gregory P. Tschebotarioff, Department ot Civil Engineering, Princeton University, who provided guidance and advice during the preparation of this work.

The Bureau of Yards and Docks, Department of the Navy, for the opportunity to undertake this year of graduate study in the field of Waterfront Engineering.

Professor Norman J. Sollenberger, Chairman of the Department of Civil Engineering, Princeton University, who encouraged the author to study this thesis subject.

Professor J. Martin Duke, Department of Civil Engineering, University of California, Los Angeles; Mr. James R. Ayers, Waterfront Consultant, Bureau of Yards and Docks, Department of the Navy; and Doctor Yasunaru Ishii, Soil Mechanics Division, Research Institute of Transportation Technology, Ministry of Transportation, Government of Japan, who provided the author important reference material.

My wife, who not only provided great understanding and moral support, but also typed the initial and final draft of this thesis

PREFACE

Engineering has benefited from a great increase in study of the effects of earthquakes on civil engineering structures over the past 15 years. The vast majority of this work concerns tall superstructures such as multistoried buildings, water towers, and industrial smoke stacks. Dams and bridges have also recieved considerable attention. There has been, however, a paucity of information regarding waterfront structures subjected to earthquake conditions. This is inconsistent with both the great financial investment required for waterfront structures and the fact that they are adversely located in areas of poor seismic foundation conditions, abrupt geological transition, and greatest seismicity. The purpose of this thesis is to setforth the most significant experimental and field data relating to earthquake phenomenon in the waterfront enviorment and to recommend methods of modern practice for design and analysis of waterfront structures subjected to earthquake loads.

To make this thesis a self sufficient document for use in waterfront engineering, earthquakes are discussed in general, field observations of earthquake damage are presented, and seismic properties of soil are discussed both as affected by local geology and in regard to engineering properties. Recommendations for waterfront engineering practice are then presented. Tsunami, or seismic sea waves, are also discussed.

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Chapter 1

EARTHQUAKES

1.01 Introduction

An earthquake is a natural phenomenon characterized by oscillation or vibration of the surface of the earth with violence of varying intensity. Their occurrence has been recorded throughout the history of man. Every part of the world has at some time been affected by an earthquake. Their intensity varies from the most violent lurching of the ground causing destruction of structures, land slides, earth ruptures, fires, and tidal waves to tremors which can be recorded only by sensitive instruments.

The engineer is interested in large earthquakes causing loss of life, depletion of economic resources, and extended disruption of normal activity. It is his responsibility to provide structures and works which will withstand the violence of these natural disasters. Through the understanding of the character and magnitude of the force unleashed by an earthquake, the engineer can carry out his grave responsibility to society.

1.02 Origin and Cause

An earthquake is a transient occurrence of vibration, or oscillation, of the earth's surface caused by a disturbance of the gravitational equilibrium of rock somewhere within the outer layer of the earth's crust. The location from

which the disturbance originates is known as the focus of the earthquake. The energy released at the focus is radiated outward by various types of elastic waves. These waves travel through and along the surface of the earth's crust in all directions with varying degrees of intensity. The pattern of surface vibration and oscillation is varied by refraction, reflection, and superposition of one type of elastic wave on another. The energy level, or intensity, of these waves diminishes with distance. The epicenter of an earthquake is the vertical projection of the focus on the surface of the earth.

There are several types of earthquakes. They are classified as tectonic, plutonic, and volcanic according to the source of the stresses at their focus, which are released as energy waves. Rock quarry blasting and high explosive bombs cause ground vibrations which are sometimes considered as man made earthquakes due to their similarity of earth motion.

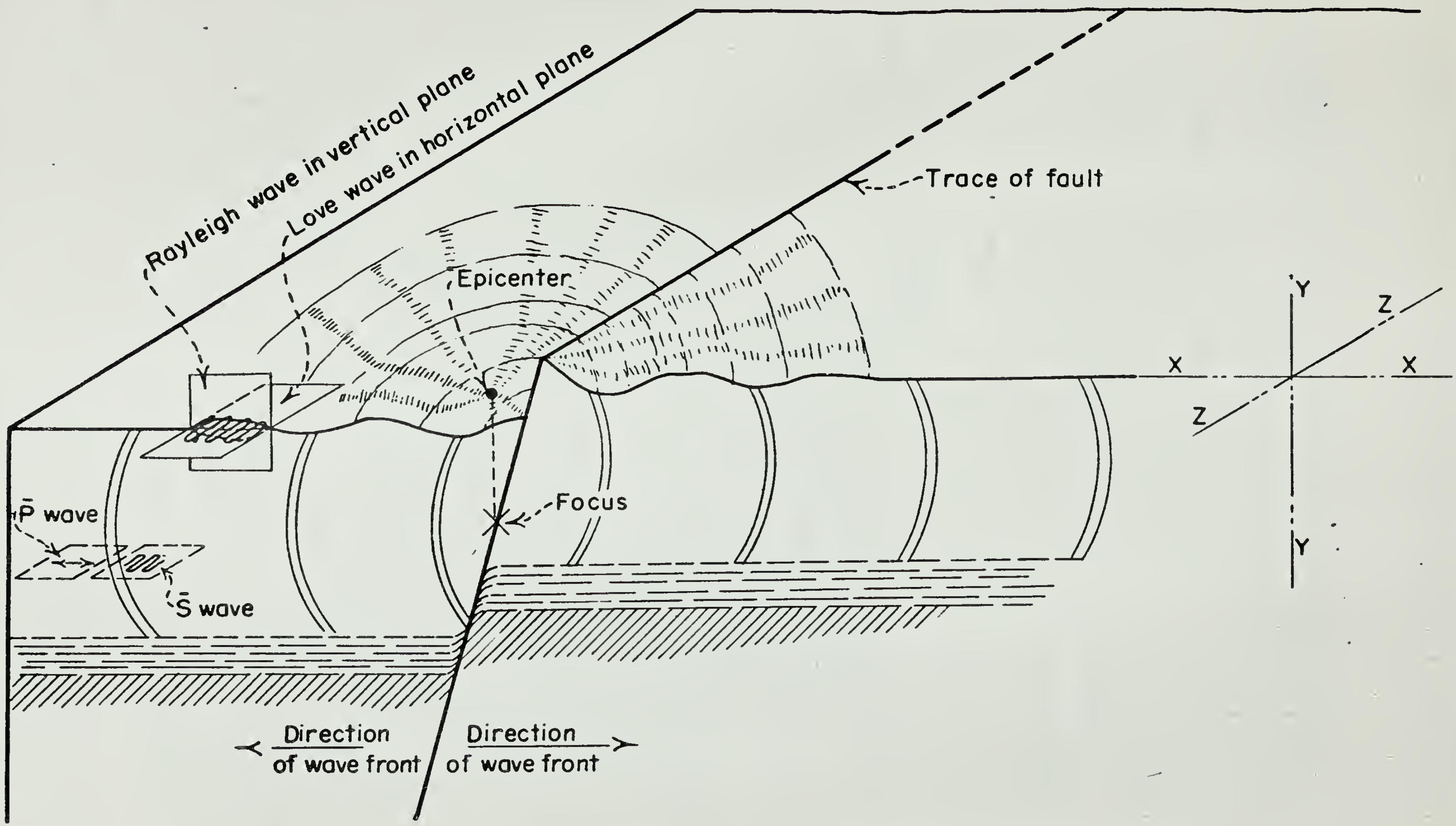
Tectonic earthquakes result from a sudden rupture of the earth's crust by either faulting or warping. They are clearly structural in nature and have an intimate relationship with the development of the structure of the earth's crust. Tectonic earthquakes are the most prevalent, and are the source of the greatest and most widespread damage. The tremendous energy which they release has been accumulating through centuries as elastic strain resulting from the slow distortion of the earth's crust. The strained formation of rock slowly

reaches its ultimate capacity and ruptures, or fails, violently. The various energy waves which cause the surface vibrations are set up, or energy of elastic strain is erratically released, by the grating of the two rock surfaces of rupture. See Plate I. Tectonic earthquake are further classified according to the depth of their focus wherein there are shallow earthquakes with foci at depths less than 70 km, intermediate earthquakes with foci between 70 and 300 km, and deep earthquakes with foci between 300 and 700 km. Shallow rupture, or faulting, causes the greatest surface vibrations and damage for total energy released as less energy is dissipated by the distance the waves must travel. Rarely does the movement along a fault extend to the surface. (37)

Deep focus motion within the earth's crust is sometimes referred to as a plutonic earthquake. The causes of these deep focus earthquakes are not clear although none have been recorded whose focal depth exceeded 700 km.

Volcanic earthquakes are caused by gas explosions at erupting volcano vents. (1) They are usually not of great intensity, and in most cases their energy is dissipated directly into the atmosphere. In some areas, such as the Hawaiian Islands, tectonic and volcanic activity occurs in the same geographical area. It then becomes difficult to determine which is the exact cause of the earthquake.

Earthquakes are invariably followed by many, up to 1000 plus, aftershocks. These are minor earthquakes in themselves



SCHEMATIC DIAGRAM OF EARTHQUAKE WAVES

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FIGURE 11

(FROM TVA, REFERENCE 34)

PLATE I

due to the readjustment of the earth's mantle necessitated by the major earthquake. They may be of considerable magnitude, and have been known to be larger than the earlier large scale earthquake. (25) Aftershocks are particularly dangerous as they occur during restoration work causing failures and slides which were incipient from the original shocks and secondary effects damage.

1.03 Seismicity of the Earth

The seismicity of the earth refers to earthquake occurrence characteristics by geographical region. There are parts of the earth that are very active seismically, or unstable, while other parts are not frequented by earthquakes. These are known as stable areas. It is difficult to determine the seismicity of any region because of the element of time. The time between occurrences of great earthquakes in some regions may exceed historical records. The relative accuracy of seismicity data varies greatly from region to region as parts of the world, such as Europe and China, have historical records dating back 2000 to 4000 years. Other parts of the world are known only through instrumental records of 50 years or less. Regardless of the time factor and lack of homogenity of records, the best guide to the seismicity of a region is its history of past earthquakes.

The surface of the earth consists of relatively inactive, or stable, blocks seperated by four groups of active, or unstable, seismic zones. These zones are relatively narrow

belts in which occur the great majority of earthquakes. See Plate II. Little is known about the relationship between earthquakes within a seismic zone except for aftershocks which are clearly individual earthquakes, usually small, which are triggered by the strong earthquake which they follow. The four groups of active seismic zones are: (37)

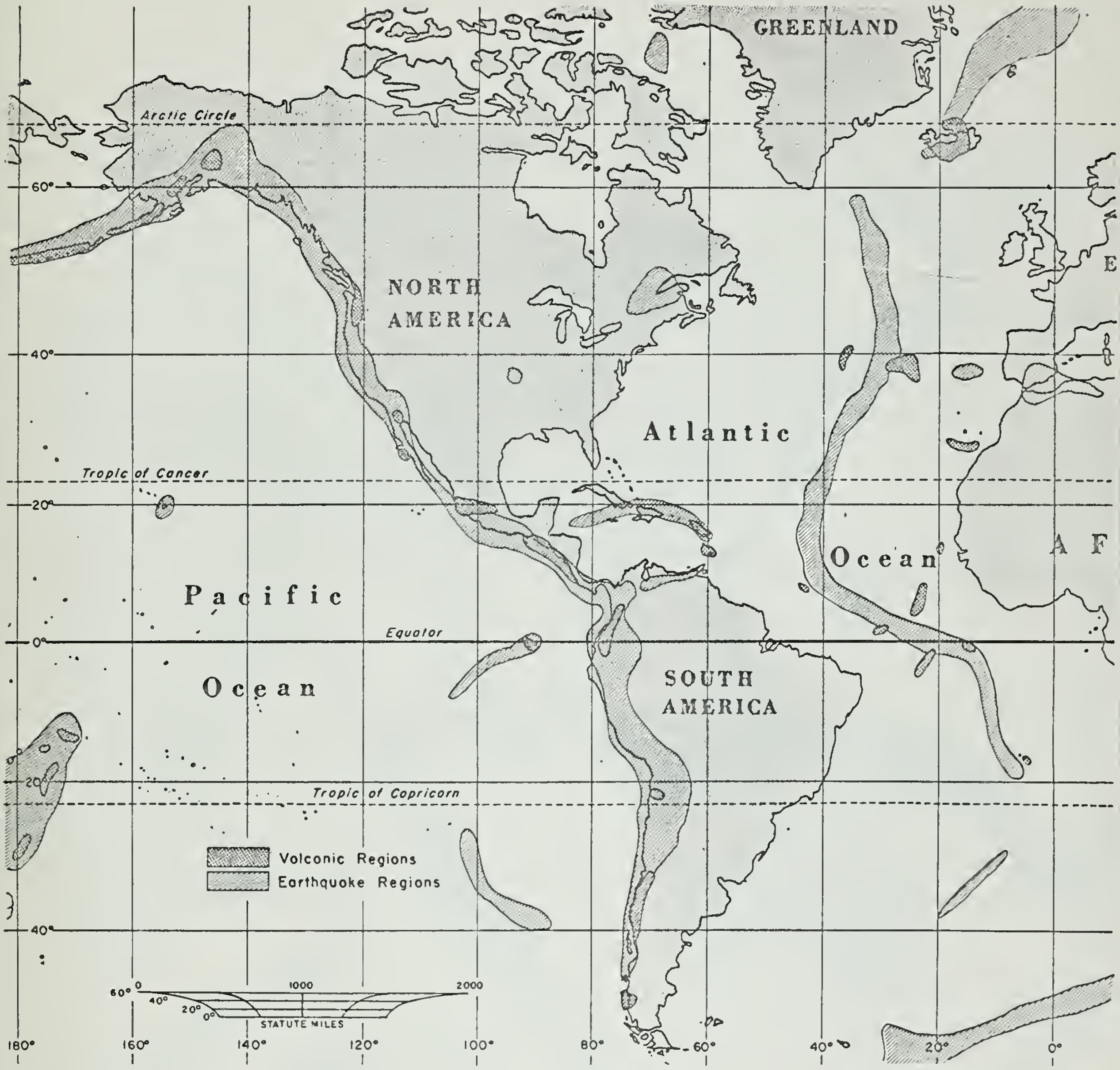
a. The circum-Pacific zone-includes about 80% of shallow shocks, 90% of intermediate shocks, and all deep shocks. Shallow seismicity is highest in Japan, western Mexico, Melanesia, and the Philippines. South America has an exceptionally large share of great shocks. South of Japan the zone divides into two branches, and south of Mexico it again divides. One branch follows the Andes Mountain range and the other passes through the Easter Islands.

b. The Mediteranean and trans-Asiatic zone- includes nearly all remaining intermediate and large shallow shocks. The epicenters follow mountain chains.

c. The principle ridges in the Atlantic, Artic, and Indian oceans- made up of three narrow belts of shallow shocks.

d. Rift structures such as those of East Africa and the Hawaiian Islands- moderate activity only.

It is generally accepted that most earthquakes are a manifestation of the energy release of faulting, and that



-FIGURE 14-
Earthquake Areas in the World – Western Hemisphere
(FROM RUDOCKS. REFERENCE 68)

PLATE II ½

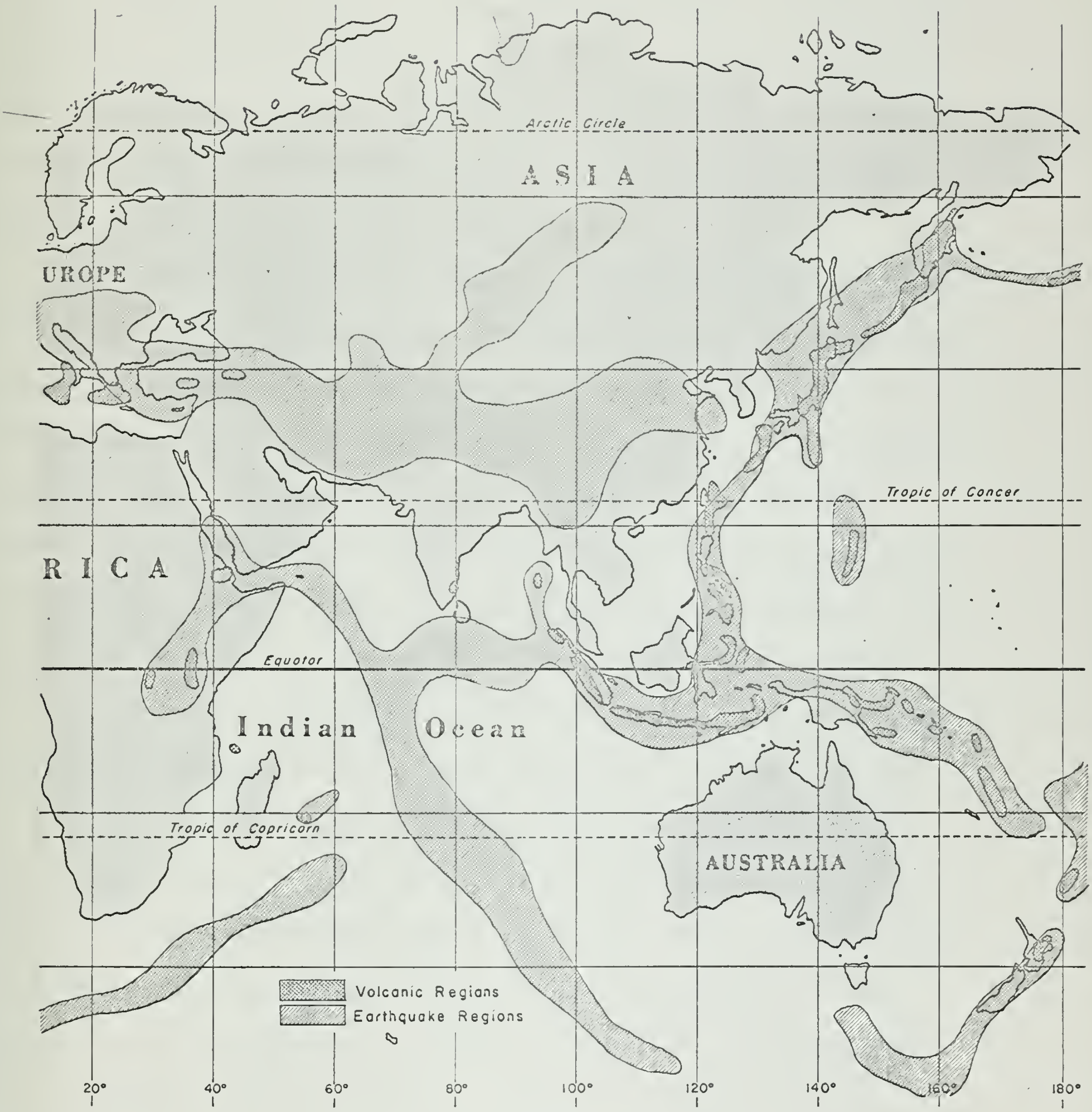


FIGURE 15
Earthquake Areas in the World – Eastern Hemisphere
(FROM BUDOCKS, REFERENCE 68)

PLATE II 2/2

faulting is a mobility mechanism of the earth's crust to release strain induced by thermal contraction. As heat from the earth is dissipated into outer space, the volume of the earth is reduced and the relatively rigid crust must accommodate itself by an infinite series of slow elastic stress accummulations and ultimate ruptures.

The fact that belts of high seismicity generally follow coastal mountain ranges might be explained by the following speculative reasoning. The body of the earth consists of a molten liquid of great density upon which floats the lighter and more rigid masses of rock which make up the earth's crust. High mountains displace more molten liquid and their base extends deeper into the core than lighter masses. This condition is offset, according to theory, by the heavy basaltic rock which underlies oceans and planes, whereas high mountains are formed by lighter rock. As the load balance is altered, such as by a reduction in weight of mountains due to erosion or melting of glaciers, the reduced load on the rock mass cause that region to slowly rise. Because of the nature of load change and the viscoplasticity of the earth's core, adjustments between adjacent floating masses may be gradual over centuries. Instead of free movement between masses, the crust becomes interlocked and local strains accumulate until a rupture finally occurs. The greatest load differential exists where tall mountains are bounded by deep ocean throghs. This is the characteristic

of the entire length of the circum-Pacific zone, and this accounts for the greater seismicity, or earth crust activity, in these regions. The relative probability of earthquakes by geographical area in the United States is illustrated by Plate III.

It is of special significance to waterfront engineers that many of the world's harbors are located in these areas of highest seismicity.

1.04 Vibrations and Propogation

Ground movement during an earthquake is neither simple nor harmonic, but is violent, chaotic, and complex. It occurs concurrently in the three axes of direction accompanied by rotational movements. The intensity of vibration is not uniform. It rises to terrifying peaks of movement, where it becomes difficult or impossible to remain standing, then it almost dies away only to return to increase in tempo. The duration of major earthquakes ranges from less than 30 seconds up to about 4 minutes. (60) Various types of energy waves traveling at varying velocities over differing routes arrive at a point at the same instant and their effects are superimposed. Seismologist, nevertheless, analyse the records of seismographs and accelerographs, and reduce the irregular and complex oscillations to relatively simple trains of vibration waves. These, in turn, are analysed mathematically according to physical laws and material properties to provide period,

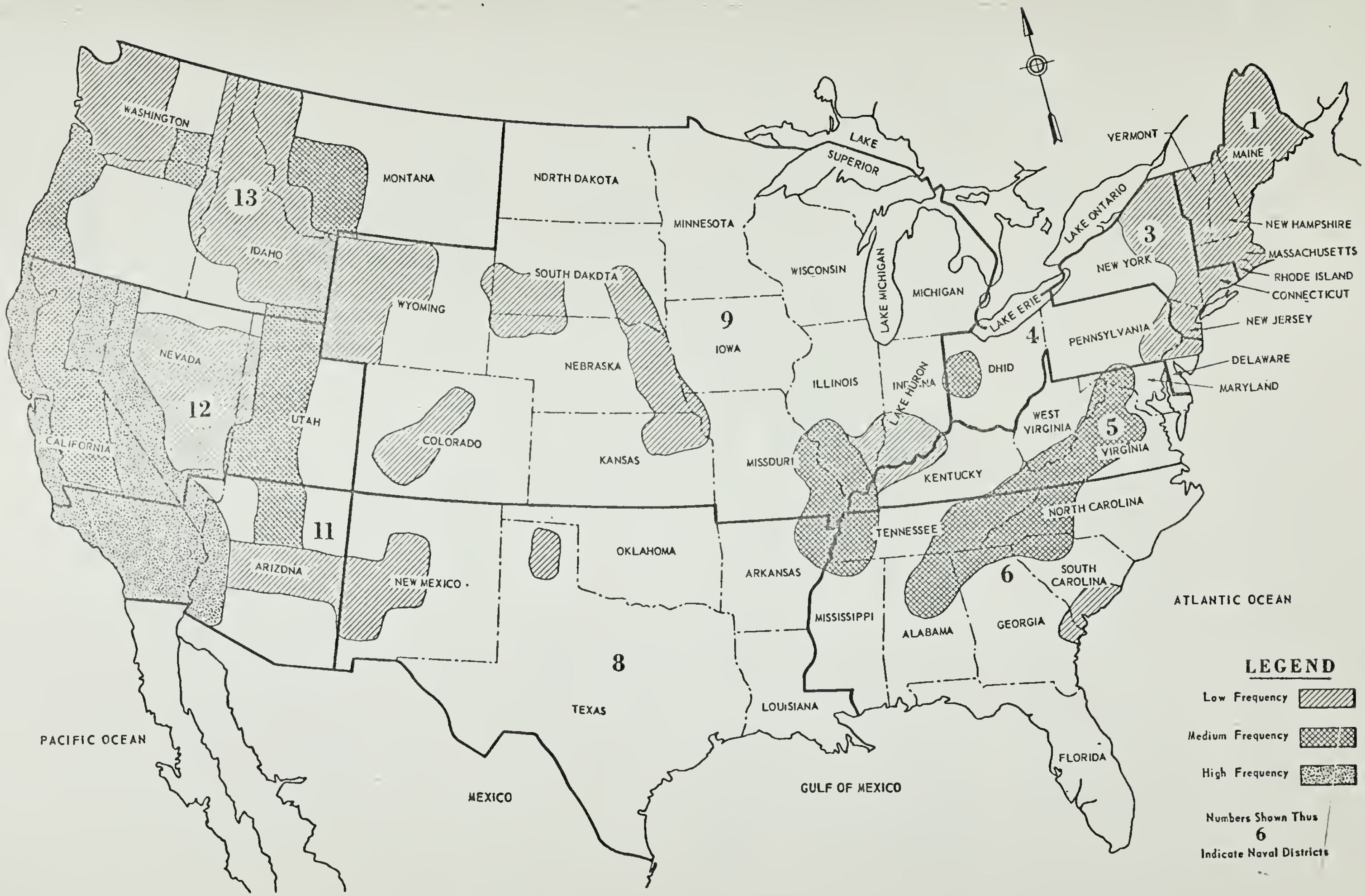


FIGURE 13
Earthquake Areas in the United States
(FROM BUDOCKS, REFERENCE 68)

PLATE III

amplitude, velocity, and acceleration which correspond reasonably well to experience.

Ground vibrations are analysed as elastic oscillation following the laws of simple harmonic motion. This motion has geometric relationships according to its definition, which states:

> "simple harmonic motion is the motion of the projection, on a straight line in the plane of a circle, of a point traveling with constant speed in that circle." (55)

The oscillation follows a sinusoidal pattern of movement. See Plate IV. The relationship of its principle parameters is expressed by:

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

where

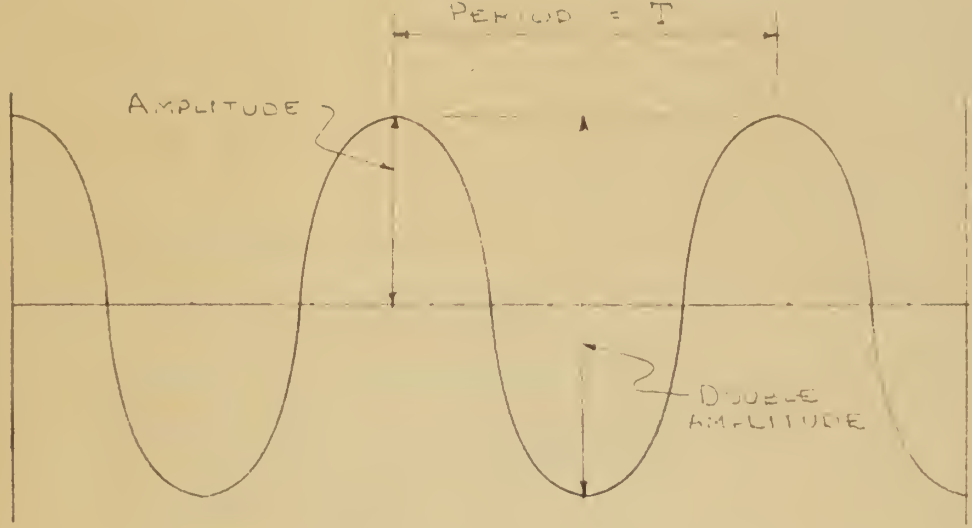
a - acceleration of the particle

A - amplitude

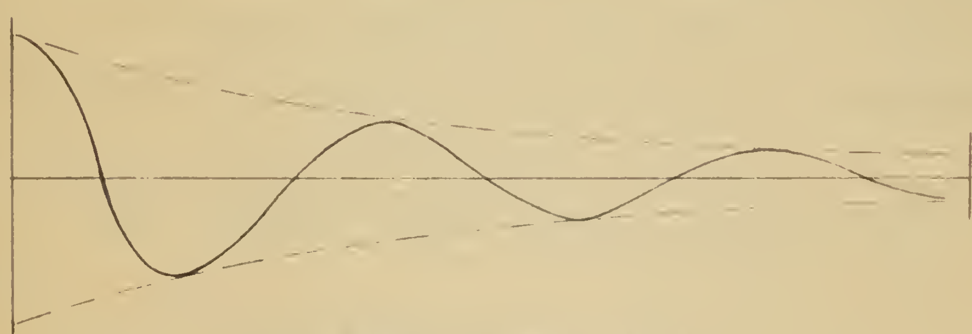
T - period

The sense of acceleration in harmonic motion reverses with each half cycle, reaching its maximum value at the extremes of amplitude where the character of vibrational motion reverses. The maximum earthquake force occurs at maximum acceleration according to Newton's second law of motion, as expressed by:

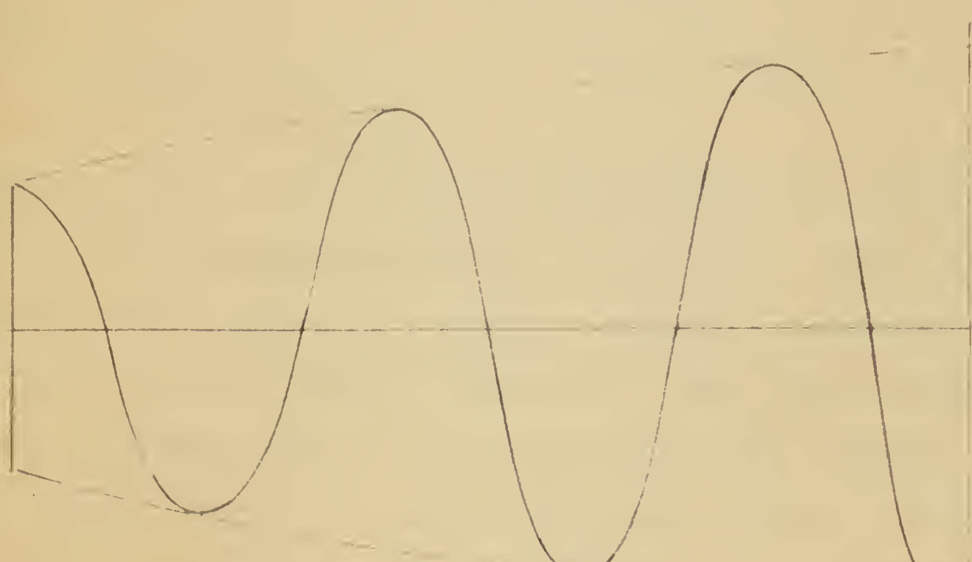
$$F = Ma = \frac{Wa}{g} = W\frac{a}{g} = WC$$



SIMPLE HARMONIC MOTION, STEADY VIBRATION



DAMPED VIBRATION



RESONANCE IN FORCED VIBRATION

PLATE IV

where

F - earthquake force

M - mass of vibrating body

W - weight of vibrating body

g - acceleration due to gravity

C - seismic factor - $\frac{a}{g}$

The seismic factor is utilized in earthquake engineering as an indicator of the magnitude of force, or destructive power, unleashed by earthquake vibration. The weight of a vibrating body is multiplied by seismic factor to provide a quantitative expression of the force on that body.

The kinetic energy of simple harmonic motion is expressed by:

$$\text{Kinetic energy} = \frac{Mv^2}{2} = \frac{W}{2g} \times \frac{a^2}{4\pi^2 n^2}$$

where

n - frequency = $\frac{1}{T}$

This indicates that the energy, and consequently the destructive power, of earthquake vibration is not only a function of the square of the acceleration, but is also a function of the reciprical of the frequency squared. This latter factor explains the great variances in destructive power between shocks, such as destructive earthquakes and harmless blasting, registering the same accelerations.

Ground vibration is essentially free vibration wherein an elastic mass is subjected to a shock, or a sudden force,

and simple harmonic vibration is set up. This vibration gradually dies out due to the damping effect of internal friction and other causes. Masses, or structures, which are in contact with a vibrating mass undergo forced vibration. The characteristics of forced vibration depend entirely on the amplitude, acceleration, and period of the externally applied vibration from which it is generated. Accordingly, much of the vibration of an earthquake is forced vibration, particularly that motion of a structure induced by the vibration of its foundation.

The attenuation of the free vibration of an elastic body to gradually bring the body to rest is known as damping. Frictional resistance which dissipates the energy of vibration as heat and inefficient connections between elements are the principal sources of damping. Partial damage, or deformation, of a structure during an earthquake also absorbs destructive energy. Plate IV illustrates dampened vibrations.

Resonance occurs when the frequency of a disturbing vibration approaches the natural frequency of full vibration of an elastic body. This is a form of forced vibration where each cycle of the disturbing vibration magnifies its amplitude. The ratio of increased amplitude to the amplitude of one vibration is given by:

$$\text{Magnification Factor} = \frac{1}{1 - \frac{T^2}{To^2}}$$

where

T_o = period of a disturbing vibration

The damping effect of internal friction and inefficient connections, including that between the foundation and the ground, and the lack of continued uniformity of frequencies during an earthquake prevents unchecked resonance. It is seldom considered in earthquake engineering. (1)

Earthquake waves travel through the earth and along its surface. See Plate I. They have been identified and designated according to their characteristics of movement as follows:

P waves (primae) - These are longitudinal, or compression and rarefaction, waves which travel through the earth's crust. They have the greatest velocity and are therefore, the first to arrive at an instrumented site. The average velocity of P waves in bed, or basement, rock is 4.5 miles per second, and their period ranges from 1/2 to 6 seconds. P waves die off with the reciprocal of distance.

S waves (secondae) - These are transverse, or shear type, waves oscillating at right angles to their path of travel. S waves are the second to arrive at a point and usually have greater amplitudes than P waves. Their average velocity is 2.7 miles per second, and their period ranges from 11 to 13 seconds. They travel through the earth's crust. S waves also die off with the

reciprocal of distance.

PP and SS waves - These are P waves and S waves after one reflection. When P waves reflect as S waves, they are designated as PS waves.

R waves (Rayleigh) - These are elliptical surface waves with their major axis in the direction of propagation. Their average velocity is 2.0 to 2.5 miles per second with periods up to 40 seconds, and their amplitude dies off exponentially with depth. This wave is the most destructive of all the waves propagated by an earthquake.

Q waves (Love) - These are transverse surface waves. They have no vertical component, and die off exponentially with depth.

Others - Seismologists continue to find other waves which are believed to be products of the complicated structure of the earth's crust. They are unimportant to earthquake engineering.

The velocity of longitudinal P waves at shallow depths for principal soil types is as follows: (55)

Material	Velocity (ft/sec)
Sand	650-6,500
Loess	1,000-2,000
Artificial fill	1,000-2,000
Alluvium	1,600-6,500
Loam	2,600-5,900
Clay	3,300-9,200

(Cont.)

Material	Velocity (ft/sec)
Marl	5,900-12,500
Salt	15,000
Sandstone	4,600-14,100
Limestone	5,600-21,000
Slate and shale	7,500-15,400
Granite	13,000-18,700
Nepheline syenite	18,000
Quartzite	20,000
Norite	20,500

Since earthquake waves are generated by the release of strain energy, they are assumed to travel through an elastic media for mathematical expediency. Instrumented earthquake measurements and laboratory experiments produce good verification. The various characteristics of seismic vibration are thereby related in accordance with the theory of elasticity. (34)

1.05 Earthquake Magnitude

Earthquake magnitude is the quantitative measure of energy released by an earthquake shock, or faulting at its source. It is obtained by data collected from several widely separated seismograph stations. The measure of magnitude was originally proposed by Richter and subsequently revised by Gutenburg and Richter. They defined magnitude, for shocks

in California, as the logarithm of the maximum trace of amplitude expressed in thousands of a millimeter which would register on a standard short-period torsion seismometer at a distance of 100 kilometers from the epicenter. The relationship between the magnitude of an earthquake shock and the total energy released is expressed by the equation:

$$E = (10)^{11} x (10)^{1.6M}$$

or

$$\text{Log } E = 11 + 1.6M$$

where

E energy in ergs (1 erg = dyne-cm = 0.738x10 ft-lb)

M Magnitude, the logarithm of the maximum trace amplitude in microns

From the above equation it should be noted that magnitude of an earthquake varies as the logarithm of the energy released. An earthquake of magnitude 7 releases 40 times as much energy as a shock of magnitude 6 and 1600 times as much as a shock of magnitude 5. The surface effect of a shock of any magnitude is intimately related to its depth.

Gutenberg and Richter have cataloged important earthquakes, based on instrumental data, from 1900 to date, in their book "The Seismicity of the Earth". (37) Included are data on epicenter location, depth, and magnitude. Additionally, they thoroughly discuss the relation between energy, acceleration, and magnitude.

1.06 Earthquake Intensity

Since the earliest studies of earthquakes, there have been attempts to divise a scale for measuring the violence of ground motion. Approximately 27 such scales have been proposed. Each expresses the violence of ground motion in terms of the effects on people and things within a particular area. None of the scales provide a completely satisfactory method for evaluating the intensity of earthquake force on structures for engineering purposes.

Earthquake intensity is the violence of ground motion at a particular location, as contrasted to earthquake magnitude, the amount of energy released at its source. The lower intensities are described by the sensations and reactions of people subjected to the earthquake. People may be frightened or awakened from sleep. The behavior of inanimate objects which are easily moved such as rattling dishes, swinging chandeliers, and ringing bells are also utilized. Increased ground motion, or higher intensity, is described by structural damage such as cracking of walls, overthrow of chimneys, and collapse. Panic of crowds and fissures in the earth's crust are also utilized. Intensity scales are subjective measurements. At best they are only approximate.

The most sophisticated scale is the Modified Mercalli scale. It was developed by Mercalli to improve on the Rossi-Forel scale, then adjusted by Concani to correlate sensory perception with dynamic analysis, and finally improved by

Plate V

MODIFIED MERCALLI INTENSITY SCALE OF 1931
(Abridged)

I. Not felt except by a very few under especially favorable circumstances. (I Rossi-Forel Scale)

II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing. (I to II Rossi-Forel Scale)

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

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. (IV to V Rossi-Forel Scale)

V. Felt by nearly everyone; many awakened. Some dishes, windows, etc. broken; a few instances of cracked plaster; unstable objects overturned. Disturbances of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop. (V to VI Rossi-Forel Scale)

VI. Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight. (VI to VII Rossi-Forel Scale)

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 Rossi-Forel Scale)

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 chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motor cars disturbed. (VIII+ to IX Rossi-Forel Scale)

Plate V (Cont.)

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. (IX Rossi-Forel Scale)

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

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.

CORRELATION OF EARTHQUAKE INTENSITY SCALES

ROSSI-FOREL		MERCALLI		OMORI		CANCANI		MERCALLI-CANCANI-SIEBERG		
Description	Grade	Description	Grade	Accel. mm/sec^2	Grade	Accel. mm/sec^2	Grade	Description	Accel. mm/sec^2	Grade
Microseismic	I	Not felt	I			0-2.5	I	Not felt	0-2.5	I
Extremely feeble	II	Extremely feeble	II			2.5-5	II	Very feeble	2.6-5	II
Very feeble	III	Feeble	III			5-10	III	Feeble	6-10	III
Feeble	IV	Medium	IV			10-25	IV	Moderate	11-25	IV
Medium	V									
Strong	VI	Strong	V			25-50	V	Fairly strong	26-50	V
Decidedly strong	VII	Very strong	VI			50-100	VI	Strong	51-100	VI
Very strong	VIII	Strongest	VII			100-250	VII	Very strong	101-250	VII
Extremely strong	IX	Ruinous	VIII	300	I	250-500	VIII	Ruinous	251-500	VIII
				900	II	500 to 1000	IX	Destructive	501 to 1000	IX
Strongest	X	Disastrous	IX	1200	III	1000 to 2500	X	Very destructive	1001 to 2500	X
		Very disastrous	X	2000	IV					
				2500	V					
		Catastrofic	XI	4000	VI	2500 to 5000	XI	Great destruction	2501 to 5000	XI
				4000 or more	VII	5000 to 10000	XII	Total destruction	5001 to 10000	XII

FIGURE 14

(FROM TVA, REFERENCE 34)

PLATE VI

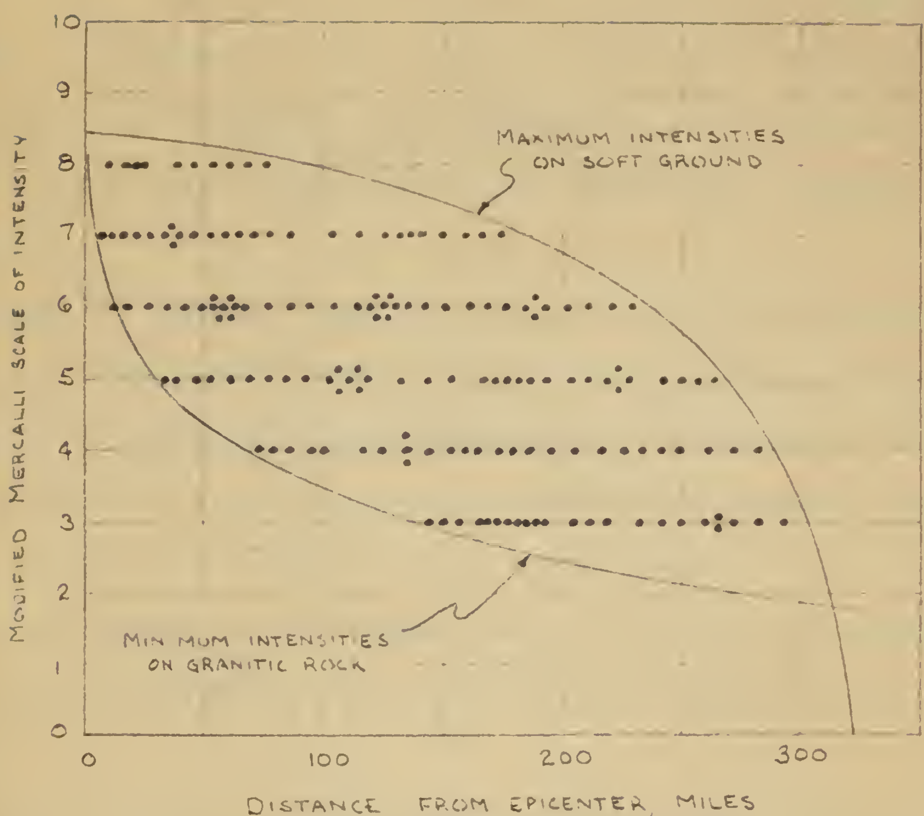
Sieberg. It is also known as the Mercalli-Cancani-Sieberg intensity scale. See Plate V. The data is scientifically collected by area according to the principles of statistical sampling and analysis.

Correlation between the most commonly used earthquake intensity scales is illustrated by Plate VI.

Earthquake intensities determined for various areas may be plotted on a map. Contours, known as isoseismal lines, may then be drawn between areas of equal intensity to produce an isoseismal map. This map gives the engineer a practical description of the effects of an earthquake over a large region, particularly when related to the location of the epicenter.

1.07 Geological Effects

Isoseismal maps indicate that local geological conditions and soil cover profoundly influence the intensity of earthquake ground motion in any particular area. Damage does not diminish uniformly in all directions with distance from the epicenter. When the intensities for a particular earthquake are plotted against distance from the epicenter, an envelope of plots is produced as illustrated by Plate VII. By correlation of the various plots with their geological description, it was determined that the bottom of the envelope was made of locations of granitic outcrop. Plots for greater intensity of motion at any particular distance were found to



INTENSITY-DISTANCE DISTRIBUTION
ENVELOPE

PLATE VII

be for softer ground, ranging up to the highest intensities for water filled alluvium and saturated filled ground. It was also found that when distance was plotted to a logarithmic scale, the bottom of the envelope fell on a straight line. Accordingly, the intensity distribution in the basement rock is very uniform and diminishes as an exponential function of distance.

Noting the narrow range of intensity near the epicenter and the wide range of intensities at some distance, it may reasonably be speculated that the greater intensities of the softer overburden are principally a magnified seismic response to the motion of the basement ro k. The fact that the principal faulting occurs in basement rock, which would therefore be the main conductor of the energy waves, substantiates this concept.

The relationship of greatest earthquake intensity for any epicenter distance for soft saturated ground, as indicated by the top curve of the Intensity-Distance Distribution Envelope, is of particular importance to the waterfront engineer. The great majority of harbors in the world are located on this type ground. The highest acceleration, greatest amplitude of ground movement, and most severe intensity of damage must be contemplated in engineering design for waterfront facilities.

1.08 Engineering Problem

The engineering problem for structures which may be affected by earthquakes includes all of the considerations of normal engineering design and construction in addition to four areas which are uniquely considerations of seismic design. It might be added at this time that structures which are designed for substantial wind loads or blast loads are, to some degree, inherently earthquake resistant.

The first consideration of the earthquake engineering problem is the determination of the seismicity of the region and the specific project area. As heretofore setforth, the earthquake history of the region in regards to both intensity and frequengy is the principal source of seismicity information for practical engineering purposes. The characteristics of the ground at the project site and the relative location of known active faults modify the regional seismicity for a particular project. The maximum accelerations and ground motion to be expected during the planned life of the structure must then be determined within the framework of this data.

The second consideration is the determination of the importance of the project and its various components, and the degree of earthquake protection required. It must always be kept in mind that the end product of earthquake engineering is a structure which is earthquake resistant and not "earthquake proof". A structure may be made relatively more earthquake resistant at an increased cost. A comprehensive

rationale is required of the economic or military importance and hazard to life and safety of each major component of a project against the relative degree of protection to be afforded and its attendant cost. Certainly a place of public assembly or an ore pier, the loss of which would close down an important plant until replaced, should be designed for greater relative earthquake resistance, regardless of the cost, than a relatively unimportant storage building. Professional engineering, the art of doing for $1.00 what others can do for $2.00, clearly requires a determination of acceptable risk predicated on the increase in original cost for additional protection weighed against the economic, military, and/or human consequences.

The third consideration of the earthquake engineering problem is the establishemnt of the design criteria. This criteria includes the allowable unit stresses for short duration seismic loading, condition of loading at time of occurrence, magnitude of lateral force and its pattern of application, factor of safety, and method of analysis. The design criteria may differ from structure to structure according to the importance of each, as explained above, but the design of any single structure should be balanced so each element resists in proportion to its ability. With the establishment of the design criteria, the structural design can be performed.

The fourth consideration is the planning of the earthquake

characteristics of the project other than structural. This includes features to reduce or eliminate hazards associated with earthquakes such as panic, fire, unleased utilities, disrupted communications, falling debris, etc.

A special consideration which, while applicable to all construction, is of particular importance to earthquake resistant construction is good field inspection. The great majority of reports on earthquake damage have stressed that poor field construction practices, stemming from lack of proper field supervision, contributed greatly to the damage. Earthquake design usually permits lower factors of safety during seismic loading than during normal loading. If a large portion of a structure's reserve strength is utilized to compensate for poor construction, little will remain for earthquake loading, the most critical condition of loading and probable source of extensive damage and failure.

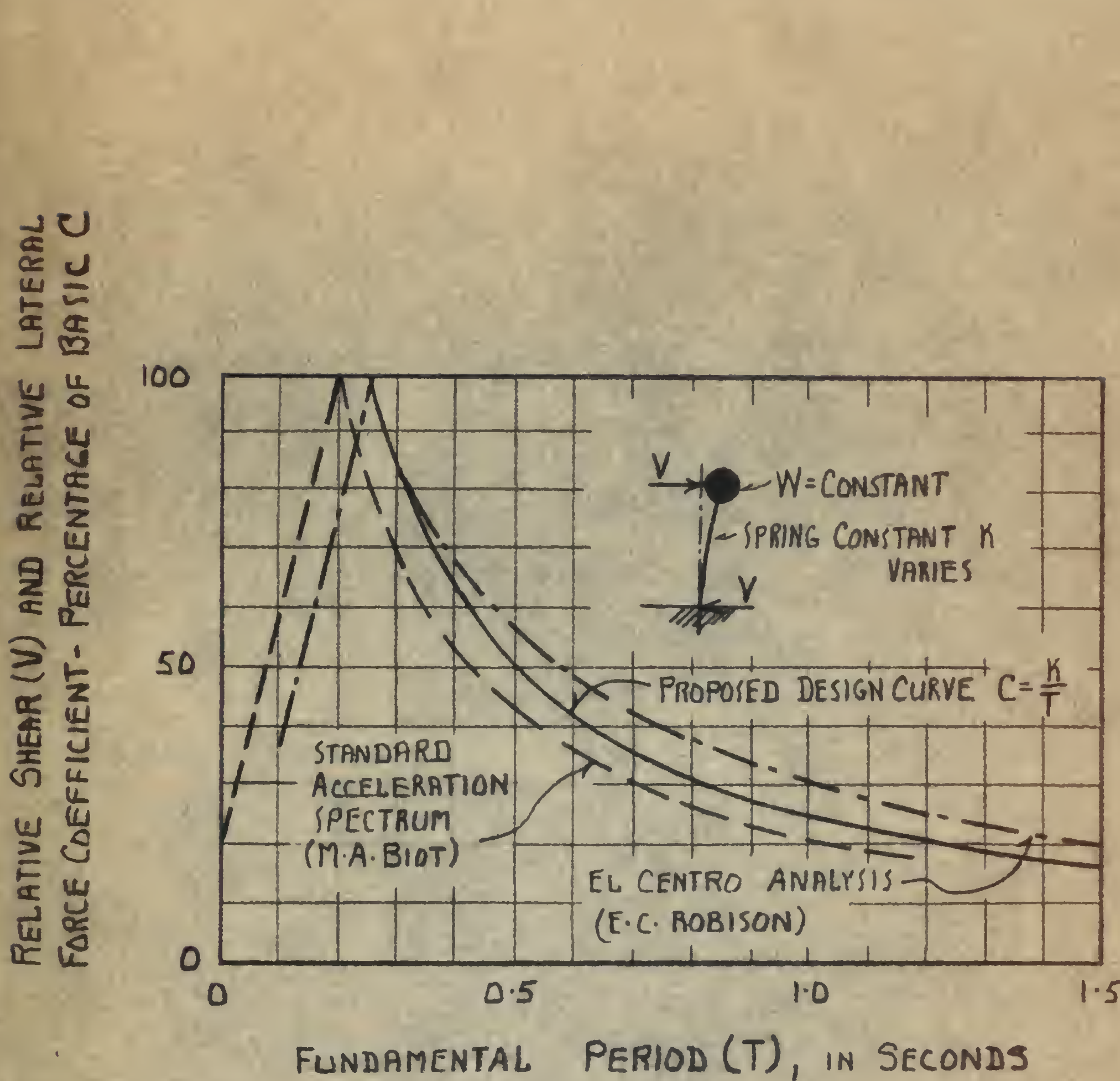
1.09 Earthquake Forces and Destructiveness

It has generally been recognized that the response of a structure during an earthquake is a dynamic vibration reaction of a transient nature. The intensity of this earthquake induced structural vibration is some function of the natural period of the structure.

To determine this relationship, the ground motion recorded for destructive earthquakes by accelerographs has been programed for shaking tables. Simplified models of

structures with various natural periods of vibration are placed on the shaking tables and their base shear recorded. A sketch of the model and the record of plots is shown by Plate VIII. These graphs are known as earthquake spectrums. Biot determined that the maximum base shear for a number of medium intensity earthquakes occurred at a period of 0.2 seconds. Robinson found the maximum base shear to occur at a period of o.25 seconds using the ground motion of the El Centro, California earthquake of May 1940. In both cases all lesser values of base shear were recorded as percentages of the maximum.

The third curve, designated "Proposed Design Curve C = K/T", is the idealized curve recommended by The Joint Committee on Lateral Forces of the San Francisco Section, ASCE, and the Structural Engineers Association of Northern California. (7) No reduction of base shear values is used for periods to the left of the maximum, and the minimum base shear is limited to one-third of the maximum. The value of the maximum base shear for any structure is determined by multiplying the weight of the structure by the applicable seismic factor, as explained in the section on Vibrations and Propagation. This value may then be modified according to the design curve for the fundamental period of vibration of the structure. Reference 7 setsforth the recommended method of determining the fundamental period of buildings and the distribution of base shear over the



RESPONSE OF SIMPLIFIED STRUCTURES TO EARTHQUAKES

(FROM A·S·C·E· TRANSACTIONS 2514, REFERENCE 7)

PLATE VIII

superstructure.

It should be noted that the maximum base shear reflected by the earthquake spectrums shown on Plate VIII occur at periods of 0.2 and 0.25 seconds. These periods of greatest structural response are reasonable as the records of earthquake ground motion reflect definite dominating periods between 0.1 and 0.3 seconds. Seismological records from Japan, where more frequent and greater intensity earthquakes are experienced, indicate that the basic natural periods of ground motion are 0.1 seconds for rock, 0.3 seconds for intermediate soils, and about 0.6 seconds for soft soils. Assuming the above data to be valid, it may reasonably be concluded that the records of earthquake motion upon which the Biot and Robinson earthquake spectrums were based are for sites on good firm soil. The design procedure for determining base shear predicated on these earthquake spectrums appear valid for structures built on rock and intermediate soil. This would make the procedure valid for the majority of large building construction, for which it was intended, as such structures are, by their very nature, generally situated on good sites. Prudence would lead this writer to use somewhat greater values of base shear, or lateral loading, for structures constructed on soft soils. The adverse effects suffered by modern tall buildings founded on saturated clay during the Mexico earthquake of July 1957 reinforces this contention. (28)

It should be kept in mind that the determination of

earthquake force and destructiveness by earthquake spectrums, as developed to date, is based on earthquake shocks of medium intensity without particular regard to the soil effects of the underlying material. Accelerograph data for earthquakes of greater intensity and the development of quantitative data on soil effects may well result in revisions and improvements to this concept.

1.10 Structural Behavior During Earthquakes

A structure is subjected to both lateral and vertical forces during an earthquake. The vertical forces are small compared with the gravitational weight of the structure and are generally disregarded. The lateral forces, on the other-hand, are large compared to conditions of normal loading and are the subject of earthquake structural design. A structure is subjected to forced vibration imposed upon it by the ground movements of its foundation. The inertia of the structure tends to resist these foundation movements. A condition of shear is thereby set up at the foundation, known as base shear, which is imparted to the structure to make it move. The lateral forces of an earthquake are therefore imposed as a base shear at the structure's foundation and counteracted by some distribution of lateral loading representing inertia and equal to the weight of the structure times the seismic factor. The earthquake forces, or base shear, and the inertia forces must satisfy the principle of statics, $F_H = 0$.

Rigid structures react to earthquake loading according to the principles of statics as setforth above. Plate IX illustrates the behavior of three rigid structures subjected to the same earthquake force of 0.1g under three conditions of loading. The first case of loading is obviously safe. The resultant of vertical and horizontal forces falls within the middle third of the base to insure both its stability and acceptability of unit foundation pressures. The second case of a top heavy load twice the weight of the supporting pier is of questionable safety. The reaction falls outside the middle third, and while stable, little reserve remains for other lateral loads; additionally, excessive foundation toe pressure is probable. The third case, where the center of gravity is above the supporting pier, is clearly unstable, and there exists an excessive concentration of foundation stresses. Such a condition of loading would require either a wider base or an anchorage to carry uplift in tension. The character of seismic loading is such that only narrow base objects are actually overthrown. The rate of reversing direction of force application does not permit time for wide base objects to fully respond before the character of force changes.

The behavior of a simple one-degree-of-freedom flexible structure during earthquake motion is also illustrated by Plate IX. If the mass is large and the columns are flexible, the natural period of vibration for the structure may be quite large. The short periods and small amplitudes of earthquake

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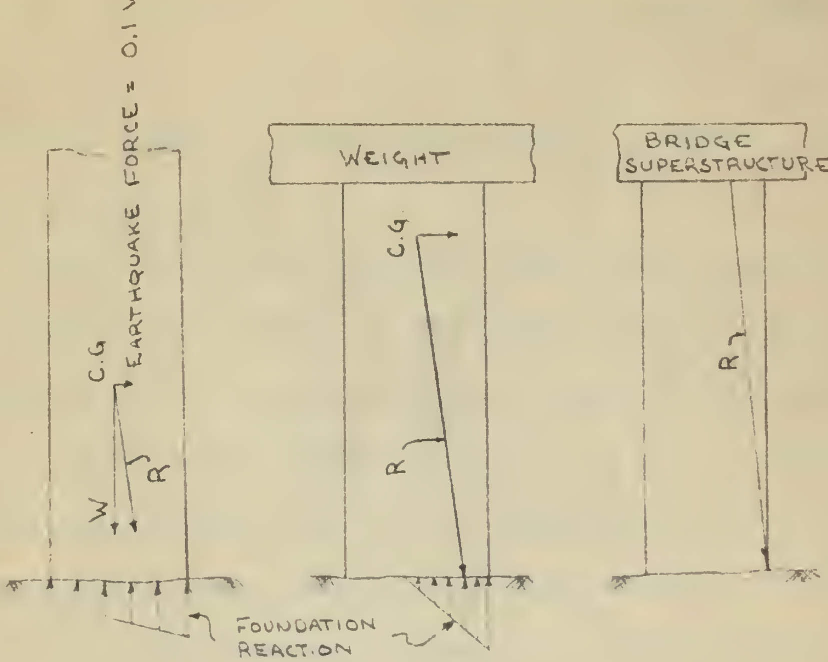
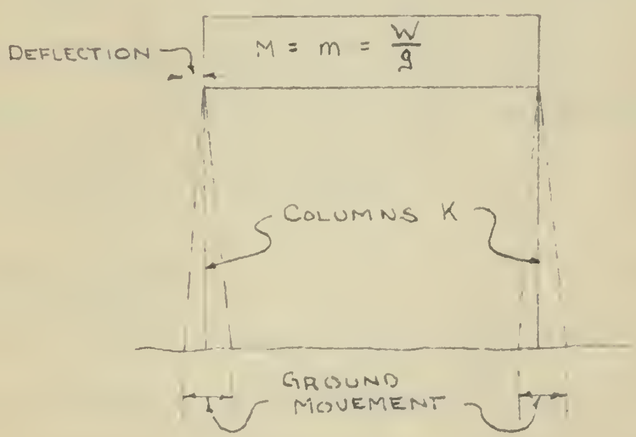


BEHAVIOR OF RIGID. STRUCTURES DURING EARTHQUAKES



BEHAVIOR OF FLEXIBLE STRUCTURES DURING EARTHQUAKES

PLATE IX

vibration will cause only slight response from such a flexible structure. The foundation will oscillate back and forth under the mass, but the base shear will be determined by the structural deflection, equal to the earthquake amplitude, instead of a function of the mass.

If the above described flexible system had a small mass and rigid columns, its period of vibration would be much shorter. During an earthquake with a longer period than it had, this structure would behave essentially as a rigid structure. If through partial failure of connections or structure-foundation integrity, the rigidity of the structure would diminish and its period would increase. As the period of the structure approached the earthquake period, the amplitude of motion at the top of the structure would increase. This occurs due to synchronization of the free vibration of the structure with that of the earthquake and magnification of the absorbed energy through resonance. The damping effects of internal friction and relatively loose connections prevents the unchecked effects of resonance.

1.11 Earthquake Design

Earthquake design is essentially the detailed analysis of a structural system by the methods of conventional structural analysis for a distribution of lateral forces which result from the acceleration of the structural mass. Its goal is an earthquake resistant structural frame, one which

will not collapse during an earthquake. This requirement may be satisfied by rigid structures which resist lateral movements through structural strength and by flexible structures which oscillate with, while damping, the vibrations induced by the earthquake, somewhat like a tree in the wind.

An example of the extreme which this latter concept might lead would be a major building which would not collapse when subjected to a heavy shock. However, close examination would reveal damaged walls, partitions, and trim; mechanical equipment requiring replacement and extensive repairs; broken utilities resulting in secondary damage; and loosened structural elements requiring repair. The cost of repairs might well constitute a large portion of the original cost of the building. (33)

It is therefore necessary either to limit deflections to values consistent with the flexural characteristics of the finish materials, or to provide mechanical details to compensate for excessive deflections, or to plan certain finish materials as expendable.

Therefore, earthquake design must be carried through the architectural and outfitting phases of project planning and design. Every facet of design and construction must be inquired into as to its reaction to earthquake loading. This places unique demands on the engineers and architects who practice in areas of frequent earthquakes.

Chapter 2

FIELD OBSERVATIONS

2.01 Introduction

Earthquakes have occurred throughout recorded history. They happen more frequently than is generally supposed. It is estimated that over a million earth tremors occur throughout the world each year of which 10.5 are of magnitude 7, Richter scale, or greater and which result in extensive damage to personal property, financial loss, and loss of life. (37) Equally destructive earthquakes can be expected to continue to occur at this rate for countless future generations.

Realizing these facts, engineers and seismologists, particularly in those areas subjected to frequent earthquakes, have undertaken scientific investigation of the phenomenon and its effects on the structures of man. The practical engineering goals are to identify those materials and techniques which result in relatively earthquake resistant construction, and to develop principles of analysis and design whereby reliable and economical structures can be constructed. This scientific investigation has taken the following three forms: (26)

a. Field observation: the recording, analysis, and correlation of destruction caused by large earthquakes - Early efforts, dating back at least three centuries, studied how the ground upon which a structure was situated influenced the probability and intensity

of damage. (26) Parallel effort was expended to learn which materials, methods of connection, and construction techniques would withstand earthquakes, and to determine by the nature of the failure how to better utilize marginal materials. With the advent of instrumented seismological records, damage observed in the field has been correlated with distance to the epicenter and the various parameters of vibration. This effort is handicapped by the difficulty of separating primary effects from secondary effects, such as fire. Additionally, seismological data are invariably obtained at locations some distance from the damage and having differing characterists. However, this type of investigation is the sound, solid foundation on which our present knowledge is based. (32) The findings in other areas of scientific earthquake investigation must necessarilly be verified by field observations of large earthquakes.

b. Field and laboratory studies: the measurement of small natural and simulated earthquakes on instrumented structures and models - This type of investigation generally involves observation of small, relatively frequent, natural earth tremors on instrumented structures. These structures are often permanently instrumented and located in areas of great earthquake probability in hopes of recording a large

scale earthquake. Additional studies are carried out in the laboratory, based on the laws of model similitude, by measuring mechanical induced vibrations. Additionally, electric analog simulations are carried out. (65, 102) Results of this work are verified by field observations.

c. Theoretical work: the approximation of natural phenomenon by idealized mathematical expressions - These expressions are developed as functions of the parameters of well established physical law. The expressions are then solved mathematically according to physical laws as a means of explaining the propagation and reactions of earthquakes. Results are compared with field observations for verification. The majority of theoretical work deals with propagation of energy through the earth's crust and vibrational response of buildings, stacks, and towers. This effort is adversely marked by both the difficulties of complex mathematics and the gross approximations of physical characteristics to permit mathematical expression. Nevertheless, reasonable approximation of natural phenomenon has been obtained. Future steps forward in the field of engineering seismology will follow upon the improved workability and established reliability of these methods.

Of the various forms of scientific investigation heretofore discussed, field observations have resulted in the great body

of knowledge. Large scale earthquakes cannot be simulated in the laboratory, and small scale experiments are not necessarily representative of full scale phenomena. Therefore, field observations take on special significance in acquiring engineering knowledge about earthquakes. Of necessity, the more sophisticated forms of scientific investigation have concentrated on the structural systems of buildings and other major engineering works. A structural system failure is an absolute failure with attendant loss of life and complete economic loss. However, damage short of structural failure may also result in loss of life, through falling debris and unleased utilities, and severe economic loss. For this reason the relatively few large scale earthquakes which result in substantial damage to a variety of engineering works are of particular interest to engineers. A great deal has been learned about how to reduce earthquake damage through field observations. Each new earthquake contributes to our engineering knowledge by providing new information, confirming old beliefs, and clarifying uncertainties.

Early field observations of earthquakes were qualitative enterprises. They have become more quantitative with the development of the science. The violence is measured by means of earthquake intensity scales similar to the Modified Mercalli scale described in Chapter 1. The intensity of motion as reflected by the effects of earthquake vibrations on animate and inanimate objects is plotted on earthquake intensity maps to define the contours, or isoseismal lines, of equal intensity.

[illegible]

Damage to various types of structures is correlated with the type of ground upon which they are situated. In the few early earthquakes where faulting could be located by surface displacements, structural damage by type of ground was also related to distance from the faulting. Seismology now provides the engineer with the epicenter location. With the establishment of the strong motion seismological recording program in 1931 (13), it became possible for engineers to corralate damage with the various parameters of vibration. The more important of the relations have been presented in Chapter 1.

Many engineers are inclined to design for earthquake loads as if they were, in fact, horizontal static loads on individual members. Through the study of field observations, however, engineers may gain a greater understanding of, and feel for, the forces with which they deal. By visualizing the erratic and lunging motion, the structure as a system of different materials with different properties, and the interaction of the subgrade-foundation-structural system responding to and damping the effects of an earthquake, an improved design will surely be achieved.

2.02 Earthquake Disaster

Many earthquakes which have occurred near communities have resulted in major disasters. San Francisco in 1906 and Tokyo in 1923 are the best known examples. Earthquakes of great violence have inflicted extensive damage without ending in disaster.

A disaster may be defined as the widespread loss of live, bodily injury, loss of personal possessions, and complete disruption of a community for a considerable period of time. (59) To engineers falls the responsibility for providing facilities which will suffer only reasonable damage and will not contribute to the hazards which might lead to disaster. Through field observations it has been learned that the direct damage caused by an earthquake is relatively minor compared with secondary damage when a disaster occurs. Engineers have also identified many of the hazards resulting from earthquakes which could readily lead to disaster. By recognizing these hazards and thereby giving them special consideration in design and construction in earthquake areas, the chances of a disaster occurring will be greatly reduced.

Structural resistance to earthquake induced failure is an important criteria for averting a disaster such as the toppling of a public building. This is the widest recognized hazard of earthquakes and the one which has received the most attention. A significant field observation is that while all parts of a structure subjected to earthquake shocks are stressed, openings, existing cracks, discontinuities, and rigid elements are subjected to high stress concentrations and reversals. (13) Symetrical structures suffer less damage than unsymetrical structures as they are subjected to less torsional stress. Severe structural damage is frequently observed along expansion or weak construction joints where portions of the structure can

move separately and pound each other with tremendous violence.

Falling debris constitutes a hazard of earthquakes. It injures and kills people when falling and impedes access upon accummulation on the ground. Falling debris is primarily nonstructural components such as parapets, curtain walls, and architectural elements of questionable value which should be avoided. The principles of structural design should be applied to nonstructural elements that might become a hazard.

Fire is probably the greatest hazard in an earthquake. It was the direct cause of most fatalities and damage in the San Francisco and Tokyo earthquakes. The nature of the fire hazard is numerous fires starting at the same time over a large area. Normal fire fighting technique of confining a fire to the area in which it was ignited cannot begin to cope with so many fires at one time. Abnormal traffic jams and debris in the streets make fire fighting even more difficult. Additionally, the water supply generally fails during earthquakes. Therefore, fire resistance should be built into each structure. Installation of fuel oil, gas, and electric lines should be shock resistant and contain automatic cut off devices in protected locations. Chimneys and flues should be earthquake proof by the addition of adequate reinforcement.

Water systems can be damaged in many ways. Elevated water tanks are among the most vulnerable structures; they must be rigorously designed for earthquake forces. Embankment slopes of reservoirs must be resistant to failure by utilizing flat

slopes and by containing methods of drainage to reduce saturation. Deep water mains are much more earthquake resistant than shallow mains. Water distribution system must be looped and valved so that sections containing failures can be isolated. Connections to structures should be flexible to resist shearing due to differential movements. All lines must have adequate structural strength as water surges double the normal line pressure should be anticipated concurrently with deformation induced stresses. A large amount of incipient damage generally appears for years after a shock. Water treatment facilities must also be earthquake resistant to prevent long duration outages and to protect the public health. Broken sprinkler systems cause considerable damage to material in otherwise sound buildings. Similarly, water from broken mains causes slides, settlement, and damage to basement.

Electric service is vital to the well being of the community. Its interruption should be reduced to a minimum or avoided. Communications, transportation, industry, other utilities, hospitals, heating systems, and residences are incapacitated without electricity. Broken lines constitute fire and electrocution hazards. Transformers should be securely fastened to withstand strong motions. Tightly strung wires are subject to breaking and loose wires can come in contact and cause shorting and wrapping. Flexible and weak poles contribute to line problems. Transmission lines over long spans are thrown into violent motion which damages insulators, lines,

and towers.

Sewage lines suffer where they leave structures and where they connect to manholes or similar collection structures. Deep lines perform better than shallow lines. Slides, sluffing of banks, and settlement of fills results in misalignment and leaks. Public health requires a minimum of random discharge and nominal outage of treatment facilities.

Communications become unreliable in earthquakes. Telephone lines are either down, as described for electric lines, or jammed with trivial calls. For emergencies, telephones are of little use and alternate methods of communications may well be a design consideration for emergency use.

Transportation is very adversely affected. Monumental traffic jams are compounded by fallen debris and accidents to a few of the vehicles moving at the time of the shock. Fire, police, and ambulance vehicles are prevented from fulfilling their missions. Rails are bent and pushed out of alignment. Highways and railways are both affected by slides, tunnel cave-ins, bridge abutment failures, and drawbridge mechanical failures. Slight runway settlements are sufficient to close airports.

Failure of storage bins, silos, tanks, vessels, and pipelines is another hazard when the contents are inflammable, explosive, poisonous, or radioactive. Their contamination of the water supply or flow into a harbor are a particular hazard. Structural integrity and emergency collection basins are required design features.

The panic of the people in the area is another design consideration. The danger of panic can be reduced by eliminating items, such as swinging fixtures, which can incite panic in public places. Ample means of access and logical location of installations can also reduce the risk of panic. The radio is a valuable way of distributing information and reducing confusion and panic.

2.03 Construction Materials

Structural steel frames perform exceptionally well when subjected to earthquake shocks. This is due to the great reserve strength of steel resulting from its yield properties. The damping characteristics of structural steel is low when secondary elements such as walls and floors are not present. Anchor bolts can be expected to yield during an earthquake and base plate details should be stiffened to resist horizontal shear. Special care should be exercised to permit hinged and moment connections to perform in the manner in which they were designed. Secondary tension diagonals should be avoided unless they can be provided with an initial stress. (105) The goal of structural steel design in earthquake areas is a balanced design where no particularly rigid structural element is subject to failure by having to carry a disproportionate part of the lateral force. In structures incorporating diaphragms, trusses, and rigid frames, these elements resist lateral forces in respect to time, and progression of the overall structural

[illegible]

deflection, in the order listed. (59) Fireproof covering for structural elements, when required by purposes of occupancy, must be vibration resistant. Many brittle materials generally used for this purpose are not structurally adequate. Reinforced concrete jackets are recommended. (32,54)

Reinforced concrete structures also performed well; however, damage at the junction of columns and beams is commonly observed. This is particularly true at corners, and to a lesser degree along perimeter walls, due to the increased stiffness of filler walls in two directions. This damage confirms the earthquake engineering maxim that earthquake forces are distributed between resisting members in the ratio of their rigidities, and that stiff members must be strong members. Damage to reinforced concrete columns is most prevalent in lower stories and occurs adjacent to rigid connections such as beams, two way slabs, and footings. (13) The location and amount of reinforcing steel in concrete are important items; the increase in the size may lead to a weaker element as cracks induced by increased rigidity may develop readily. Frequently, slight cracks appear in floor slabs parallel to and over beams. Movement along construction joints and pour lines is another common defect which is attributable to poor construction practice.

Masonry bearing wall with wood interior structures have been found to be the most vulnerable of all types to earthquake damage. They normally include no provisions for horizontal loads. Brick parapets, chimneys, foundations, and walls are

noted for their vulnerability. Rigid pilasters and walls often reflect shear cracks near their base. There are countless examples, however, of brick structures which have successfully withstood earthquake. Quality of workmanship is one of the indispensable elements of its successful use. Poor mortar is very often associated with failures. Lime mortar fails to react chemically to produce a calcium carbonate bond, and straight cement mortar performs poorly due to shrinkage and irregular curing. A lime-cement-sand mortar made in the ratio of 1:1:9 performs very well when used in conjunction with good workmanship. Header courses, ties, or horizontal reinforcement is required to tie together the exterior and interiors withs. Adequate interior bracing is required in masonry construction. The bracing must be relatively stiff to work in conjunction with the stiff masonry exterior walls. Joint and beam anchors which will develop the full strength of the materials are required. Reinforced concrete floor and ceiling slabs with proper anchorage to the brick walls, or reinforced concrete bond beams at floor and ceiling lines with proper anchorage to the floor and ceiling, greatly enhances the earthquake resistance of a masonry structure. All masonry elements with any degree of freedom, such as parapet walls and chimneys, require vertical reinforcement. Intelligent engineering and sound construction practices can produce reasonably earthquake resistant masonry structures.

Wood frame construction displays a high degree of earthquake

resistance. Stucco walls and plaster partitions provide added resistance to lateral forces. The most common damage, excluding toppling of brick chimneys crashing through roofs is lateral failure of the underpinning due to insufficient bracing. Most earthquake damage may be avoided by adequately braced underpinning, sills bolted to the foundation, and reinforced chimneys, either free to vibrate independently or rigidly connected to the structure.

Asphalt pavements are permanently deformed by earth movement, and concrete pavements are cracked and subjected to both heave and settlement during an earthquake. Asphalt pavement is often easier to repair while concrete pavement offers greater resistance to the lower magnitudes of movement and subsequent damage. As only limited areas are affected and repairs are not costly, considerations other than earthquake hazards should dictate the choice of materials.

Elevated water towers, especially when partially full, tall smoke stacks, and miscellaneous roof structures are particularly vulnerable in earthquakes. Lateral bracing and foundations are the key to elevated water tank design; however, they and tall smoke stacks require special study when designing for earthquake forces. The movement of a building compounds the movement of a separate structure resting upon its roof. Such structures also require special treatment.

2.04 San Francisco Earthquake of 1906

The San Francisco earthquake occurred at 5:12 AM, 18 April 1906 as a result of near vertical faulting of the San Andreas Rift. It was accompanied by horizontal displacements at the surface of the earth averaging 10 feet and up to a maximum of 21 feet. The faulting occurred along a nearly straight line over a distance of 270 miles. The fault passes within 10 miles of the center of San Francisco. (54)

The shock was the most severe recorded in the United States, being of both long duration and great intensity. It registered a magnitude of 8 1/4 on the Richter scale (37) and a maximum intensity of IV on the Modified Mercalli intensity scale. (18) The sensible duration of the shock was about 1 minute. (54) The nature of the motion was neither simple nor harmonic. It was described as "vicious, violent, chaotic motion" that "would rise up to a terrifying maximum, and then apparently die away, only to increase the tempo of the vibration until a person wondered if it would ever stop." (32) People walking about had difficulty remaining on their feet, and men on horseback were knocked to the ground along with their mounts. Many people suffered from nausea due to the movement, and animals in general were affected with terror. (54)

The field observations of the San Francisco earthquake demonstrated with remarkable clarity the relationship between soil conditions and intensity as measured by degree of damage. Without exception, the areas where firmly cemented bedrock was

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either exposed or covered by a very thin mantle of soil experienced the lowest grades of intensity characterized by cracked plaster and occasional fallen chimney.

This was contrasted by low lands and along portions of the waterfront where sand and alluvial deposits were thicker. The destructive effects were increased in magnitude and prevalence. Filled land suffered the highest grades of intensity with the greatest amount of damage occurring close to the water's edge, lessening in intensity upon approaching solid land to the back of the waterfront. Damage in these areas was characterized by shear failures and strains at the base of the building as though a sharp blow had been struck. The ground surface was deformed into waves and small open fissures were formed, especially adjacent to the wharves. Structures near the waterfront generally slumped seaward, in some cases as much as 2 feet . Settlements in fill in excess of 2 feet accompanied by broken slabs and foundations were prevalent. The behavior of fill suggested "that the materials used in filling were shaken together so as to occupy less space with the accompanying development of waves fissures, and structural damage." (54) It was also noted that the structural damage and subsidance were somewhat less on older fill. One area of fill extending into the bay was noted as an exception to the rule of greatest intensity of damage; it turned out to be composed of broken rock derived from the grading of neighboring rocky hills.

The benefits of piling was noted. A narrow fringe of

waterfront of partially filled and partially natural land was subjected to intensities sufficient to severely crack and occasionally collapse brickwork and masonry. Warehouses and buildings on piling were damaged less than structures in other areas of waterfront founded on similar soil. Similarly, the cable car system in areas of filled land had been constructed on deep piling to alleviate settlement. During the earthquake the cable car line was substantially less affected than the street on both sides which settled 2 feet. In general, "buildings erected upon good foundations withstood the ordeal well, even when the streets around them were depressed and fissured." (54)

The hilly area of San Francisco experienced numerous slides where the angle of repose was greater than that required for stability under vibratory forces. These slices were prevalent in unconsolidated fill not supported on the sides.

The preponderance of field observations dealt with the earthquake effects on buildings from which several important principles were demonstrated. It was shown that the relatively rigid steel and reinforced concrete structures of that period performed well if designed for lateral forces of 8% or more. (37) Wood structures rarely failed although they experienced large deflections and some damage. The ability of underpinning to take horizontal loads was one, if not the principal, key to the intensity of the damage to wood structures. Brickwork and masonry were unsatisfactory without reinforcement. Many fire

[illegible]

proofing materials failed structurally, vice thermally, indicating that they must have adequate shear and compressive strengths, as well as adequate tensile strength. The observations also brought out the fundamental fact that lateral forces in buildings are resisted first by the most rigid elements, generally walls or partitions.

Mud near the shore line of San Francisco Bay undulated within its mass and moved horizontally towards deeper water during the earthquake. Damage to small boat piers as shown in Plate X resulted from the horizontal displacements of the mud along the bottom. Ridges and troughs of up to 3 foot amplitude remained set in the mud within the tidal zone, and presumably in deeper water, after the shock. The tidal mud probably behaved as a quasi-liquid, having been thrown into waves by the agitation to which it was subjected. When the agitation attenuated, the mud returned to a quasi-solid state with the wave form preserved as it had existed at the instant of change. (54)

The shock was felt by many ships within San Francisco Bay and was also reported by others many miles out to sea in deep water. The shock was variously described as if the vessel "had struck on rocks at full speed" by ships near the quake and "had dragged over soft ground" by ships at greater distance. The vibrations were also likened to the quiver when the chain was running out the hawser and the effect on the hull of starting a large dynamo, but more exaggerated. Distinct shocks were felt on board ships in the harbor and loosely piled books were

[illegible]

shaken from a table. "One vessel was hurled against the wharf time and again, throwing down piles of lumber and shingles." (54) A more accurate description would probably have the wharf hurled against the ship time and again.

A great part of the damage which occurred in the commercial section of San Francisco was caused by the fire that followed the earthquake and by the dynamiting to contain the fire. The conflagration was compounded by the lack of water with which to combat the fire due to earthquake damage to the underground water lines. (39)

2.05 Tokyo Earthquake of 1923

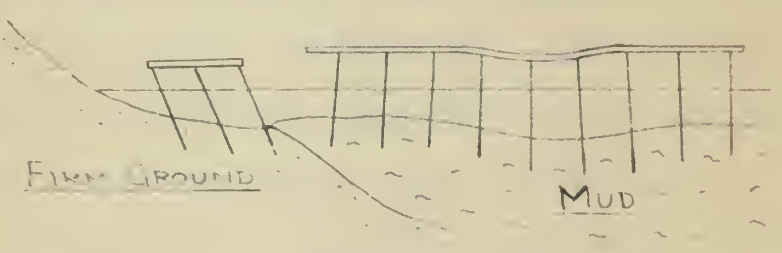
The Great Kanto Earthquake which devastated Tokyo, the port city of Yokohama, and many smaller cities and villages occurred about noon, 1 September 1923. It was the most destructive earthquake in recorded history resulting in 400,000 fatalities, 700,000 dwellings one half to completely collapsed, the capital of an empire laid in ruins, and a principal sea port destroyed. The magnitude of the shock was 7.9 on the Richter scale (39) and the intensity ranged up to X on the Modified-Mercalli scale. The destruction of the violent shock was compounded by the conflagration, including fire storms, which followed. The water system for combating the fire was destroyed by the shock.

The epicenter occurred in the middle of deep watered Sagami Bay about 40 miles south by southwest of Yokohama.

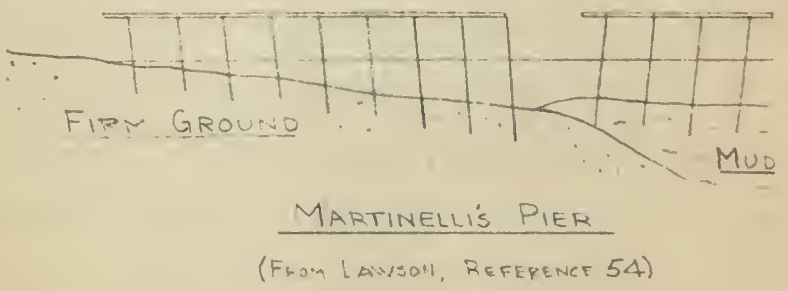
PIER DAMAGE DUE TO MUD MOVEMENT

SAN FRANCISCO BAY - 1906

SMALL BOAT PIERS - PLAN



BAILEY'S PIER



MARTINELLI'S PIER

(FROM LAWSON, REFERENCE 54)

PLATE X

Spectacular distortions occurred along the floor of the bay where 689 feet of subsidance occurred in one area bordered by an area of 820 feet uplift. Tsunami, or seismic sea waves, occurred as a result of this movement of the bottom reaching heights of 30 to 36 feet along the south shore of Sagami Bay. The damage was almost complete along the shore of the bay due to both the great intensity of the shock and sweep of tsunami resembling a very rapidly rising and very high tide. (30) Many of the inhabitants of this area were drowned.

Damage was extremely severe in Yokohama harbor. The custom-house pier and the new harbor quay wall collapsed, in part, under the shock. Both piers were crowded with passengers and well wishers. Two to three hundred people were thrown into the bay when the custom-house pier collapsed 100 yards from shore. The crecent shaped breakwater that protected the commercial harbor from the expanse of Tokyo Bay sank into the sea, leaving only fragments of the two light structures which marked the entrance to the harbor remaining above the surface.

Late in the afternoon when the houses near the waterfront were burning, oil poured into the harbor. Boats and scows loaded with people who had earlier taken refuge to escape the conflagration were ignited. Thousands of people were either burned or drowned as part of the harbor became a sheet of flames.

Most means of communications and transportation failed. Yokohama's two railroad stations, on filled land, collapsed. The rails were so badly damaged that they looked like coils of

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rope. Bridges were out, rivers and canal embankments had crumbled, and roads were useless due to fissures, earth slides, and conjestion of refugees. Communication terminal structures were destroyed, and telegraph and telephone wires were entangled, broken, and down. The initial communications from the area were sent by the ships in Yokohama harbor. Immediate food supplies to prevent mass starvation was provided by the ships in the harbor, and the brunt of the relief effort of bringing in food, clothing, and reconstruction materials, along with removal of refugees, was carried out by ships, especially those of the Japanese Imperial Navy. The importance of the seaport in a disaster where other forms of transportation cannot be depended on and the requirement for earthquake resistant waterfront facilities is well recognized in Japan. (78)

2.06 Long Beach Earthquake of 1933

The Long Beach earthquake occurred at 5:54 PM, on 10 March 1933. Strong motion of the initial shock lasted from 10 to 20 seconds, and 34 aftershocks were recorded that day followed by 30, 5, 3, 4, 1, and 1 on the succeeding 6 days. The magnitude of the shock was 6.25 Richter scale and the intensity as determined by damage ranged from VII to IX on the Modified Mercalli scale. The epicenter was located 3 1/2 miles offshore from Newport Beach and the depth of the focus was 6 miles. The dominant horizontal period of vibration was 0.3 seconds with a trace amplitude of 7 centimeters and a corresponding

acceleration of 0.23g. The estimated horizontal ground displacement was 0.5 centimeters. The vertical record indicated comparable movements of waves with periods of 1.0 to 1.5 seconds superimposed with waves with periods of 0.1 to 0.2 seconds.

This earthquake marked the beginning of the era in U. S. seismology where extensive quantitative data obtained through seismological instrumentation became available. Field observations took on expanded significance with the correlation of damage, soil condition and distance from shock with the physical characteristics of the shock.

It was observed that damage did not decrease uniformly with distance from the center of shock. The damage increased with the thickness and the fineness of recently deposited and relatively unconsolidated alluvium and fill. Groundwater near the surface intensified the shock

In the harbor substantial settlement and lateral movement of rock mound bulkheads was observed. Timber sheet pile bulkheads which were not anchored with backstays well behind the plane of rupture also moved towards the channel resulting in severe settlement of earth fill and pavement behind the bulkhead. Adequately anchored bulkheads withstood the effects of the earthquake and required no subsequent repairs.

Piers were generally observed to have suffered damage at their point of connection with the land. Timber wharves and waterfront structures supported on timber piling underwent

maximum lateral movement towards the channel in excess of one foot. (13) The reference states no direct cause for this movement. However, it may be conjectured that this movement was due to either, or some combination of, vibration induced horizontal consolidation in the direction of least resistance and/or a slide failure along a slip circle with the piles acting as dowels after sufficient movement to mobilize the passive resistance of the fill. A shallow slip circle is suggested as the piling was deflected towards the channel, vice a rotation towards the shoreside, and the fact that the area shoreward of the wharf was unaffected. Transit shed columns and roof trusses supported on both the wharf piling and the shoreside suffered excessive strains and numerous failures due to the movement of the timber pile wharf structure towards the channel. Inclination to slide would cease with the termination of the earthquake vibrations.

Underground potable water lines, fire mains, and vitrified tile drainage line were damaged, particularly where they passed through bulkheads on to piers and wharfs. This permitted a considerable amount of fill to be washed out with resulting settlements and cracking of transit shed floors. Some of this floor settlement might very well have been due to a partial slide failure as conjectured in the previous paragraph.

The damaged piping was located 3 to 10 feet underground, although it was generally observed that deep lines suffered little damage, even in unconsolidated fills which underwent

[illegible]

significant settlement. Practically all pipeline connections to transit sheds were broken resulting in an outage of the fire and sprinkler systems at the time most likely needed.

No movement of rigid concrete pile wharf structures could be detected. It cannot be concluded, however, that relatively rigid waterfront structures perform better than flexible structures as many variables independent of the structures may dictate movement and failure in an earthquake.

Damage occurred to street pavements in the form of cracks and settlements, but was not as extensive as that to transit shed floors. The shore end of a newly constructed mole was thrown out of line and some settlement of fill occurred. No permanent damage to the mole was discovered. Horizontal and vertical movement was considerably greater in marshy and filled areas than it was on solid ground.

2.07 Agadir Earthquake of 1960

Agadir, Morocco, located on the Atlantic coast of North Africa, was struck by a violent earthquake at 11:41 PM on 29 February 1960. The magnitude of the shock was a relatively moderate energy release of 5.75 Richter scale. The epicenter was located about 10 miles northwest of Agadir at the base of the Atlas Mountains, and the focus was only 3 to 4 miles deep. Disproportionately high intensities, in relation to the magnitude, of IV and X Modified Mercalli scale were experienced. These intensities generally correspond with energy releases

30 to 50 times that which occurred at Agadir, and were a result of the shallow focus. Analysis of structural damage to a modern well designed reinforced concrete structure located near the waterfront indicated that it was subjected to horizontal accelerations of .08 to .10g. (23) About 10,000 persons were killed in this earthquake.

Morocco was believed to be a stable area as no important seismic activity had ever been recorded. It was known, however, that Agadir was situated on an important continental fracture between the great folded Maghren area to the North and the horizontally stratified Sahara, and that it was crossed by a network of faults. (23) Accordingly, this earthquake is important to waterfront engineers as it demonstrates the effects of earthquake shocks on a modern harbor which had not been designed for seismic conditions.

In the harbor heavy concrete block gravity quay walls were displaced horizontally, in some cases as much as 30 inches. This is not an indication of exceptional seismic activity when the weight of the water displaced and the additional lateral pressures of pore water under acceleration are taken into consideration for stability. The increased lateral pressure described as due to excess pore water pressure might be equally well explained as a loss of shear strength of loose granular material where liquefaction of the soil-water mixture occurs during earthquake motion. As the earthquake lateral forces are oscillating, or reversing direction, slight movements of the

marginal restraint on the liquefied granular material, that is the gravity quay wall, are accumulative in one direction. Differential movement of the blocks generally occurs at the ends of a gravity quay wall as was observed at Agadir, see Photo I, and at corners. This photo also shows a small slide of a rock faced jetty across the channel.

The horizontal movement of the quay wall accompanied by consolidation of fill resulted in extensive settlements behind both the shoreside berths, Photo II, and the south berths, Photo III. In both cases one luffing crane rail was located on the gravity quay wall and the other was located on wooden ties on fill. The differential settlement of from 1 to 4 feet between rails, in conjunction with the earthquake motion, tipped over 5 cranes causing considerable secondary damage to the cranes and adjoining transit sheds. The settlement extended well back of the quay walls and seriously damaged the paved area. There appeared to be negligible settlement of the gravity quay wall.

The aggravated settlement of new unconsolidated fills when acted upon by a vibrating load is demonstrated by the settlement crater adjacent to the quay walls shown in Photos II and III.

Photo IV illustrates the common slide failures which generally occur along the waterfront. It should be noted that the cracks are all parallel to the rip rap slope and the waterline. Differential settlement under the transit shed is evidenced by the level of the eave line and the cracked wall. The settlement appears to be greatest in the middle of the shed as would be

FIG 7

Photo I: Differential Movement at Corner of Gravity Quay Wall
Agadir 1960

Photo 11: Outward Movement and Settlement behind North Gravity
Quay Wall - Agadir 1960

Fig 17

Photo III: Outward Movement and Settlement behind South Gravity
Quay Wall - Agadir 1960

Fig 13

expected for the consolidation of fill under uniform loads. Extreme settlement behind the quay wall may also be observed.

The good performance of properly designed structural steel in an earthquake is also shown by Photo IV. Strap, or tension, diagonal bracing running longitudinally at the eave line of an adjacent shed evidenced buckling. This demonstrates that secondary tension members should contain initial stresses. (107)

2.08 Chile Earthquake of 1960

A series of strong motion earthquakes occurred in the South Central region of Chile on 21 and 22 may 1960. No less then 13 earthquakes of magnitude 6 Richter scale, or greater, were recorded. Damage occurred over a region 300 miles long extending from Concepción to Puerto Montt. The first large shock, of magnitude 7.75, occurred at 6:02 AM on 21 May 1960 approximately 30 miles south of Concepción in the area of Arauco Penisula. The largest shock was of magnitude 8.75 (35), larger than either the San Francisco earthquake of 1906 or the Tokyo earthquake of 1923, occurred at 3:10 PM on 22 May 1960. Its epicenter was some unresolved distance offshore from Valdivia. These distrubances were believed to have taken place at depths of about 30 miles. Modified Mercalli scale intensities of VIII at Concepción, X at Valdivia, VII to VIII at Orsono, and XI at Puerto Montt were recorded. (25)

Coastal settlements of up to 5 feet occurred at Valdiva

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while uplifts of 4 feet were noted on the mainland and 6 feet on a nearby island. A lava flow erupted from a new vent in a volcanic mountain located inland in the Andes range. The underwater shock generated a tsunami, or seismic wave, which reached a maximum height of approximately 28 feet above mean sea level west of Valdivia. The maximum heights of the tsunami waves when they reached Hawaii and Japan were 15.5 and 13 feet respectively. Substantial damage and loss of life occurred at both locations. Some 4000 people were killed as a result of the earthquake in Chile and another 450,000 peoples' homes were damaged or destroyed.

Chile is recognized as an active seismic area. Earthquake resistant structural design has been required by the building code in recent years. Direct evidence of the effectiveness of the code was observed in Concepción. This city was severly damaged by a major earthquake in 1939. New construction to replace destroyed buildings was of seismic design, and the 1960 earthquake provided an unparalleled opportunity to contrast its relative resistance with nonseismic designed structures. It was observed that these modern structures survived the shocks with minimum damage. The few instances of poor performance could be traced to design errors or faulty construction practices and to special structures for which the provisions of the code were inadequate. Lateral coefficient of 0.12g uniformly distributed and an increase in allowable strength of 20% produced outstanding results. By contrast, residential construction

and several buildings which survived the 1939 earthquake suffered extensive damage. The experience around Concepción brought out with clarity the need of adequate inspections of construction and forceful enforcement of the provisions of the code.

As generally observed in earthquake investigations, the damage was most severe along the waterfronts of Talcahuano (Concepcion), Valdivia, and Puerto Montt. The damage in many locations was aggravated by a 22 to 28 feet rise in the water level due to the tsunami.

The damage in Talcahuano harbor, located adjacent to the city of Concepción, was due solely to the forces of earthquake acceleration. A heavy reinforced concrete ore pier resting on steel H piles suffered only slight damage to a bent of batter piles at the transition to the approach mole. It is interesting to note that the damaged piles were not a part of the seismic design. This pier was designed to carry a 5 1/2 ton ore unloader, weighing 210 tons, and 2 railroad tracks on piling driven to 60 ton bearing in accordance with the Engineering News formula. The pier was designed as a frame over its length with 2 inches deflections between points of pile cap fixity and foundation fixity of the piling computed at 52 feet below the deck. This results in a seismic factor of 0.008g with a 5 second period of vibration in the direction of its length and .10g with a 0.17 second period for its width. (105) After the bent of extra design batter piles failed, the pier rode out

[illegible]

the earthquake without further damage.

Quay wall damage occurred at the naval base located at Talcahuano. A reinforced concrete caisson quay wall founded 31 feet below and extending 10 feet above the mean water level slid as much as 14 feet into the channel. See Photo V. Overturning rotations of up to 8 degrees due to either earthquake induced lateral pressures or settlement was observed. The subgrade is fine, loose sand mixed with shells of small sea animals and a clay sensitive to remoulding. This failure was induced by some combination of earth pressures behind the wall magnified by the earth shocks, toe crushing of the poor foundation materials by the rock pad when subjected to rocking of the caisson, and low frictional resistance between the caisson and rock pad. These movements were accumulative over a long period of time and several earthquakes. Sliding of up to 6 feet was observed after the 1939 earthquake; however, the greatest part of the movement and rotation occurred during the 1960 shock. Subsidence behind the quay wall, loss of fill, and distortions at corners and ends accompanied the lateral movement. Similar damage, consisting of 10 feet lateral displacement and overturning rotations of up to 10 degrees, occurred to a concrete block quay wall founded 37 feet below mean water at the same naval base. See Photo VI. It is interesting to note that a nearby rigid masonry graving dock built in the 1890's on bed rock evidences a complete lack of damage. (95)

Waterfront damage was also extensive at Puerto Montt where

[illegible]

Photo V: Outward Movement and Rotation of Molo Chacabuco,
Naval Shipyard, Talcahuano, Chile 1960

Photo VI:

the intensity of the shock was XI on the Modified Mercalli scale. An 18 foot high concrete trapezoidal section, resting on a 9 1/3 foot base, of a 58 foot gravity quay wall was overturned and dropped into the harbor from its position on a large, stable caisson. Similarly, a 45 foot gravity quay wall of more slender proportions overturned. Both quay walls evidenced sliding along their base. A total of over 1500 feet of concrete quay wall failed.

Two sections of sheet piling bulkhead were located along this same waterfront, and were presumably subjected to equal lateral forces. A 28 foot section from top to bottom of wall, with two levels of tie back anchors survived without damage. An adjacent 25 foot section with a single level of anchorage failed due to bending and ultimate cantilever failure of the sheet piling above the tie back wale. The sheet pile bending introduced rotation to the wale which, in turn, caused the 2 1/2 inch diameter tie rods to fail at the connection. There was no evidence of direct tension failure of the tie back anchors or kick out at the toe from inadequate depth of penetration. The failure above the tie back wale was believed to be the result of reduced cross-section due to excessive rusting rather than faulty design.

Various ports in Chile experienced tsunami waves of up to 28 feet above mean water level. A drawdown of up to 20 feet could reasonably be expected to have accompanied these seismic water waves. Very large lateral earth pressures due to

saturated backfill unrestrained and unbouyed by the normal water level could very well have accounted for some of the gravity quay wall lateral movement and overturning experienced along the coast.

2.09 Conclusions

Civil engineering techniques are largely devoted to the analysis of primary stresses over the principal structural system and to the investigation of sites and common field materials. This intelligence is integrated with known physical properties of materials as attenuated by time proven safety factors to result in an engineered design. The quality of the design is greatly tempered by the additional ingredient of engineering judgement. Many considerations of design are not satisfied by recognized engineering techniques. Indeed, engineering techniques profit from engineering judgements within the limits of their variables. This prerequisite to engineering sophistication, engineering judgement, is acquired through experience.

Experience in major engineering works may be both difficult and expensive to acquire. However, the study of the experiences and observations of others is a feasible alternative. Valuable insight may be acquired through such study.

Many important aspects of waterfront engineering in earthquake areas have been setforth in this chapter. The most important basic conclusions, which have been observed repeatedly

and which are of universal application, are:

Earthquake damage is most severe at irregularities in the surface geology, particularly in zones of transition from fragile to solid formations. Waterfront installations are located in such an area and are noted for the violence of the shocks they must sustain. This condition is compounded by the fact that the areas of greatest seismic activity are along the continental coast lines and among the islands of the world.

Structures with deep foundations and deeply buried pipe lines are inherently earthquake resistant. Short rigid connections between elements should be avoided.

Soft ground, particularly where the ground water level is high, moves with greater accelerations, velocities, and displacements than does harder ground.

Earth slides, including liquefaction of saturated fills, are a common occurrence during earthquakes.

Stiff elements absorb a disproportionate amount of the load and suffer the most damage during an earthquake. The ideal of earthquake engineering is a balanced design wherein each element shares in bearing the load according to its ability. Failure of a stiff element increases the period of vibration of a structure and frequently improves the structures ability to withstand the shock.

Secondary damage resulting from earthquakes is often much worse than damage caused directly by the earthquake motion. Earthquake disaster is the result of secondary occurrences.

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Structures and elements which have successfully withstood a shock may have incipient failures. They may be dangerous in future earthquake of less intensity. Each shock causes damage and this damage is latent.

Earthquake damage is <u>not</u> beyond the scope of man to control. The damage which occurs to facilities built by man is due to man's lack of foresight and his misunderstanding of the forces involved.

Chapter 3

SEISMIC SOIL PROPERTIES

3.01 Introduction

The first design consideration for structures which will resist the forces of earthquakes is the seismic characteristic of the ground. The crust of the earth is made up of soil and rock, and the soil, in turn, is made up of gravel, sand, silt and clay. There are considerable differences in the seismic behavior of these basic materials depending on their mechanical properties, moisture content, mixture with each other, and manner of loading. The magnitude of seismic force to which they are subjected is another variable of their behavior as an engineering material.

Many examples of structural damage are attributable to the change of stress distribution in the ground or the change of engineering properties of the soil during earthquakes. They include subsidence of fills, settlement of structures and embankments, landslides, and movement of retaining and quay walls. The importance of these changes in engineering properties, as related to the intensity of seismic vibrations, to engineering design is readily apparent.

There are two distinct approaches to the evaluation of seismic properties of soil. They are both essential to the understanding required for intelligent engineering design. The first is the correlation of earthquake damage with the

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geological character of the underlying material. The second is the analysis of the basic engineering properties of soil under the influence of seismic loads.

This chapter is divided into two parts to describe separately the results of these two distinct approaches to the study of seismic properties of soil.

3.1 - Geological Effects

3.11 General

The correlation of earthquake damage to the character of the underlying materials was the earliest form of field observation dating back at least three centuries. Observations have been made and recorded in all parts of the world. With the development of the sciences of geology, seismology, structural analysis, and statistical compulation, improved techniques and more constant points of reference were introduced. The world wide correlation of this data has resulted in practical knowledge of how location, with its particular features of geology, affects the intensity of seismic loads and the damage to various types of structures. The statistical efforts of the Japanese based on a large number of similar residences and frequent major earthquakes have contributed greatly to our knowledge. The first part of this chapter briefly describes the relationship of geology and seismic effects. This material must be understood in relation to the propagation of seismic energy through the earth's crust, distance from the epicenter,

and intensity of vibration as setforth in Chapter 1.

3.12 Influence of Soil Type

The characteristics of the soil upon which a structure is founded is considered to be the most significant factor relating to the magnitude of seismic force upon that structure. A structure founded on hard rock normally suffers only slight damage unless the rock is very fragile. The probability of damage increases, and the difference in the amount of damage to structures of various construction materials becomes discernible, as the soil becomes softer. The amount of structural damage, by construction material, corresponds to the strength of that material to resist the unique stresses to which the structure is subjected during an earthquake, as discussed in Chapter 2.

In Japan, ordinary wooden residences with heavy sod or tile roofs are certain to collapse during a violent earthquake when built on soft ground unless designed for lateral loads of 0.3 to 0.4g. The damage rate for these residences decreases conspicuously when founded on intermediate soils. The damage rate is usually very small and of minor nature for hard ground sites. A similar relation was found between structural damage to reinforced concrete buildings and soil conditions in the area of downtown Tokyo after the 1923 earthquake. (7)

Earthquake damage surveys in Japan have repeatedly and overwhelmingly noted the desirability of construction on firm tertiary formations, compact conglomerate, and rock. Structures

founded on littoral sand dunes, beaches, bars, spits, river flood plains, and diluvial volcanic formations suffer greater damage. However, far and away the greatest damage is suffered by structures founded on river deltas, drowned valleys, reclaimed lagoons, muddy alluvium, and made land. Damage experienced by the second group of ground types, while considerable, is closer to that suffered by the first hard group than to the third very soft group. (26)

Accordingly, there is considerable justification for the use of seismic coefficients which vary according to the nature of the soil conditions at the site. The seismic coefficient would be relatively small for hard rock and would increase with the softness of the soil, with modifications for types of construction. Predicated on extensive statistical surveys of earthquake damage in Japan, the following percentages of reduction to seismic design coefficients for various values of soil bearing pressure were developed. (7)

Percentage Effect of Soil Type on Seismic Force

Soil	Type of Construction			Corresponding soil bearing capacity
	Wood and light steel	Reinforced concrete and concrete block	Masonry	kips/sq ft
Soft	100	100	100	1.0 or less
Intermediate	80	90	100	———
Hard	60	80	100	4.0 or more

These reduction percentages reflect a conservative estimate of the soil condition-damage relationship.

Seismological records in Japan indicate that the basic natural periods of ground motion are approximately 0.1 second for rock, 0.3 second for intermediate soil, and 0.6 second for soft soil, varying over a wider range. (7, 52) Earthquake damage surveys have revealed that structures with the same period of natural vibrations as the ground often experience spectacular damage. The Japanese wooden residences with heavy roofs, mentioned above, have an average natural period of around 0.5 seconds, corresponding closely to the soft soil upon which they experience the greatest rate of damage.

There have been frequent examples of this natural period of structure to natural period of ground motion relationship resulting in severe damage. It has led to serious consideration of design methods utilizing such a relationship. However, it was found that structures on soft and intermediate soils undergo a change in natural period of vibration with increased vibration. Within the range of accelerations of most destructive earthquakes, the natural periods of structures have been measured to increase by the following percentages. (7)

Type of Construction	% Period Elongation
Concrete Block	140 to 230
Brick Masonry	180
Reinforced Concrete	200
Prefabricated Reinforced Concrete	300

It has also been found that comparitively large amplitude motion occurs over a wide range of periods in ground consisting

of thick and soft layers. Consequently, structures founded on such ground are at a disadvantage in an earthquake regardless of their natural period. (52) Consideration of a design procedure utilizing the natural period of the structure in relation to the natural period of ground motion has generally been abandoned.

One exception has been observed to the rule of least damage on hard soil. Small earthquakes cause more damage to masonry structures on hard soil than on soft soil. The natural period of rigid masonry structures is very short, corresponding closely to the period of the under lying rock. The vibrations are magnified through resonance to account for the greater damage. This explains the 100 percent seismic force factor for masonry construction on hard soil in the Percentage Effect of Soil Type on Seismic Force table. Violent earthquakes generally inflict severe damage on masonry structures despite their being built on soft soil. Related observations were made of the Mexico earthquake of 28 July 1957. Modern tall structures suffered extensive damage in Mexico City some 170 miles from the epicenter while old rigid masonry structures were undamaged. Modern buildings similar to those damaged in Mexico City were essentially undamaged in Acapulco some 60 miles from the epicenter. The cause of this unusual damage pattern was the 1000 feet of saturated unconsolidated clay which constitutes the bed of former Lake Texcoco. The great distance from the epicenter necessarily results in a longer predominant basement

rock period of slight amplitude. This rock oscillation was magnified many fold by the deep strata of saturated clay. The period of the amplified earth motion approximated the relatively long period of the tall flexible buildings causing extraordinary damage while having no significant effect on the rigid, short period, masonry structures. (28)

Ground with a single surface layer over basement rock has a resonance curve type of spectral response. The period of the predominant frequency is also the period for which the amplitude is greatest. Structures with a natural period of vibration approximately that of the predominant frequency of the ground will be adversely affected due to magnification of the ground motion through resonance. When the number of surface layers is greater than one, resonance phenomenon producing predominant frequency and amplitude occurs rarely. This type ground generally consists of thick soft layers wherein the earthquake motion may have comparatively large amplitude over a wide range of periods. As a result, and as previously setforth, structures founded on this type material may be adversely affected regardless of their natural period of vibration. (52)

3.13 Accelerograph Records

The strong motion accelerograph program of the United States Coast and Geodetic Survey initiated in 1931 has borne out much of the preceding material of this chapter. Damped velocity spectra indicate that above a certain minimum period,

the velocity of earthquake motion is distributed randomly about a mean value of velocity which is independent of period. In several instances where more than one earthquake has been recorded by a station, examination of the spectra did not disclose any special features that could be attributed to the locality. Also, where more than one station recorded a single earthquake, there were no characteristics of the spectra which were attributable to the particular earthquake. It should be recognized that these records may be influenced by the vibrational properties of the building housing the instrument and its interaction with the foundation material, but this lack of relationship seems to bear out the advisability of disregarding natural period of the ground as a design parameter.

Two interesting cases of strong motion accelerograph station site condition are worth noting. The station at Helena, Montana, was on rock, and the 1935 acceleration spectra indicated a predominance of short period waves. A similar verification of soil type-period of vibration relationship was noted at the Seattle, Washington, station situated on water-soaked fill; the 1949 acceleration spectra indicated a predominant period of 0.9 second, while those recorded on firm alluvium at Olympia were much smaller. Both stations were about equidistant from the epicenter. (26)

Vibrational observations on full scale structures undertaken by the U. S. C. G. S. have verified the fact that the period of structures increases during an earthquake due to

yielding of the foundation. These observations also revealed that when a structure subjected to a strong earthquake becomes damaged, its period increases due to loss of rigidity. As the elastic resilience of the structure is partially destroyed, its internal damping will be somewhat higher and this will offer additional protection to the structure,partially offsetting its reduced elastic strength. (104)

3.14 Foundations and Settlement

A complication of earthquake damage reports by Duke has revealed that soil type is the principal determiner of damage. In nearly all earthquakes studied settlements and lateral, or rotational, displacements of bridge and building substructures have occurred while foundations of better soils and rocks have not suffered similarly. A Modified Mercalli intensity of approximately VIII is the lower limit earthquake violence which results in bridge foundation failure. The fact that end bridge piers are usually situated on firmer ground than center piers contributes to earthquake damage through differing reactions and movements. On a number of occasions bridge piles in soft ground have been forced deeper into or partially pulled out of the ground. Duke recommends design practice to include, specifically for earthquake protection, the utilization of sound vertical load transfer practice, the provision for seismic horizontal design forces between substructure and ground for foundations, the minimizing of earthquake induced differential

[illegible]

settlement and relative horizontal movement of footing, and the incorporation of features to prevent rotation of bridge piers and bents in vertical plane. (27)

3.15 Fills and Consolidation

Duke made a similar study of damage to earth dams. Cracks parallel to the crest was the most frequent type of damage, and slide were much more common on the saturated upstream slopes then on the drier downstream slope. He noted that dams designed and constructed in accordance with modern practice, including controlled compaction of selected materials, prevention of excessive pore pressures and seepage, relatively flat slopes, firm foundations, and provisions for lateral seismic force have not been damaged by earthquakes. It cannot be stated for certain that modern dams will not fail when located over a large fault although damage is inevitable. A Modified Mercalli intensity of approximately VII will cause damage to older, less scientifically constructed dams. Duke recommends engineering design investigation of sliding failure of part or all of the upstream and downstream faces for extreme conditions of pore pressure and saturation, longitudinal cracking due to seismic distortion, transverse displacement with respect to abutments, settlement of both the dam and its foundation, loading of appurtenant structures by differential movements of soil, fault movement across the dam, and scour effects of earthquakes induced water waves. (27)

Much greater settlements will occur during an earthquake in any type of semiconsolidated soil, particularly lacustrine, deltaic, and flood plain deposits containing a large proportion of clay and silt than in natural deposits of granular material. Fine silts and clays may actually flow due to temporary liquefaction. This characteristic is known as thixotrophy and occurs where severe agitation destroys the existing particle structure, similar to the response of sensitive clays to pile driving. In general, seismic waves consolidate soil if it is wet and flatten its slopes if the soil is dry. Naturally, consolidated embankments of wet soil also have flatter slopes. The average settlement of 12 new and relatively dry fills, averaging 40 feet in height, during the 1931 Napier, New Zealand earthquake was 11.3 per cent. Relatively wet fills settled 13.6 per cent although no data is available on their age. (34) Such settlements can reasonably be expected to be less for old fills than for new fills regardless of whether they are wet or dry.

3.16 Landslides

Landslides are a frequent occurrence during earthquakes. They range from slight creep of detritus to great rock slides weighing millions of tons and covering hundreds of square miles. In addition to the numerous slides of steep mountain slopes, earthquake triggered landslides have been reported in flat, plains areas. Relatively steep and saturated shoreline embankment

are frequent victims of sliding during earthquakes. In fact, most scales of earthquake intensity utilize the event of landslides as an index of intensity in the range of the more destructive earthquakes. At the same time, and of particular significance, there have been many landslides triggered by low intensity seismic activity. The sensitivity of the slope is as important a prerequisite to sliding during earthquakes as is seismic shock.

A study of landslides during earthquakes conducted by Leeds established the following important generalizations:

a. Landslides are the result of many interacting causes, among which earthquakes occasionally function as the direct cause, or trigger.

b. The great majority of landslides are not earthquake triggered.

c. Earthquakes of Modified Mercalli Intensity VI, or greater, usually trigger landslides. Intensities as low as IV have been known to trigger landslides. The number and severity of landslides increase with intensity. (37)

3.17 Slopes and Embankments

The most stable materials for slopes and embankments subjected to earthquake shock are plastic clays with a low degree of sensitivity to remoulding. Dense sand both above and below water level is stable; while dry loose sand is quite sensitive to seismic disturbance. Saturated loose sand is

hazardous when subjected to a shock as it may undergo instantaneous liquefaction and react as a flow slide.

Observations of various slope and embankment materials indicate the following tentative conclusions: (1, 69)

a. Very steep slopes of weak, fractured, and brittle rocks or unsaturated loess are vulnerable to seismic shocks. The mechanics of failure are opening of tension cracks, overloading the toe of the slope, and liquefaction phenomenon.

b. Loose, saturated sand or mountain detritus may be liquefied by shocks with sudden collapse and flow slides. Cuts through mountain detritus on smooth and sloping rock strata are also vulnerable, particularly if ground water travels along the rock surface.

c. Dry and relatively loose cohesionless material sloping at its angle of repose will be slightly flattened by shallow sloughing due to transient shocks.

d. Well compacted cohesionless embankments, either dry or saturated, on a reasonably flat slope are considered safe during earthquake shocks.

e. Many soft, nonplastic or slightly plastic silts, known as mud, are very sensitive and may liquefy during seismic shock.

f. Relatively flat slopes in clay and clay deposits underlying embankments are considered safe during earthquake shock.

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g. A clay foundation beneath a loose sand embankment may be overstressed if transient shocks cause a significant loss of strength within the embankment.

3.2 - Engineering Properties

3.21 General

Understanding the fundamental engineering properties of soil as influenced by seismic vibrations is basic to engineering design. It is necessary to know the seismic properties of soil both as a foundation material and as a construction material. Accordingly, engineers and scientists in areas affected by frequent earthquake damage have undertaken studies to acquire this knowledge. Model and specimen tests, application of the principles of engineering mechanics, and detailed analysis of field observations have been utilized to gain this knowledge. The resulting data have, for the most part, been correlated to the engineering properties of the material in the static condition. The changes in properties are thereby understood relative to the easily acquired, well understood, and usable engineering properties. The results have been checked by the other approaches to establish reliability of the data.

3.22 Granular Material

The engineering properties of fine granular materials, sand and silt, are adversely affected by seismic vibrations. As described in the first part of this chapter, structures

founded on certain granular soils suffer twice as much damage as those founded on harder, compacted soils. Similarly, damage associated with differential settlement and subsidence is common during earthquakes.

The shearing strength, or yield value, of sand diminishes with the increase of acceleration of vibration. A series of experiments performed in Japan and reported by Okamoto (77) demonstrates this effect. A shear box was filled with sandy soil and vibrated on a shaking table. Direct shear tests were performed and shearing strengths were measured under varying vibration frequencies and accelerations. The plot of typical test data is shown on Plate XI to provide a qualitative illustration. The experiments indicated that during vibration the shearing strength of soil could be correlated more reliably with acceleration than any other factor.

It should be noted that at accelerations above some value between 0.3g and 0.8g, depending on the material, the shearing strength of sand is drastically reduced. In this state the sand flows like a liquid and develops hydrostatic pressures. As a result, it is sometimes referred to as being in a state of "liquefaction". The value of acceleration at which this phenomenon occurs is dependent on both the relative density and degree of saturation of the sand. Loose sand and saturated sand are liquified by lower accelerations than are dense dry sand. (77)

While the liquefaction of dry sand has been observed in

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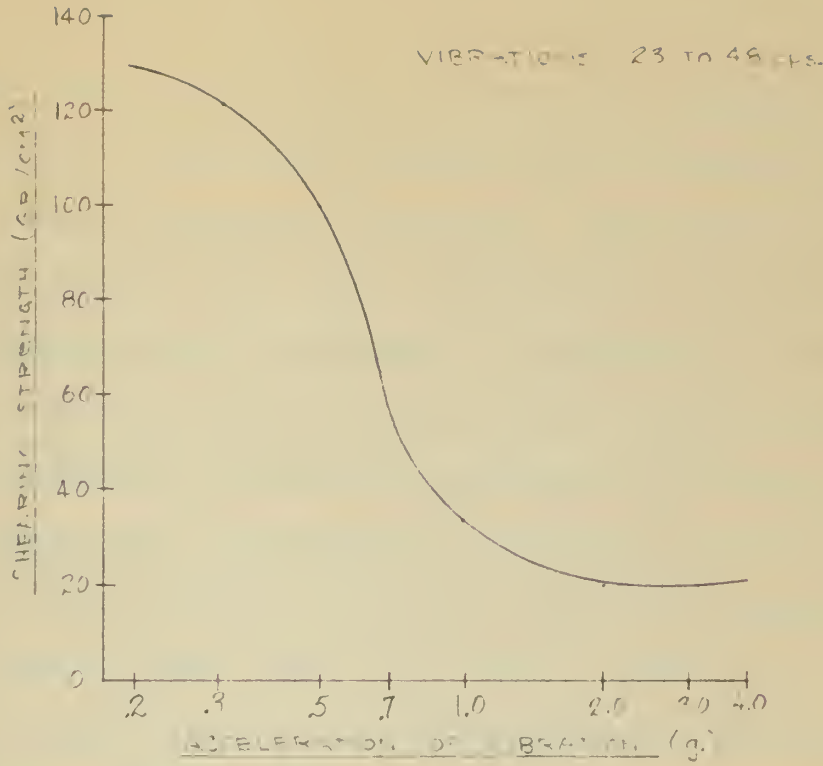
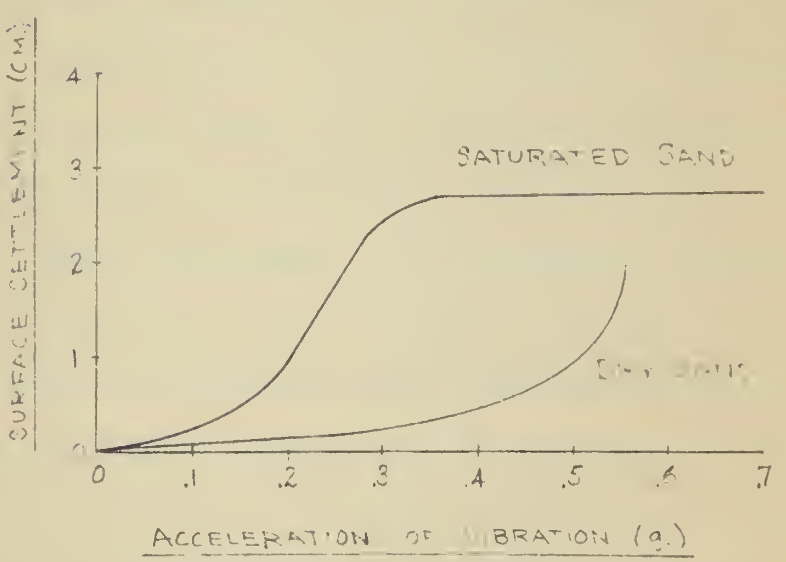


SHEARING STRENGTH OF SAND v. ACCELERATION OF VIBRATION



SETTLEMENT OF SATURATED & DRY SAND

PLATE XI

the laboratory and on the surface of the ground adjacent to vibrating equipment, the writer doubts that it occurs to any great depth during earthquake vibrations. The load of overlying material and the fact that vibrations are not uniform with depth prohibits such occurrence. Liquefaction of the surface of dry granular material does help to explain excessive settlement and damage to shallow foundations and utility lines.

On the other hand, liquefaction of saturated granular material has frequently been observed in the field during earthquakes, and at other times under ideal conditions, as well as in the laboratory. This phenomenon is of particular interest to the engineer working with waterfront structures as much of the subgrade in that environment is saturated.

Liquefaction of saturated granular material occurs when a vibration, or shock, disturbs the intergranular structure, or condition of happenstance balance of grain on adjoining grain, and these grains under the influence of gravity seek new positions of static equilibrium. The new position of each grain is lower within the mass, and the mass as a whole becomes denser. This denser mass has a reduced void ratio, and a portion of the water of saturation must permeate through the newly reduced volume of voids and flow out of the mass. When the permeability of the granular material is such that the water cannot be expelled fast enough, excess pore pressures rapidly build up which hold grain away from grain. Without the interlock between grains, the mass becomes a dense fluid of sand

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and water. Earthquake vibrations can induce a sequence of these reactions which keep the saturated granular material in a state of liquefaction for a short duration until the water is squeezed out and a denser equilibrium achieved. Subsequent vibrations of greater intensity require even greater densities for stability, and transient periods of liquefaction occur during this process. Liquefaction is aided by the bouyant effect of the water wherein the weight which must be overcome by vibration to dislodge the material's intergranular structure and to maintain its suspension, is reduced by the unit weight of the water.

The intensity of acceleration required to induce liquefaction increases with the relative density of the granular material. Similarly, the more permeable a material, the greater is its resistance to liquefaction. Gravels do not experience liquefaction. (58) Silts, on the other hand, are particularly susceptible. As extensive damage cannot be avoided during liquefaction of saturated granular materials, engineering effort to avoid its use is recommended. Where its use is absolutely required, it should be made resistant to all but the higher accelerations by increasing its relative density and its permeability.

Spectacular subsidence of the ground has been observed after destructive earthquakes due to the liquefaction of saturated granular soil. As pointed out earlier, a form of liquefaction occurs at the surface of dry granular material.

However, greater accelerations are required to induce aggravated settlements. Also, without the benefit of excessive pore pressure and bouyancy effects of the water of saturation, equilibrium is achieved at lesser relative densities and the settlement is not nearly as spectacular. The relationship between settlements in saturated and dry sands at various accelerations is illustrated by Plate XI. (58) During destructive earthquakes relatively few areas are subjected to accelerations falling within the steep zone of the dry sand curve while extensive areas fall within the steep zone of the saturated sand curve.

3.23 Clay

Each shock during an earthquake will cause an increase in shear stress in cohesive soil and thereby increase the likelihood of shear deformations. At the same time, however, the soil will mobilize shear strength during earthquake shocks. The stability of the soil is dependent on the composite results of these two effects. Laboratory tests have been undertaken by Seed (86) to ascertain the shear strength of a compacted silty clay and a medium sensitive undisturbed clay during earthquake vibration.

It was recognized through the analogy of transient load tests that when a soil is loaded to failure in a period of time corresponding to an earthquake pulse, the strength is greater than that developed under static loading conditions.

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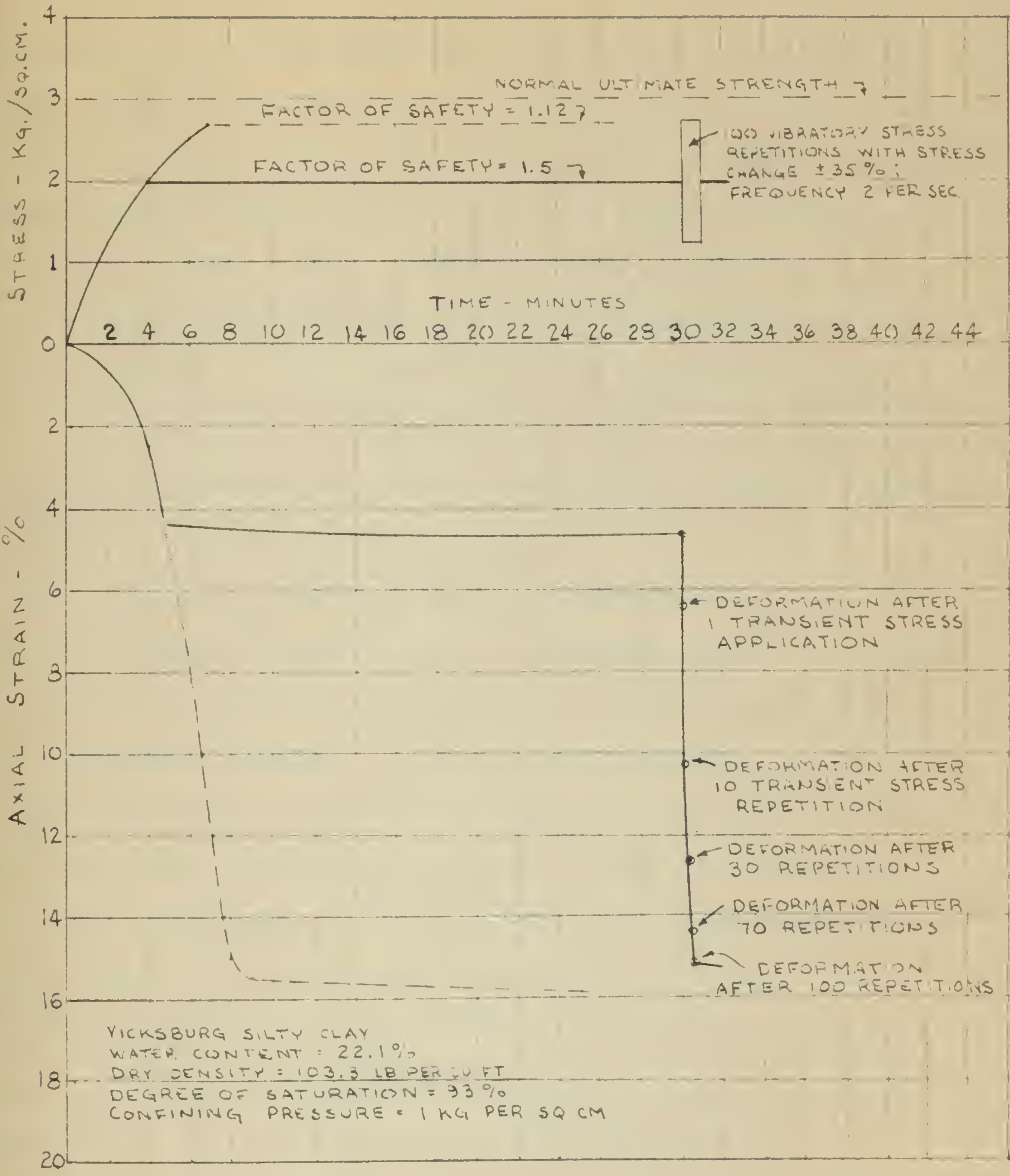
However, this reaction is applicable to only the first transient load of an earthquake. Subsequent reversals and reapplications of the stress may deform the soil sufficiently either to remold the soil enough to cause a reduction in strength or to cause excessive shear displacement even though failure does not occur. It was concluded that valid tests results simulating earthquake effects must be based on a number and frequency of pulses known to occur in nature and of such intensity as not to cause failure. A review of seismograph records indicated as many as 50 to 60 shocks of significant intensity could be anticipated during an earthquake, including its aftershocks. Stress pulses applied at the rate of 2 cycles per second were also indicated.

To reflect actual conditions the specimens were subjected to sustained stresses, in equilibrium, lower than that required to cause failure to simulate the dead load on the soil structure at the time of the earthquake. Unconsolidated-undrained triaxial compression tests were performed utilizing a special transient loading device.

A typical example of the results of this test prcedure on compacted silty clay is illustrated by Plate XII. A speciman is loaded to 66 per cent of its ultimate strength, corresponding to a factor of safety of 1.5 by conventional loading procedure and allowed to come to equilibrium. A series of 100 transient stress pulses corresponding to $\pm$35 per cent of the initial stress is then applied. The stress-deformation-time



STRESS & DEFORMATION OF SILTY CLAY WITH TIME DURING EARTHQUAKE LOADING

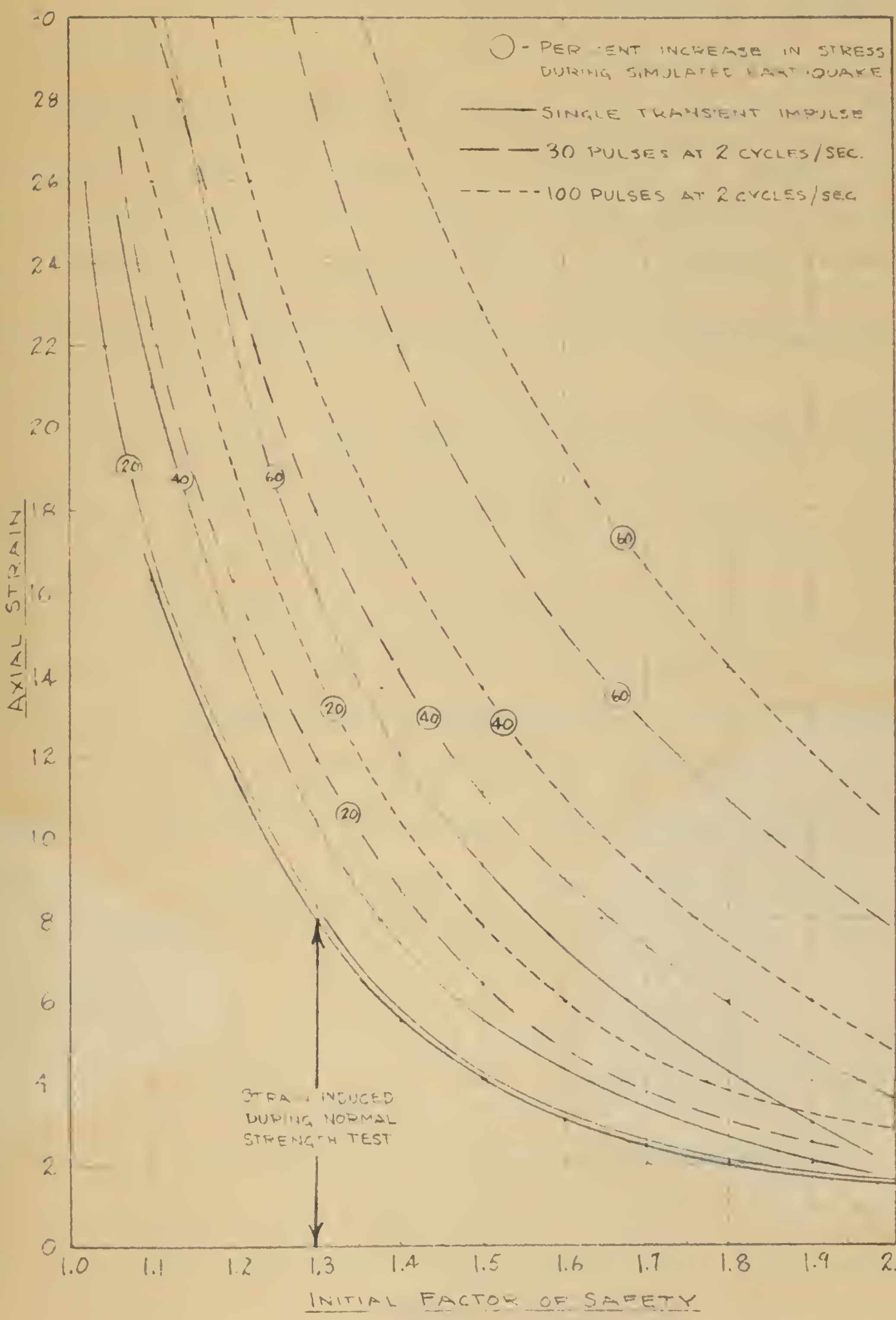
(FROM SEED REFERENCE 86)

PLATE XII

relationship is plotted along with the results of a similar specimen loaded conventially to a satety factor of 1.12. It should be noted that while the maximum applied stress, including the earthquake stress, was less than the normal stress of the soil, and while the lowest safety factor attained was 1.12 based on normal strength and 1.52 based on transient strength, the soil deformed 11 per cent during the transient loading cycle. This amount of deformation can rarely be accommodated in engineering structures. The strain after different numbers of transient loads is readily apparent.

The effect on identical samples subjected to variations in initial stress corresponding to different factors of safety and to various transient intensities and durations of earthquake stress was studied. Plate XIII illustrates the results of these tests for a series of a single, of 30, and of 100 transient pulses corresponding to 20, 40 and 60 per cent of the initial sustained stress. It should be noted that while a single pulse equal to 20 per cent of the initial stress causes negligible deformation, even at a factor of safety as low as 1.1, series of 30 and 100 transient pulses induce deformations of up to 6 and 10 per cent respectively.

It can readily be seen that the principal effect of earthquake stress application on compacted silty clay is large magnitude deformations which do not cause clear cut failure. However, soil samples exhibiting large deformations are generally considered to have failed at 20 to 25 per cent strain.



SOIL DEFORMATIONS BY VARIOUS COMBINATIONS OF SUSTAINED & VIBRATORY STRESSES

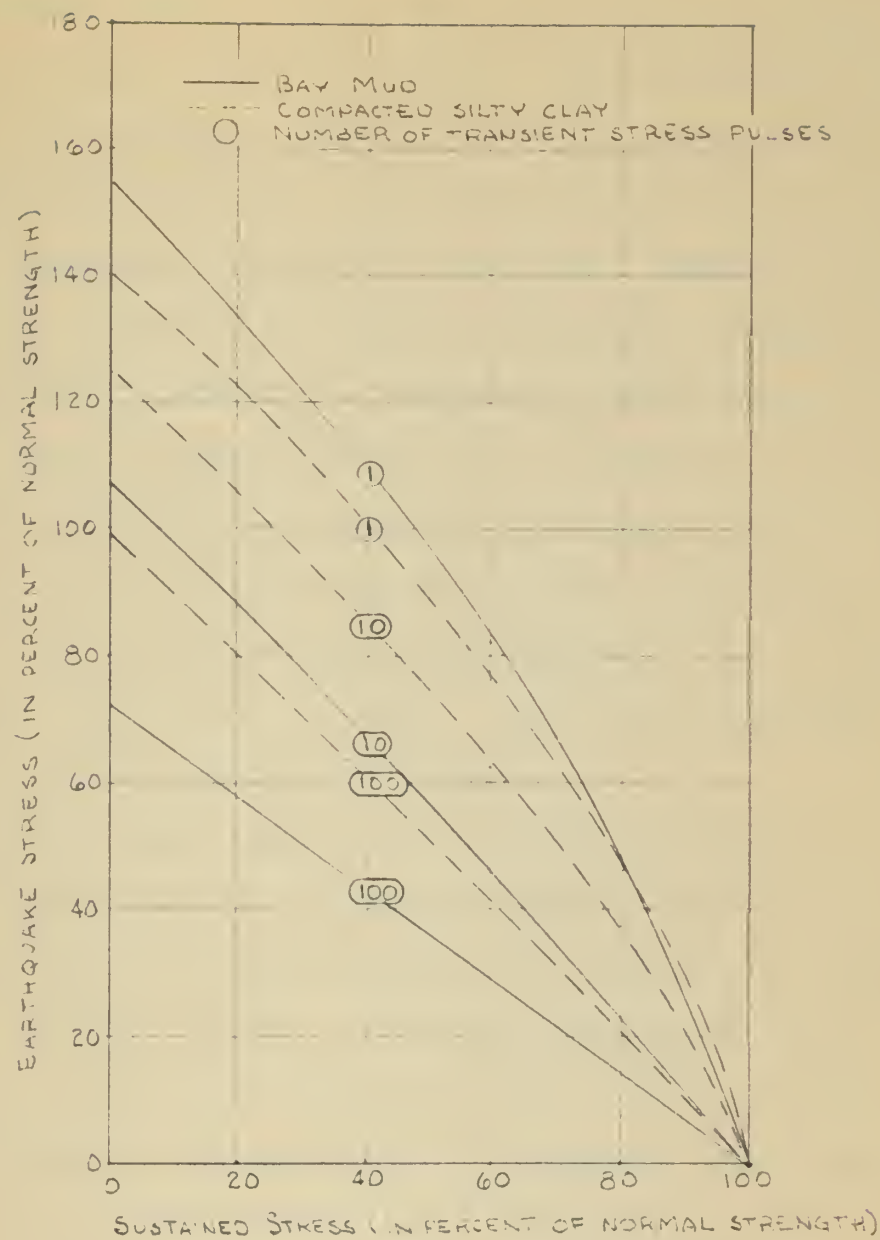
(FROM SEED REFERENCE 86)

PLATE XIII

It may be noted on Plate XIII that for the normal strength test the maximum stress occurs at an axial strain of about 25 per cent. By plotting the combination of initial sustained stress and transient stress increase, as percentages of normal ultimate strength, for selected durations of transient pulses which result in 25 per cent deformation, as shown in Plate XIV, ultimate strength data becomes available for all combinations of earthquake loading.

Plate XIV clearly shows the combination of initial stress and earthquake stress intensities which cause failure. It may be noted that for single transient pulses the combination of initial and earthquake stress intensity causing failure is about 135 per cent of the normal stress to induce failure. Similarly, a combination of about 120 per cent normal strength is required for 10 pulses and a combination of about 100 per cent normal strength is required for 100 pulses. This data reflects a factor of safety of 1.0 and an increased factor of safety may be utilized by increasing the loads correspondingly before using the plotted data.

Sensitive undisturbed clays perform somewhat differently. Unlike the unsensitive compacted clay which does not change its strength characteristics appreciably with deformation, sensitive undisturbed clays experience an increase in pore water pressure and loss of strength with deformation. They are apt to display greater strength under a single transient stress pulse, and will assuredly have less strength under repeated



COMBINATIONS OF SUSTAINED AND VIBRATORY STRESS CAUSING FAILURE IN CLAY
(FROM SEED, REFERENCE 86)

PLATE XIV

transient stress pulses. Plate XIV illustrates the stress conditions causing failure to sensitive undisturbed clay in comparison with compacted silty clay. It may be noted that only 10 transient pulses are required to cause failure of a sensitive undisturbed clay specimen when the combined initial and earthquake stress is 100 per cent of normal ultimate strength, while 100 transient pulses are required by the compacted silty clay under similar loading conditions. It should also be noted that sensitive undisturbed, saturated clays are subject to liquefaction, with corresponding hydrostatic properties as described in the section on granular materials, when earthquake pulses destroy its initial strength. This is an important consideration to the waterfront engineer.

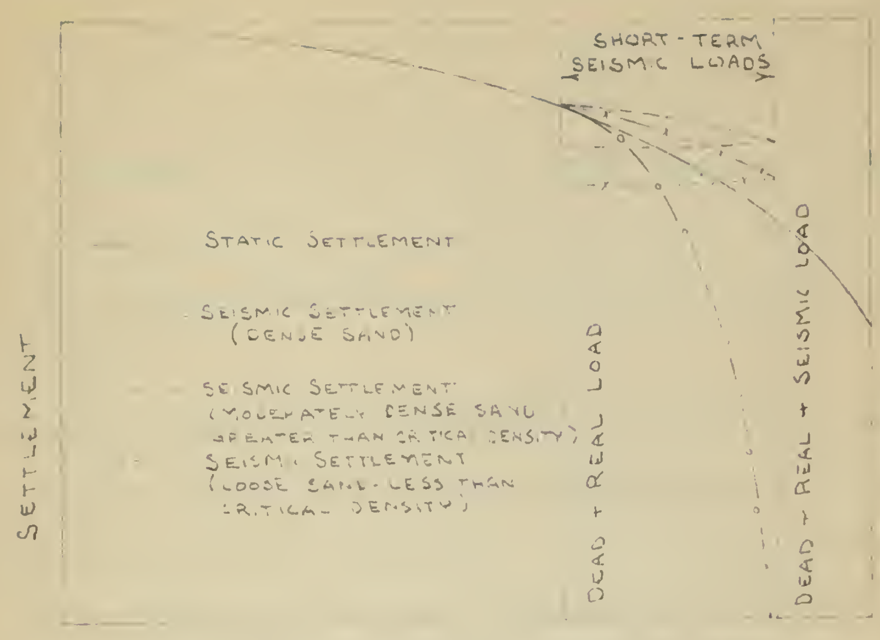
Accordingly, under simulated earthquake loading the strength values of clay do not differ greatly from the ultimate strength determined by normal methods even though the strengths of clay vary greatly by type. Seed has demonstrated that a total stress of 100 to 120 per cent of the normal strength would be required to induce failure of compacted silty clay under earthquake loading conditions. Similarly, a total stress of 80 to 100 per cent of the normal strength would be required to induce failure of a medium sensitive undisturbed clay. Therefore, engineering design taking into consideration both sustained and earthquake stresses and providing a factor of safety of 1.0 for compacted silty clay and 1.15 for undisturbed clay, predicated on the normal ultimate strength of the soil,

should assure against failure during an earthquake. Large shear deformation must be anticipated when the design is based on the prevention of failure while utilizing the full strength of the soil. The deformations can be reduced in relation to the stress-strain characteristics of the soil by using proportionally greater factors of safety for the combined sustained and earthquake stress.

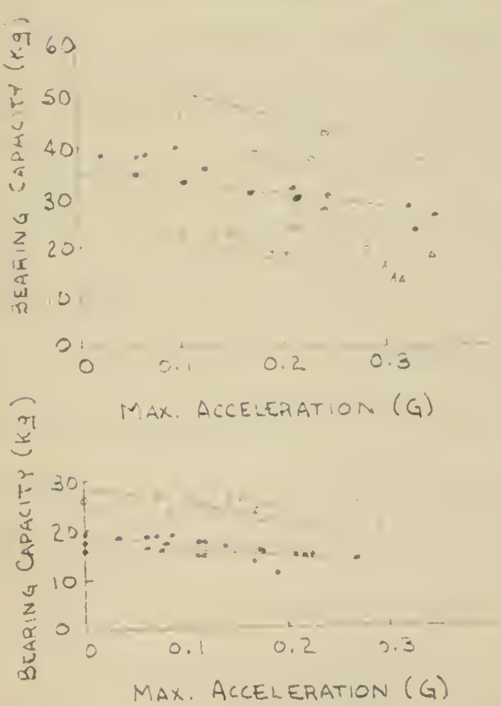
3.24 Bearing on Sand

Structural failure and damage during earthquake is often attributable to excessive foundation deformation. Therefore, the control of earthquake induced differential movement within acceptable limits is an important engineering consideration. The structural analysis of the superstructure should include a check to make sure specified limits of differential settlement of foundations can be accommodated.

A typical settlement diagram for a spread footing on sandy soil due to a sustained load and an earthquake load is illustrated by Plate XV. (62) The settlement occurs quite rapidly due to the fast drainage characteristic of sandy soil. Its magnitude is determined primarily by the structure of the soil. The settlements caused by seismic disturbances are relatively small for dense sandy soils and increase with the decreasing density of the soil. When the density of saturated sand is less than a certain critical value, the soil structure collapses during earthquake through liquefaction and large



SETTLEMENT DIAGRAM FOR FOOTING
IN SANDY SOIL
(FROM MOORE, REFERENCE 62)



BEARING CAPACITY OF DRY & SATURATED SAND
VS.
ACCELERATION OF VIBRATION
(FROM OKAMOTO REFERENCE 77)

PLATE XV

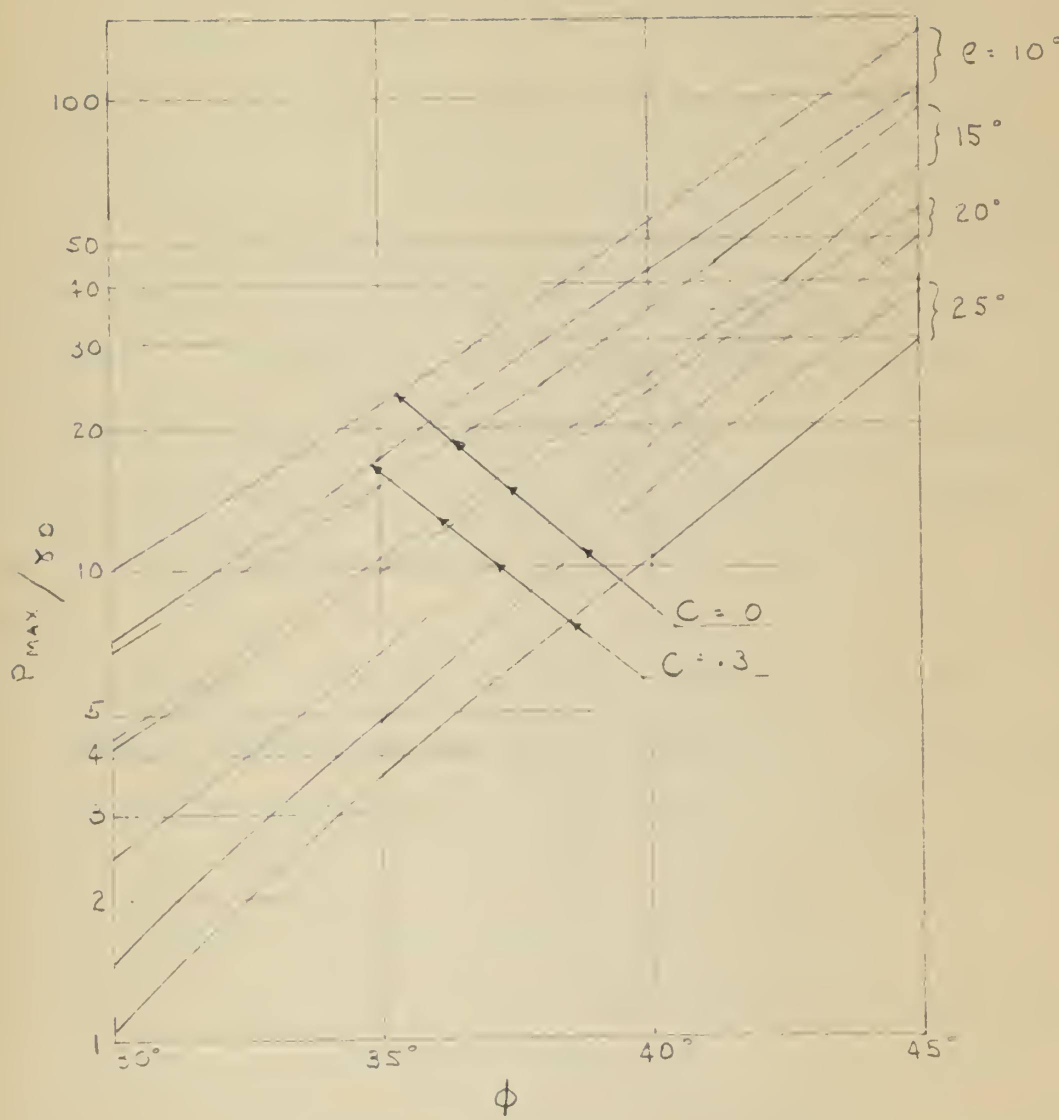
order settlement occurs. Structures founded on dense sand formations do not generally suffer damage during earthquakes, although short duration loads greater than those incurred by permanent loading are experienced. Accordingly, an increase in bearing pressures of 20 to 30 per cent over the allowable bearing pressure of dense sand may be permitted for short time seismic loads without sacrificing an adequate factor of safety against failure.

To determine the bearing capacity of sandy soil beneath obliquely loaded foundations during earthquakes, a test program was carried out at Tokyo University. (77) Frequencies, accelerations, and amplitudes utilized correspond to actual earthquake experience. The relationship between acceleration and bearing capacity for both dry sand and saturated sand is shown on Plate XV. It was impossible to determine the exact bearing of dry sand when the acceleration reached about 0.3g due to the violent surface motion. It was found that bearing capacity diminishes linearly with acceleration up to the limit of measurable intensities, and that this was a very reliable relationship. The results of a full series of oblique loading tests for angles of internal friction ranging from 30 to 45 degrees are illustrated in Plate XVI. (77) The elements which comprise the ordinate $p_{max}/\gamma_m b$ and other factors of this graph are:

p_{max} - ultimate bearing pressure of soil

γ_m - unit weight of soil

b - width of the footing.



BEARING CAPACITY OF SANDS
FOR INCLINED LOADS
(FROM OKAMOTO REFERENCE 77)

PLATE XVI

For saturated sand

δ'_m - bouyed unit weight of soil

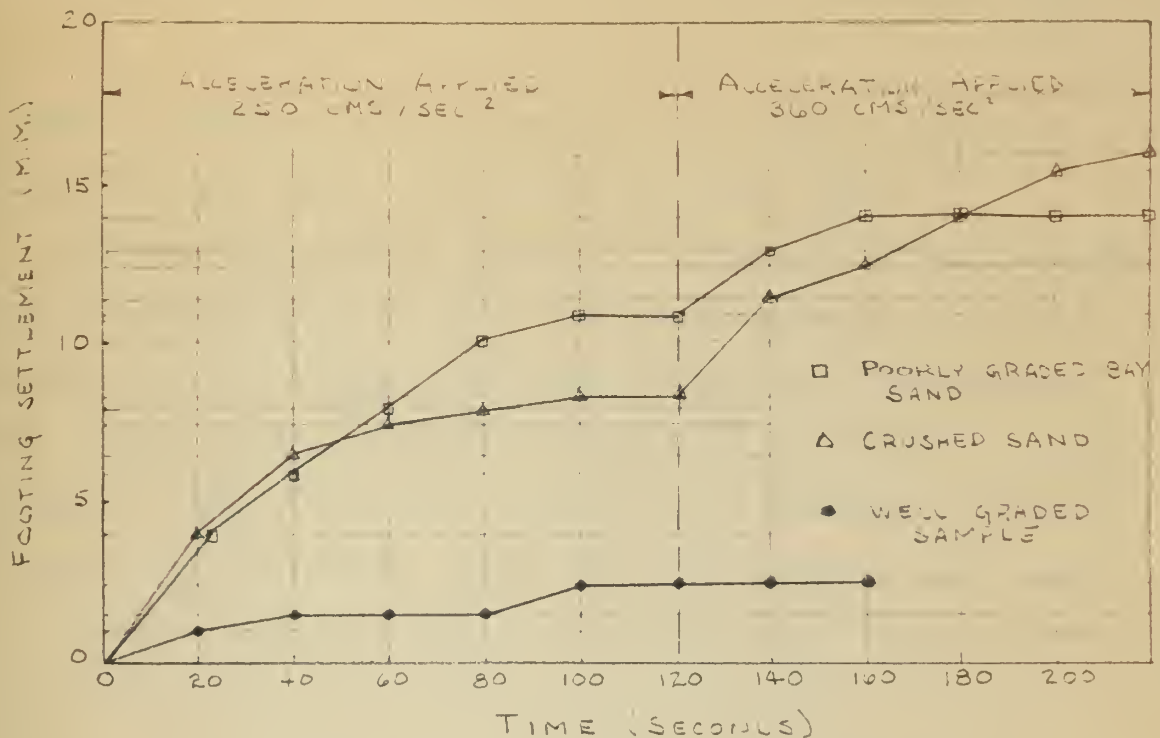
ϕ - internal angle of friction

C - seismic factor

ϵ - angle of loading with the vertical

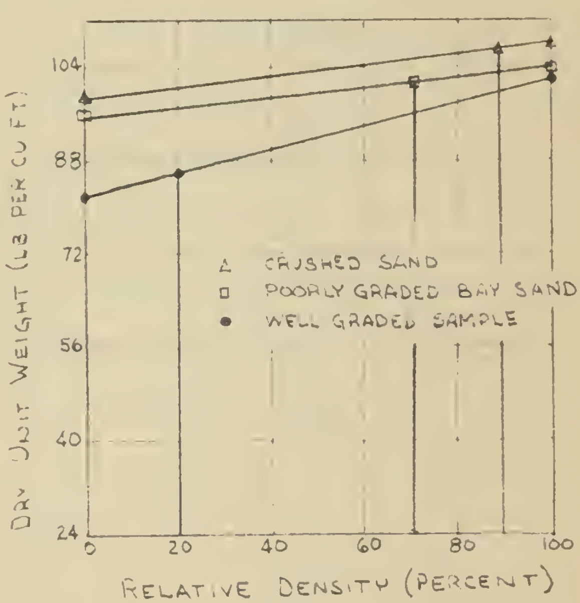
The relative density, or degree of compaction, of sandy soil may be beneficially utilized as the criteria for earthquake resistance to settlement of fill in lieu of the shear strength. Earthquake vibration increases the relative density of loose granular material according to its natural characteristics. Excessive deformation of earth structures and fills constructed of sandy soil may be avoided by utilizing a degree of compaction, or relative density, greater than that induced by vibrations.

The settlement of a model footing on natural, in situ, sands of various composition and the corresponding relationship of vibration induced compaction to the range of possible relative densities is shown by Plate XVII. (65) It should be noted that a uniform, or poorly graded, sand settles 3 to 4 times as much as a well graded sand, while vibratory compaction is not as effective on well graded sand as it is on a uniform sand. The 0 per cent relative density was obtained by pouring the material from a funnel held at a low and uniform height while the 100 per cent relative density is that produced by the Modified Proctor Compaction method. This data is based on a sound sand with a good specific gravity. It was found that



FOOTING SETTLEMENT IN SAND DUE TO VIBRATION

(FROM MURPHY, REFERENCE 65)



RELATIVE DENSITY IN SAND DUE TO VIBRATION

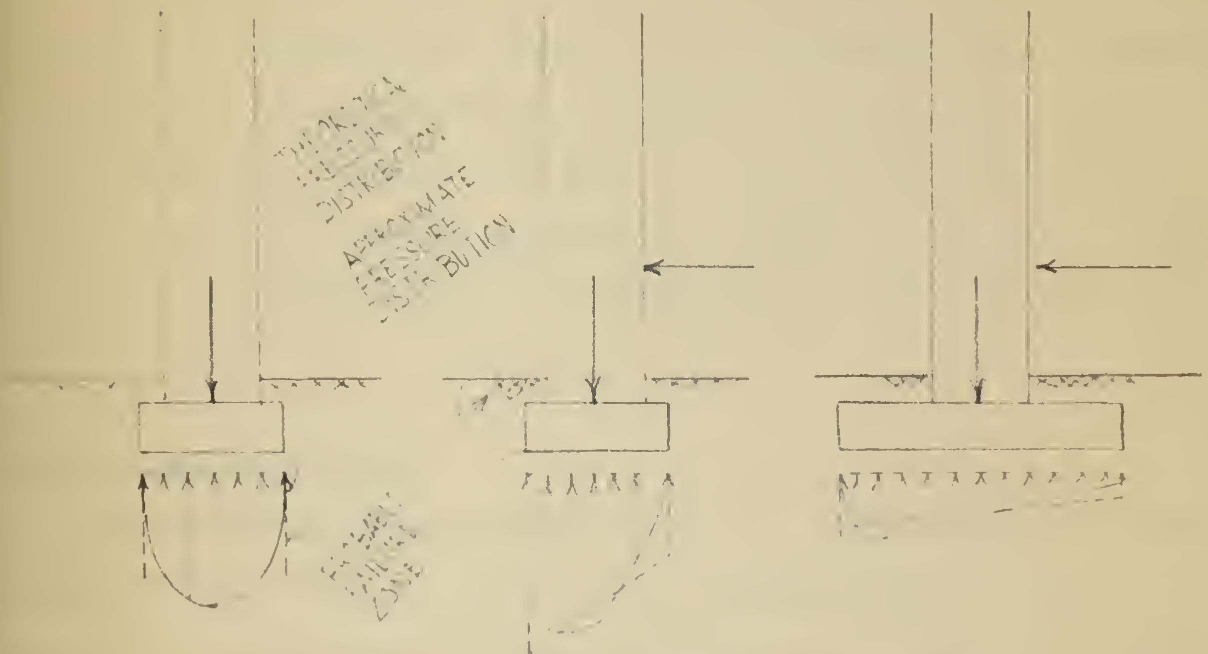
(FROM MURPHY, REFERENCE 65)

PLATE XVII

the above described relationships do not hold if there is over 8 to 10 per cent of silt or clay in the sand.

A frequent problem of structural footings bearing on sand, and to a lesser degree on clay, is the practice of eccentric loading. Due to proximity of adjacent structure or property lines, a bearing wall or pier is placed on the edge of a spread footing. In such cases the resultant soil pressure, for both the static and earthquake loads, will be eccentric with that of the bearing wall or column. A bending moment will be produced for which these members were not designed. Failures, slippage of inadequately tied elements, and structural rotations occur during earthquakes. (13)

Concentrically loaded footings can also present trouble during earthquake loading. The left figure of Plate XVIII illustrates the theoretical and approximate pressure distributions for a rigid footing on sandy soil. The highest pressure occurs under the center of the footing and the material is confined by the overburden and friction forces developed by the base of the footing. When designed on the basis of average pressure as indicated by the approximate pressure diagram, confinement is adequate to prevent foundation soil overstress. When the superstructure is subjected to lateral earthquake loads, the redistribution of the theoretical and approximate soil pressures is as shown by the center figure. Concentration of the load at the edge, or toe, of the footing often exceeds the confinement capacity of the overburden and overstressing



CONCENTRIC LOADING (STATIC CONDITIONS) | ECCENTRIC LOADING (SEISMIC CONDITIONS) ASSUMED | ECCENTRIC LOADING (SEISMIC CONDITIONS) RECOMMENDED

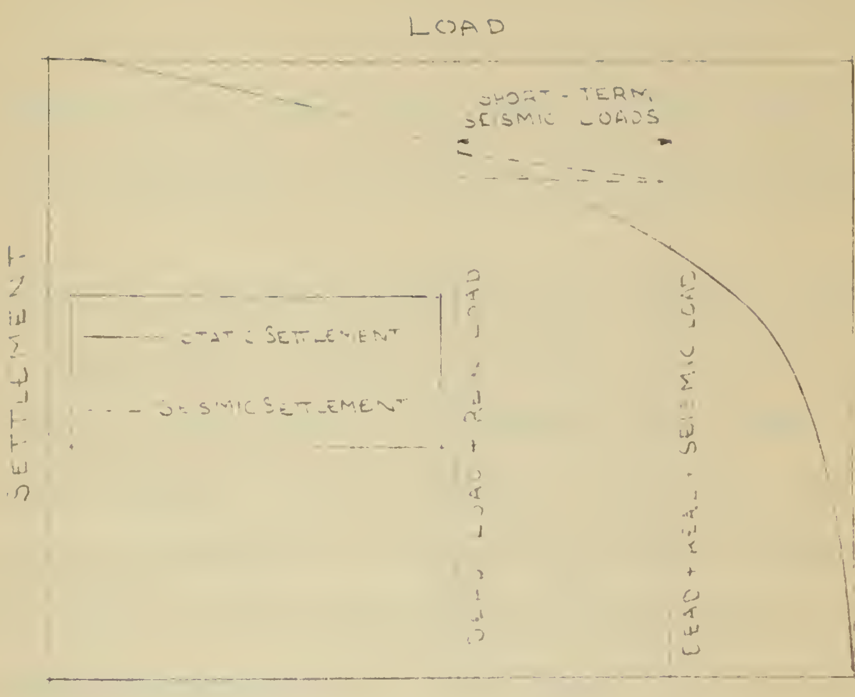
PRESSURE DISTRIBUTIONS
(RIGID FOUNDATIONS ON SANDY SOILS)
(FROM MOORE REFERENCE 62)

of the foundation soil occurs. The failure is in the form of a plastic flow of overstressed sand at the edge and toe of the footing. Progressive settlement with earthquake oscillation and the resulting rotation of the footing causes foundation failure and corresponding damage to the superstructure. Design should limit the maximum edge pressure of individual spread foundation with freedom of motion to 70 or 80 per cent of the normal allowable bearing pressure for typical sandy soil. The recommended seismic pressure distribution is shown by the right figure. (62)

3.25 Bearing on Clay

Clays are generally considered to be less desirable foundation materials than sand due to their compressibility, consolidation, and plastic creep characteristics. However, clays react better to short duration loads than do sands. As a result of the intergranular structure of clay, it cannot adjust to short duration changes in load despite the high pore pressures which may develop during an earthquake shock. The permeability characteristics of clayey soils cause a time lag for consolidation which prevents significant deflections from occurring.

A typical settlement diagram for a spread footing on clayey soil is illustrated by Plate XIX. Unlike sandy soil, settlement under the permanently applied static load requires considerable time to achieve a condition of equilibrium, and



LOAD - SETTLEMENT PATTERNS
(SPREAD FOOTINGS ON CLAYEY SOILS)
(FROM MOORE REFERENCE (62)

PLATE XIX

static loads in excess of the design range causes continuous yielding and plastic failure, as shown. However, short duration seismic loading would not cause appreciable settlement. Accordingly, foundations on clayey soil may generally be designed for comparatively high seismic loads without excessive settlements. No adjustment in allowable bearing pressure is required.

Construction on reclaimed marshland is a special problem of waterfront engineering in seismic regions. Piles are often required to support structures and equipment where weak and compressible clay stratas are encountered. Two cases must be investigated for the transfer of lateral force to competent material. The first is the assumption that the lateral force acts on the superstructure and is transferred to the ground by means of the bending resistance of the piling. The second is the assumption that lateral force is transmitted by the soft clay to the piles which are deflected between the relatively rigid pile tips and caps. The magnitude of these deflections is defined by recorded ground motion data which indicates shear waves of large amplitude and relatively slow in velocity in soft saturated clay. It is impractical to design the piling foundation to resist movements; however, the foundation should be tied together to vibrate as a unit. The pile caps and grade beams should be designed so that the adjacent soil offers resistance within its permissible lateral earth pressure characteristics. Batter piles should generally be avoided even

though they offer effective resistance to lateral loads. The drag down forces produced within a consolidating strata of clay under residual load may introduce rotations into pile caps and induce strains sufficient to render batter piles useless for lateral loads. (62)

Laboratory tests on gelatine have shown the superiorty of structural raft foundations over pile foundations for bearing on weak and compressible clays. It was found that the raft foundations must have adequate structural strength to act as a unit, and that notches and offsets along its bottom surface require smoothing out for most effective resistance to seismic loads. (65)

Allowable bearing pressure is dependent upon the general characteristics of the soil. The allowable bearing pressures for seismic loads may be increased from 25 to 100 per cent of those applicable to the sustained static loads for typical clayey soils. However, the sensitivity of the clay is one characteristic which must be given particular attention when determining the seismic load allowable bearing pressure. Experience and judgement must augment the available laboratory data. (56)

Frequently, the earthquake load analysis of a superstructure will indicate a distribution of loads among footings differing substantially from the static load distribution. The selection of footing size on clayey soil must be based on the static load pursuant to Teraghis' Theory of Consolidation.

An attempt to size the footings for the maximum load which might ever be experienced, including the seismic load, would result in differential settlements between footings. As a result of the time-rate of consolidation characteristic of clay, each footing would achieve equilibrium for its respective bearing pressure on the basis of the sustained load. Unacceptable deformations to the superstructure and overstressing of members might very well result from such practice. A foundation tied together by grade beams largely eliminates this type problem, and the record of this type of foundation in earthquakes is excellent.

Also, the effect of high toe pressures under foundations subjected to eccentric earthquake loading is not as severe for clayey soils as it is for sandy soil. This is because clay derives its strength through intermolecular attraction, or cohesion, instead of internal friction, like sandy soil, which depends upon its confining pressure. No reduction in allowable bearing pressures is generally required for either concentration or eccentric earthquake loading conditions. Refer to Plate XV for the analogous conditions for sandy soils. (62)

3.26 Lateral Earth Pressure

The deformation and movement of retaining walls have decisive influence on the magnitude and distribution of lateral earth pressure behind the walls. Lateral earth pressure might take any value between the active and passive states depending

on these conditions. During an earthquake the earth pressure is also a function of backfill conditions, foundation charac- teristics, and the character of the vibrations. To determine the nature and magnitude of earth pressure during earthquake, Ishii has carried out a series of model tests for fixed and movable walls using sand backfill. (48)

During vibration the sand behind a restraining wall goes through three distinct phases. The first phase occurs up to 0.5g where the sand particles have the same motion as the vibrations and settlement increases linearly with the acceler- ation. Lateral earth pressure increases somewhat, and its dis- tribution is nearly hydrostatic. The second phase occurs between about 0.5 to 0.8g during which time the movement of sand above a certain boundry becomes critical. It goes out of phase with the driving vibration and undergoes accelerations 5 to 10 times greater. Settlement becomes considerable and lateral earth pressure increases rapidly. The motion above the boundry is unsteady, but becomes steady after the density of the sand increases due to settlement. The rate of settle- ment then becomes small and the acceleration of the sand de- creases to that of the driving vibration. The third phase occurs when acceleration is again increased and the critical state reappears.

Lateral earth pressure due to vibration is made up of a dynamic increment and a static increment. The static increment adds to the at rest active lateral earth pressure to make up

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the residual lateral earth pressure which remains after the vibration ceases. It is the dominant contributor to the total earth pressure during vibration. The lateral earth pressure due to vibration increases with the intensity of vibration and continues to increase after the vibration reaches a steady state. During the critical phase of the vibrating sand, the magnitude of the dynamic increment, the rate of increase in residual lateral earth pressure, and the rate of settlement are substantial. After the critical phase becomes steady, due to the increase in sand density, the dynamic increment and rate of increase in residual lateral earth pressure become small. The violence of the critical phase is dependent on the density of the sand backfill in addition to the intensity of vibration.

An important finding is the existance of a phase difference between the driving vibration and the dynamic increment of the lateral earth pressure due to the vibration. This phase difference is unimportant for fixed walls, but reaches one-half period for movable walls. The dynamic increment is a minimum at a certain wall displacement and increases with the increase or decrease of wall displacement. Its diagram of lateral pressure is bowshaped. At the same time the residual lateral earth pressure increases with displacement of the wall and reaches a substantial value when the wall displacement exceeds a certain amount. Its diagram of lateral pressure is hydrostatic. Predicated on design pressures being those which are

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effective when the vibrating wall reaches its farthest out position, it was found that the design pressure was near the maximum earth pressure recorded for rigid walls and equal to the minimum earth pressure recorded for movable walls for any particular condition of vibration.

The total lateral earth pressure in vibration, that is, the at rest active lateral earth pressure plus the static increment which adds thereto with vibration to produce the residual lateral earth pressure, plus the dynamic increment of lateral earth pressure, when the vibrating wall reaches its farthest out position, is equal to or lower than the earth pressure determined by the Mononobe-Okave formula, as given in section 4.14, for seismic coefficient of lateral earth pressure. The residual lateral earth pressure for movable walls is about equal to that determined by the Mononobe-Okabe formula; while it is 10 to 15 per cent smaller than determined for rigid walls. At the same time, the dynamic increment of lateral earth pressure for rigid walls is about equal to the Mononobe-Okabe value; while it is about half that value for movable walls. The phase difference of the dynamic increment when the vibrating wall reaches its farthest out position causes its addition to the residual lateral earth pressure for rigid walls and its subtraction from the residual lateral earth pressure for movable walls. As a result of the dominant character of the residual lateral earth pressure in seismic loading, the resultant pressure may be considered to be hydrostatic for sand backfill. (48)

The validity of the Mononobe-Okabe formula has also been checked experimentally by Jacobsen at Stanford University for the U. S. Tennessee Valley Authority. He achieved very good experimental conformity with the theoretical values up to accelerations of 0.4g. This range is applicable to even the greatest accelerations utilized in engineering design. For higher accelerations the theoretical pressures were progressively greater than his experimental values. Also of general interest to the subject of earthquake engineering is that Jacobsen verified, with striking conformity, the Westergaard theory of dynamic water pressure during his series of TVA tests. See Plate XX. (34)

Chapter 4

WATERFRONT EARTHQUAKE ENGINEERING

4.01 Introduction

The material presented in the first three chapters of this thesis has laid the groundwork for the earthquake design analysis of waterfront structures. The general nature of earthquake phenomenon was described along with those features of the science of seismology of particular interest to engineering. Field observations of major earthquakes, particularly at waterfront locations, were then described to identify those materials and techniques of construction which are most vulnerable to earthquake damage. Lastly, the seismic properties of soils were described as pretaining to both the geological effects of the site and the engineering properties of soil. This fourth chapter will undertake to apply this basic data to the problem of engineering design for seismic conditions at waterfront installations.

This chapter is divided into two parts. The first part describes the basic principles and techniques of earthquake design analysis. The second part discusses the principal types of waterfront structures under seismic conditions and recommends methods and considerations of design. The basic techniques described in the first part of this chapter are used in the second part to permit the analysis of the seismic loads in order to carry through intelligent engineering design.

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Waterfront structures must be analyzed for each of the conditions of loading which they will experience during their lifetime. The condition of earthquake loading is one of the most severe to be experienced by a structure. It will be the basis of determining the size of some of the structural elements of any installation. Accordingly, earthquake resistant design adds to the cost of the installation. Realizing that it is problematical whether the installation will ever be subjected to an earthquake loading condition, the amount of reserve structural strength is less than that for conventional design analysis. The object of earthquake design is to provide sufficient strength to resist earthquake force near the limit of material strength and to incur nominal damage.

Engineering judgement is an important ingredient of earthquake design. The designer must conceive his problem as one of movement instead of as a condition of static loading despite carrying out the seismic analysis using the analogies of static loads. Earthquake engineering is a rapidly developing science. Its practitioners must keep abreast of the developments and improvements which are being brought about by continued scientific investigation.

4.1 - Earthquake Design Principles

4.11 General

Engineering analysis for any condition of loading requires techniques for determining the magnitude of loading and the

distribution of these loads. This first part of Chapter 4 presents techniques of design analysis unique to seismic loading conditions. These techniques identify the magnitude of force and its mode of application due to seismic motion by static load analogy. The balance of analysis is carried out according to the general principles of engineering design.

4.12 Seismic Factor

The majority of civil engineering structures, including waterfront and earth structures, are designed for earthquake resistance on the basis of the seismic factor method. The weight of the structure, including the dead load and some reasonable portion of the live load, is multiplied by the seismic factor to define the horizontal earthquake force. This force is assumed to act statically through the center of gravity of the structure's mass. The stability of the structure is analyzed according to the principles of statics for the aggregate of the gravity loads and earthquake forces. Refer to Chapter 1 for the basic philosophy of the seismic factor method of representing earthquake force.

The extent of earthquake resistance built into a structure and the resulting increase in total cost are directly related to the value of the seismic factor. Accordingly, the selection of the seismic factor is probably the most important decision of earthquake design. The following factors must be considered in arriving at this value:

a. Regional probability of the occurrence of destructive earthquakes

b. Regional probability of the intensity of destructive earthquakes

c. Character and distance of all known faults

d. Type and condition of foundation soil

e. Structural character of proposed structure

f. Economic or military importance of proposed structure

g. Extent of hazard to human safety

h. Requirements of local building codes

In view of the fact that earthquake damage is most extensive in areas of abrupt geological change and on unconsolidated grounds, water filled alluvium, and saturated fill, and as these conditions are usually satisfied by waterfront locations, substantial seismic factors are required. In general, the following seismic factors are recommended for waterfront structures and miscellaneous other structures located at waterfront installations, excepting structures with extensive super structures:

Low frequency areas	.10g
Medium frequency areas	.15g
High frequency areas	.20g

Refer to Plate III for area classification in the United States, and to Plate II and local records as the basis for determination outside the United States. (68) The above recommended values should be reviewed and modified in light of the factors

listed earlier in this section. Seismic factors of up to 0.3g are recommended for structures of particular economic or military importance in areas of high earthquake frequency. Recommendations for large superstructures is beyond the scope of this thesis.

4.13 Bouyed Seismic Factor

The seismic factor utilized for design in air should be increased in water to account for the effects of bouyancy. The apparent weight of a mass due to gravity is substantially reduced by the counter effect of bouyancy. At the same time, however, the forced vibration of earthquake motion remains unchanged. Consequently, the lateral force of earthquake loading does not remain a fixed ratio, defined by earthquake acceleration to acceleration of gravity, of the weight of the mass when that mass is submerged in water.

This increase in seismic factor for a soil mass under water is expressed as follows: (2, 50)

$$C' = \frac{\gamma_s}{\gamma_s - 1} C = \frac{\gamma_s}{\gamma'} C$$

and

$$C' = \frac{\gamma_s}{\gamma'} C = \frac{\dfrac{G + e}{1 + e}}{\dfrac{G - 1}{1 + e}} C = \frac{G + e}{G - 1} C$$

or

$$C' = \frac{\gamma_{ms}}{\gamma_{ms} - 64} C = \frac{\gamma_{ms}}{\gamma'_m} C$$

where

C - seismic factor

C' - bouyed seismic factor

γ_s - saturated unit weight (Metric)

γ' - bouyed unit weight (Metric

G - specific gravity

e - void ratio

γ_{ms} - saturated unit weight (English)

γ'_m - bouyed unit weight (English)

Similarly, the increased seismic factor for a solid mass, such as used in gravity quay walls and rigid type breakwaters, under water is expressed by the following equation:

$$C' = \frac{(62.4 \times G)}{(62.4 \times G) - 64} C$$

The equation for bouyed seismic factor for a soil mass is based on the simplified assumption that relative movement of soil particles and void water is prevented by the frictional resistance of the soil. The actual value for soil would fall somewhere within the limits expressed by the soil mass equation and the solid mass equation as applied to the specific gravity of soil. In view of the fact that dynamic water pressure would have to be added to the latter for void water, the equation for the soil mass is closest to the actual bouyed seismic factor value for the entire soil mass, and its use is recommended.

4.14 Lateral Earth Pressure

The lateral earth pressure of granular soil when subjected

to earthquake motion is expressed by the Mononobe-Okabe equation. As described in Chapter 3, model tests performed by both Jacobsen and Ishii produced results close to those of the equation within the range of accelerations used in design. The Mononobe-Okabe method was derived from the Coulomb theory by applying a static seismic force to the mass. The equation is an extension of the Mueller-Breslau equation for the computation of earth pressure coefficients by the addition of a mass equivalent to the static seismic force. The equation for a horizontal ground surface is as follows:

$$p_h = (p_s + \sum \gamma_m h)\, K_{s\_} \qquad (K_{sa} \text{ or } K_{sp} \text{ as applicable})$$

$$K_{sa} = \frac{\cos^2(\phi - \beta - \theta)}{\cos\theta \cos^2\beta \cos(\delta + \beta + \theta)} \times \left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta)}{\cos(\delta + \beta + \theta)\cos\beta}}\right]^2$$

$$K_{sp} = \frac{\cos^2(\phi + \beta - \theta)}{\cos\theta \cos^2\beta \cos(\delta + \beta - \theta)} \times \left[1 - \sqrt{\frac{\sin(\phi - \delta)\sin(\phi - \theta)}{\cos(\delta + \beta - \theta)\cos\beta}}\right]^2$$

where

p_h - intensity of lateral earth pressure

p_s - intensity of surcharge pressure

γ_m - unit weight of soil, use γ_m' bouyed unit weight below water level

h - depth below ground surface

ϕ - angle of internal friction of the soil

K_{sa} - coefficient of seismic active lateral earth pressure

K_{sp} - coefficient of seismic passive lateral earth pressure

β - angle between wall surface and the vertical

δ - angle of friction between soil and wall, use standard sign convention, general case: + active, - passive (101)

θ - angle of seismic factor, where $\theta = \tan^{-1} C$ or $\theta' = \tan^{-1} C'$

C - seismic factor

C' - bouyed seismic factor

To take into account the difference in unit weight and the seismic factor above and below the water level, the lateral earth pressure is computed at the water level and at the bottom of the wall. Bouyed unit weight of the soil is used below the water line in conjunction with coefficients of seismic earth pressure determined by use of θ' for bouyed seismic factor. These values and the value of the surcharge pressure are then connected by straight line to give the lateral earth pressure diagram. The increment of dynamic earth pressure may be determined by subtracting the conventional earth pressure utilizing active or passive coefficients for static loading from the seismic values utilizing the Mononobe-Okabe coefficients.

The above equations for coefficients of active and passive lateral earth pressure are based on the simplified assumption that failure occurs along plane surfaces. The values obtained for the active case are very little different from those which

result from the curved surface of actual failure. Its use is recommended as written. However, this assumption of plane surfaces of failure gives unduely high values for the passive case which, when used with reduced factors of safety, results in unsafe practice. Accordingly, in recognition that the Mononobe-Okabe equation accurately defines the increment of seismic lateral earth pressures over the Coulomb value, it is recommended that the value of K_p for curved surface of failure, as given by tables in Tschebotarioff's book, "Soil Mechanics, Foundations, and Earth Structures" (101), be multiplied by the ratio of K_{sp} with θ equal the applicable seismic value to the static K_p with θ equal zero, utilizing the above Mononobe-Okabe passive case equations for the ratio values. This is to say, the ratio of seismic coefficient of passive earth pressure to static coefficient of passive earth pressure for a plane surface of failure. The coefficient of passive earth pressure for a curved surface of failure will thereby be adjusted to a lower value to represent the reduction in passive earth pressure due to earthquake vibration of the resisting soil.

4.15 Bearing Capacity

Current procedure for establishing bearing capacity during earthquake loading condition is essentially similar to that for normal conditions of static loading. Only limited modifications are required to account for the effects of dynamic loading and soil properties when subjected to vibration.

The reaction of foundations, piles, and pile groups are computed by conventional static methods. The seismic effect is represented by a lateral force equal to the weight of the mass times the seismic factor acting through the center of gravity of the mass. The foundation reaction for the earthquake loading condition satisfies the principles of statics for the aggregate of dead load, some reasonable portion of live load, and seismic load.

The ultimate bearing capacity for foundations should be calculated on the basis of Terzaghi's formula for sandy soil and Fellenius' formula for clayey soil as follows:

for sandy soil,

$$p_{max} = \frac{\gamma_m b N'_\gamma}{2} + \gamma_m h N'_q$$

for strip foundations on clayey soil,

$$p_{max} = 5.52c + \gamma'_m h$$

for rectangular foundations on clayey soil,

$$p_{max} = 5.52c\left(1 + 0.38\frac{h}{b} + 0.44\frac{b}{L}\right) + \gamma_m h$$

where

p_{max} - ultimate bearing capacity

γ_m - unit weight of soil, γ'_m should be used below the water level

c - cohesion of soil per unit area, $c = q_u/2$

q_u - unconfined compressive strength

b - width of foundation

h - depth of foundation

[illegible]

[illegible]

[illegible]

$$[illegible]$$

[illegible]

$$[illegible]$$

L - length of foundation

N'_γ ,N'_q - bearing capacity factor for uncompacted sand, charts available in reference (100)

As set forth in the article concerning Bearing on Sand, second part of Chapter 3, a reduction in bearing pressure is in order for obliquely loaded foundations for the case of earthquake loading. The lateral earthquake force plus any other oblique or eccentric load may be resolved with the vertical load into an equivalent resultant force. The angle this resultant makes with the vertical, angle e, is then used in conjunction with Plate XVI to determine the ultimate bearing pressure when greater than 10°. This value, ultimate bearing pressure for surface loading, may be compared with the first term of the Terzaghi formula, $\gamma_m b N'_\gamma /2$; whichever is least should be used in the design. In either event, the last term of the Terzaghi formula should be used to take into account the depth of the footing. Oblique bearing load reduction need not be considered for e angle has less than 30°.

The allowable bearing pressure should also be reduced for individual spread footings which are free to rock back and forth as illustrated in Plate XIII. There would be no requirement for such a reduction for footings tied together by a system of grade beams, as is strongly recommended for seismic foundation design in granular soils. This reduction would be for the maximum bearing pressure at the toe of the footing under most critical lateral load condition, as follows:

[illegible]

Very loose sand	70%
loose sand	80%
medium sand	90%

The ample factor of safety for soils in bearing are adequate for all other normal contingencies of seismic design. Terzghis' formula for granular material requires a factor of safety of at least 3. The following factors of safety proposed by Tschebotarioff for clayey soils are recommended: (101)

Degree of sensitivity	Sensitivity ratio S	Factor of Safety	
		Permanent structures	Temporary structures
High	4	3.0	2.5
Medium	2-4	2.7	2.0
Slight	1-2	2.5	1.8
Not sensitive	1	2.2	1.6

The factor of safety to be used for bearing capacity of piling depends upon the method of determining their ultimate bearing capacity. If the ultimate bearing capacity is determined by either load test or by static bearing capacity formula, the above factor of safety table for permanent structures is applicable to clayey soil. A factor of safety of 2.5 is recommended for sandy soils. If the ultimate bearing capacity is determined by pile driving formula, a factor of safety of 6 or 7 should be used for all types of soil.

When deep foundations such as caissons are used in areas of seismic disturbance, the benefits of skin friction should be included in computing bearing capacity. This also holds

[illegible]

true for piling where bearing capacity is computed. The recommended procedure for determining the safe load is as follows:

$$R = p_{max} A + Cf_s h$$

where

R -ultimate resistance of foundation

p_{max} - ultimate bearing capacity

A - area of foundation

C - circumference of foundation

h - embedded depth of foundation

f_s - skin friction per unit area

for clayey soils,

$$f_s = cS = \frac{q_u S}{2}$$

for sandy soils,

$$f_s = \frac{1}{2} \gamma_m h K_n \delta$$

c - cohesion of soil per unit area

S - constant of reduced strength due to remolding, judgement factor ≤ 1, considering thixotrophy

γ_m - unit weight of soil, γ_m' should be used below the water level

δ - angle of wall friction

K_n - coefficient of earth pressure at rest

for loose sand, $K_n = 0.5$

for dense sand, $K_n = 0.7$

For pile clusters, the circumference C should be that of the

perimeter of the pile group. The factors of safety setforth in the preceeding table are recommended for clayey soil and 2.5 is recommended for sandy soil.

4.16 Slope Stability

Analysis of slope stability has many applications in waterfront engineering for both the static and seismic conditions of loading. Seismic slope analysis, like static analysis, is accomplished by use of the Swedish circle method. The forces of earthquake acceleration are added to the gravitional forces of conventional analysis. The vertical component of the seismic force may be taken as one third the horizontal component obtained by the conventional method of weight times seismic factor. (34) The direction of the seismic mass should be selected to give the condition of least stability. The vertical component could be figured either upward to deduct from the normal force developing frictional resistance or downward to add to the rotational moment of shear forces, and the horizontal component would be computed to act in the direction of sliding. As setforth in the clay section of Chapter 3, clayey soils have increased strength under conditions of short duration loading. The value of cohesion used for the slope stability analysis may be between 140 to 260% of the conventional value under static loading. (80) Accordingly, the allowable cohesion may be taken as twice the value used for the static investigation, with a reduction in percentage of

[illegible]

increased strength for sensitive clays.

The factor of safety for slope stability is defined as the ratio of total shearing resistance to the total shearing force. The most critical period of slope stability for waterfront structures is directly after construction. The foundation material consolidates under the load of the structure and its backfill, resulting in a greater value of cohesion. The fact that sandy soil is the prevalent backfill material for waterfront structures also reduces the hazard of slope failure under seismic conditions. The general method of analysis for seismic vibration, with its additions and subtractions to mass forces, inherently results in an additional factor of safety due to the phase difference of vibration at different locations within the slip circle mass.

The analysis must be based on saturated unit weights in the wet zone above the low water line, normal unit weight above the saturation line, and bouyed unit weight below the low water line. The bouyed seismic factor is applicable below the low water line while the normal seismic factor is used above. Adjustments for pore pressure, commonly used in earth dam embankment investigations, are not required for this analysis.

4.17 Dynamic Water Pressure

Extensive investigation of dynamic water pressure has been undertaken for the seismic design of dams. The most significant work was accomplished by Westergaard in connection with the

design of Hoover Dam. (106) His formulas are based on the assumption of simple harmonic motion perpendicular to the vertical face of the dam. The resultant pressures are the equivalent of a body of water confined between the upstream face of the dam and a parabola moving back and forth with the dam while the remainder of the reservoir is inactive. The parabola is known as the Westergaard parabola, and is defined by the following equation:

$$x = \frac{7}{8}\sqrt{Hy}$$

where

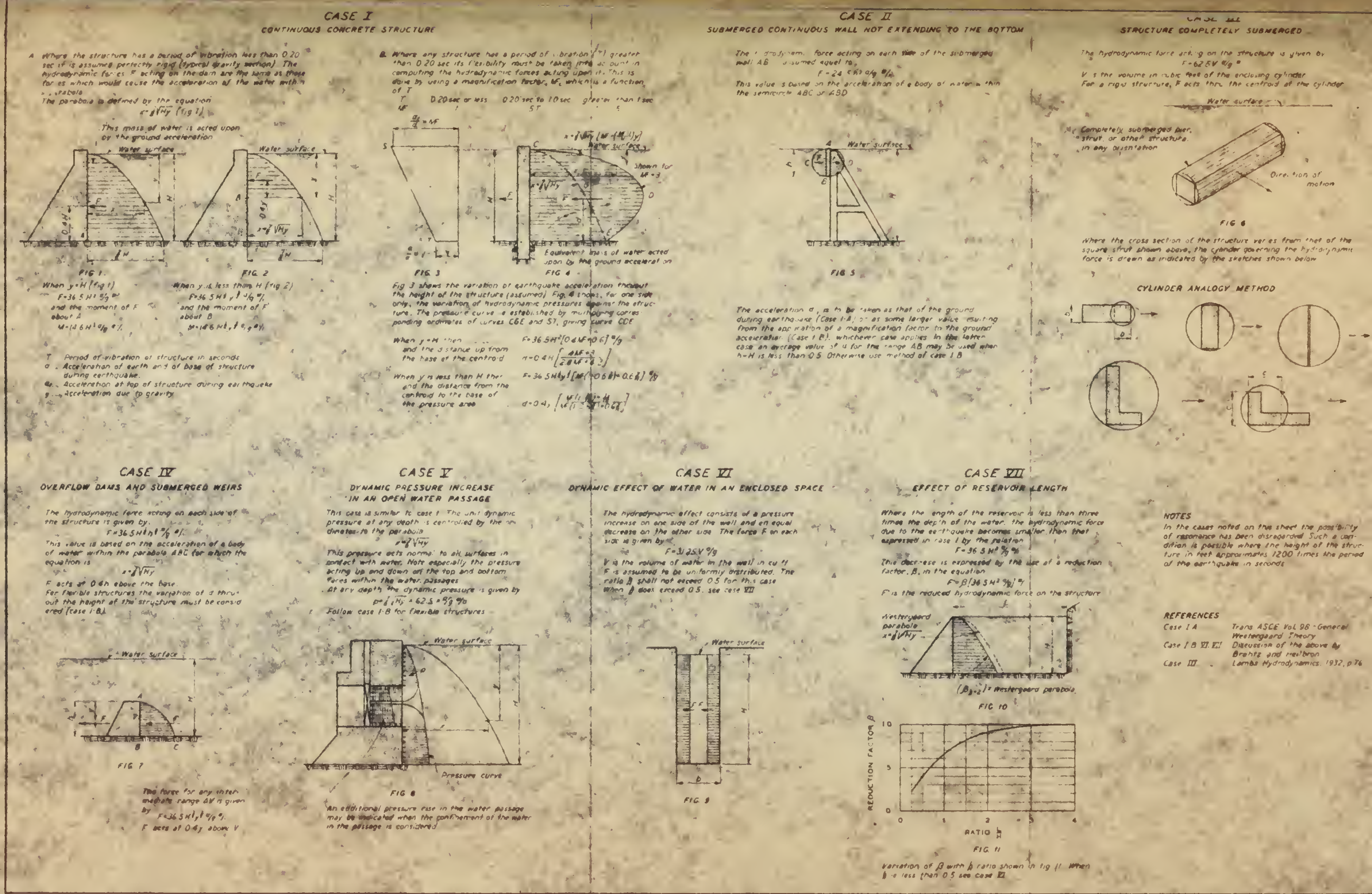
x - horizontal width of water at depth y

y - any depth of water

H - water depth of dam

The resulting mass of water is multiplied by the seismic factor, as is the mass of the dam, to define the lateral earthquake force. The resultant acts through the center of gravity. The normal hydrostatic pressure of water against the face of the dam continues to act in addition to the dynamic water pressure.

Flexible structures experience an increased dynamic water pressure with height. Plate XX illustrates the mechanics of this increase as well as the diagrams and equations for dynamic water pressure on various types of structures. (93) Plate XX was derived for dams; accordingly, it is based on water weighing 62.5 pounds per cubic foot. It must be adjusted for salt water, weighing 64 pounds per cubic foot, by substitution of the



(FROM TVA, REFERENCE 93)

~~Figure 306.~~—*Dynamic water pressure during earthquake.*

PLATE XX

following values

37.3	vice	36.5
19.4	vice	14.6
25.2	vice	24.5
64.0	vice	62.5
32.0	vice	31.25

As dynamic water pressure is an oscillating phenomenon, the water pressure on the upstream face consecutively increases and decreases by the amount of the dynamic water pressure. When there is water on both sides of a barrier like structure, the seismic investigation should be predicated on a simultaneous increase and decrease of water pressure of equal magnitude occurring on the two sides of the barrier. The total effect of dynamic water pressure would be twice the values expressed for Cases I and IV shown on Plate XX.

Dynamic water pressure is not taken into consideration either in the backfill behind or the water in front of quay wall in seismic design. This is because the complexity of wall motion and the interrelation of dynamic pore water pressure, outboard dynamic water pressure, and earthpressure, including their phase differences, are not fully understood. However, the values of lateral earth pressure given by the Mononobe-Okabe equation employing the bouyant seismic factor includes the net effect of earthquake motion as verified in lab tests by Ishii. (48)

The principles of and expressions for dynamic water pressure

may, however, be utilized to analyze other type of waterfront structures for seismic loading conditions. For instance, they may be used to investigate floating type caisson gates for graving docks when the caisson gate is in place and the dock pumped out. They may also be used to analyze lateral pressure on a marine railway with vessel and cradle in the water. There are many loading conditions for seismic design on waterfront structures which can be articulated by use of the principles of dynamic water pressure as setforth in Plate XX.

The water entrapped in or between structural elements of a quay wall should be computed on the basis of its mass force for seismic investigation instead of its dynamic water pressure. (50)

4.2 Earthquake Design of Waterfront Structures

4.21 General

With the development of techniques of analyzing the magnitude and distribution of earthquake loads, it is now possible to analyze various waterfront structures for conditions of earthquake loading. The second part of Chapter 4 will discuss several major types of waterfront facilities in an attempt to identify the principal considerations of seismic design. This discussion will concern itself with those features of design which are unique to the earthquake loading condition. The normal considerations of engineering design will receive little or no consideration.

4.22 Site Investigation

In addition to the conventional site and subsoil investigations, a geological investigation should be undertaken of sites for major waterfront installations to locate active and inactive faults in the immediate and general area. Information regarding major active faults in the area is useful in detemining the seismic factor for the project. The location of faults in the immediate area is important to the site planning of structures.

Construction of graving docks, shipbuilding ways, marine railways, major pier installations, and other important structures should never be planned over an active fault. Construction at such a site should be avoided even though the fault terminates several thousand feet beneath the surface. Faults believed to be dormant should be avoided whenever possible as their inactivity may only be a relative thing. Such locations should be used as a major construction site only after thorough investigation and competent geological advice.

It is often impractical to avoid constructing structures like seawalls, breakwater type structures, and long quay walls over an active fault. It would be economically infeasible to incorporate enough strength in such a structure to enable it to resist all the forces released in the fault zone. The design philosophy for such a structure should be that structural damage would be inevitable within the fault zone for an earthquake emanating from the fault over which the structure was constructed.

The paramount design consideration would be to develop a structure which could be easily repaired after earthquake and which would cause relatively small damage to adjacent structures when it underwent earthquake distortion.

4.23 Flexible Sheetpile Bulkheads

Modern practice in harbor construction has placed increased reliance on flexible steel sheetpile bulkheads due to their relatively low cost and their ease and speed of construction. The largest sections of sheetpiling permit depth of construction of approximately 30 feet under seismic conditions. This depth may be extended by use of a relieving platform or light weight backfill. The importance of this type construction has led to considerable work in both laboratory tests and field measurements to determine the actual load on sheetpile bulkhead for economical engineering design. (101) However, special considerations are required for the seismic design of such bulkheads. Ishii and other Japanese engineers have undertaken investigations of this type of construction under earthquake conditions. (2, 50) They have proposed modifications to the static method of analysis developed by G. P. Tschebotarioff and based on the assumption of a plastic hinge at the dredge line, as determined by a series of tests.

The lateral earth pressure of granular material used in the seismic investigation for the design of a sheetpile bulkhead should be computed by the Mononobe-Okabe formula. The

applicable seismic factor should be utilized to determine the lateral earth pressure above the low water line, and the bouyed seismic factor must be used below the low water line. The design weight of the backfill should, as in normal design procedure, be the saturated unit weight within the tide and capillary saturation zone, the soil unit weight above the saturated line, and the bouyed unit weight below the low water line. The surcharge load contributing to lateral earth pressure need not be the greatest pressure utilized in the normal design analysis, but may be some reasonable portion of the maximum loading anticipated at the time of seismic occurrence. Two-thirds the design surcharge load is recommended for the seismic loading condition based on normal conditions of wharf usage.

The embedded length below the dredge line of the steel sheetpiling should be 120% of that computed by the free earth support method. Free earth support embedment provides a safety factor of approximately 2 for normal design loading when the coefficient of passive lateral earth pressure provides for friction between the soil and sheetpiling, and the 120% value provides a safety factor of approximately 1.5 under seismic conditions of loading. The increased depth of embedment is required due to the loss of passive resistance caused by earthquake vibration of the foundation soil. Japanese experience indicates that this depth is sufficient to maintain the stability of the toe of the bulkhead during major earthquakes.

The tie rod design is the most critical element of the

seismic investigation. Embedment deformations and increased lateral earth pressure due to seismic motion result in large increases in the tie rod load. The most likely mode of failure would be through the tie rod due to a summation of effects of other seismic displacements. As a result, an increased factor of safety should be used for design of the tie rod. An allowable strength for the tie rod of 13,000 psi is recommended for static condtions and 20,000 psi for seismic conditions. This represents a factor of safety of 2.5 and 1.67 respectively. The load on the tie rod is computed according to the principles of statics based on the lateral earth pressure being supported by the tie rod and the hinge, or location of zero moment, at the dredge line. These increased factors of safety take into account bending of the tie rod due to surcharge and consolidation of the backfill or subgrade material. Increased factors of safety are in order when there are questions about the mechanical properties of the subgrade and/or backfill materials. It should be recognized that these factors of safety are much greater than indicated as the allowable strength is based on the strength at the elastic limit. The ultimate strength of the material provides an additional reserve of safety although permanent deformations will be incurred.

The computations for tie rod tension permits the determination of maximum bending moment for both the static and seismic cases. The required section of steel sheetpile may then be computed by conventional methods. It is recommended

that the allowable strength of the sheetpile under static and seismic conditions be 20 to 24 thousand psi and 28 to 32 thousand psi respectively, depending on the certainty of the soil data and importance of the installation. The value of maximum bending moment derived by the equivalent beam method is 40 to 50% less than that computed by the free earth support method, taking into account the reduction in bending moment resulting from the flexibility of the sheetpile.

Anchor plates or deadman anchorages are required to withstand the pull of the tie rods. They must be placed sufficiently behind the bulkhead to provide space for the active failure wedge and the passive failure wedge. The active failure wedge starts at the dredge line and the passive wedge starts at the bottom of the anchorage making an angle with the vertical of $(45^\circ - \phi/2)$ and $(45^\circ + \phi/2)$ respectively. These wedges should not intersect before they intersect at the surface of the ground. During earthquake vibrations the active wedge becomes somewhat flatter and the passive wedge becomes somewhat steeper. These effects essentially cancel each other; however, a clear zone between the static active wedge and static passive wedge equal to the depth of anchorage embedment to offset the greater depth of the active wedge is recommended for added assurance. The lateral resistance of the anchor plate should be designed for three times the tension in the anchor rod for the static loading condition and twice the tension in the tie rod for the seismic

condition of loading.

The lateral earth pressure and bending moment in the sheetpile is greatly increased when backfilled with a fluid clay material. This is true even after consolidation of the dredged material. Accordingly, when a sheetpile bulkhead is to be backfilled with dredge material containing more than 5% clay, the installation of a sand dike is recommended behind the bulkhead. This dike should be of clean sand, and should rest on its natural angle of repose away from the bulkhead. The composite backfill section will react as though it were of sand. When soft clay is encountered below the dredge line, removal and replacement of the soft clay with clean sand should be investigated to provide adequate stability by the Swedish circle method of analysis.

Tie rods and anchorages are not required when a relieving platform is utilized as the horizontal lateral earth pressure and seismic forces are resisted by the platform and pile structure. The maximum bending moment in the sheetpile decreases with the reduced depth of lateral earth pressure and elimination of surcharge effect. However, the large mass of the relieving platform and fill material which it carries results in a large lateral force during an earthquake. The batter piles must be capable of withstanding this large horizontal force in addition to the reaction of the lateral earth pressure. The height of the platform must therefore be designed on the basis of a balance of bending moment in the sheetpiles and bearing capacity

of the batter piles. (50)

Marginal wharf type structures are beneficial in areas of soft foundation material. The berthing depth is permitted to slope upward underneath the wharf on its natural angle of repose. This greatly reduces the required depth of flexible bulkhead and resulting lateral earth pressure. The relatively small mass of a marginal wharf, as compared with an earth filled relieving platform, does not result in excessively large lateral earthquake force. The probable mode of failure of marginal wharfs is by sliding. Accordingly, the entire installation should be analysed by the method of Swedish slip circles for the condition of seismic lateral loads.

Particular emphasis in the design of sheetpile bulkhead is required at corners as field observations have repeatedly shown that damage is greatest at this location. The back anchorages for sheetpile bulkheads necessarily cross at corners, resulting in a reduced area to develop the passive resistance for anchorage. Installation of cellular type construction or king posts at corners is beneficial. Any detail which provides excessive rigidity at the corner should be avoided in a flexible sheetpile bulkhead system. Special selection and placement of the backfill material will aid in preventing excessive corner displacements. (2, 50)

4.24 Gravity Type Quay Walls

Gravity type quay walls are popular structures in harbor

[illegible]

developments. They are simple to construct and quite often low in cost, depending on local conditions. They are either of massive stone or concrete sections, or they are constructed from caissons floated into place and later filled to result in a dense mass. They are durable structures and suffer little damage due to ship impact. Their use is somewhat limited by the depth of water. Great depths of water requires great mass to resist the lateral earth pressure and excessive bearing pressures. This is particularly true in the earthquake loading condition where excessive toe pressures are encountered in addition to the seismic increase in lateral force.

The external forces considered in the design of gravity type quay walls are lateral earth pressure, the mass force of the wall, dynamic water pressure, and the resultant bearing pressure. The dynamic water pressure is included in the value for seismic lateral earth pressure obtained by the Mononobe-Okabe formula. Seismic design requires that the ratio of the horizontal force, made up of seismic lateral earth pressure and the lateral seismic mass force , to the weight of gravity exceed the coefficient of friction of the base. The coefficient of base friction values generally used are between 0.5 and 0.6. Japanese practice recommends a safety factor of 1.2 or greater, for the static condition of loading with this safety factor being reduced as low as 1.0 for the seismic loading condition. (50) Some lateral movement of the wall is inevitable if the earthquake vibration is great enough to liquify the backfill.

Gravity quay wall design will generally require some embedment of the gravity section below the dredge line. This will result in a small passive wedge at the toe of the gravity section which will resist sliding, and which will provide an added factor of safety.

The foundation bearing pressure diagram for both the static and seismic condition of loading must be checked against the allowable bearing capacity. Allowable bearing capacity predicated on factors of safety as heretofore setforth should be utilized for the static condition of loading. Increased allowable bearing capacity should be used for the seismic loading condition; however, the maximum bearing pressure at the toe of the base should be predicated on factors of safety equal to those for temporary structures as setforth earlier in this chapter. A sand and rock base should be placed to distribute the load to the foundation material before setting a gravity type quay wall in final position. It is important that a blanket of sand be placed on clay foundation material to protect it from the remolding effects of the rock course or of the quay wall itself while undergoing earthquake induced movement. It should be recognized in analyzing the bearing capacity at the base of a gravity quay wall that the backfill contributes to the load by some undeterminable amount due to friction between the backfill and the back side of the gravity section.

[illegible]

Therefore, determination of bearing capacity at the base predicated on the general strip foundation formulas is not completely applicable to this type of gravity retaining structure.

Analysis of the slope stability of gravity quay walls is important for seismic conditions of loading. The lateral force resulting from the great mass of gravity quay wall sections, as compounded by the increased bouyed seismic factor below the water line, results in substantial slide inducing moments which must be resisted along the curve surface of potential failure. The seismic lateral force in the slide analysis should be based on the mass of both the gravity quay wall and the granular backfill active pressure wedge. The lower limits for the factor of safety for stability of slopes under seismic condition of loading is 1.2. A factor of safety of 1.5 is applicable to the static condition of loading. These values may be reduced to 1.2 for the static case and 1.0 for the seismic case for temporary or unimportant structures. This type analysis is generally not required for sandy or gravely soil foundations; however, a foundation material containing excess of somewhere between 5 to 10% clay acts as though it were a clay. (101) This type analysis is also particularly applicable to marginal wharf type installations.

Special precautions should be taken to prevent aggravated earthquake damage at corners, approaches, and transitions between different type structures as is generally encountered in earthquake field investigations. These include increase of

seismic factor by some 20%, installation of tie backs at corners, and/or connecting transition sections of quay wall in the direction of the quay wall. Recognizing the greater intensity of earthquake motion near the surface of the ground, light utility trench sections running along the top and acting as part of the quay wall section must be connected to withstand the uplift of overturning moment.

4.25 Backfill Material

Granular materials are preferable for backfill of quay walls. They exert less lateral earth pressure than saturated clayey soil backfills. The greater the angle of internal friction of granular backfill material, the smaller the section of sheetpiling, the less embedment depth, the smaller the tie rod diameter, and the smaller anchorage plate required for both the static and seismic conditions of loading. Similarly, backfill material with a large angle of internal friction has a large coefficient of friction between the backfill and wall section, resulting in further economies in the aformentioned material dimensions. Gravity quay walls experience similar reductions in lateral earth pressure resulting in reduced mass and base area for bearing and frictional resistance. In addition to the benefits of reduced lateral earth pressure associated with increased angle of internal friction, increased relative compaction of the backfill material reduces settlement and other adverse effects of earthquake motion. The greater

[illegible]

the relative compaction, the greater is the acceleration of vibration required to liquefy the backfill material as described in Chapter 3.

Accordingly, it is desirable to obtain a backfill material with the greatest possible angle of internal position friction and to place it in as compact a state as possible. The selection of backfill material must, of course, be predicated on the basis of an economic balance of backfill costs with steel costs. Fine sand is the most common material used for backfill due to its relatively low cost and general availability. This material should not contain more than a trace of clay. Granular material placed in the wet settles in an extremely loose state. Efforts should be made to increase its relative compaction. While this is difficult in the wet, any improvement will be beneficial during conditions of earthquake motion. One very effective way of compacting clean granular material in the wet is vibroflotation. This method may be used beneficially for important structures where the consequenses of earthquake outage offsets the additional cost of vibroflotation. For projects where this cost is not justified, large concrete vibrators or a crane headache ball may be used to improve the condition of the backfill. Heavy construction equipment, including surface vibrator compactors, may be used when there is an adequate backfill cover over the tie rods. Whenever there is the possibility of excessive consolidations of underlying clay strata, the tie rods should be protected from the load of settleing surcharge material

by conduit or box structures. (101) Gravel is much superior to sand as an earthquake resistant backfill material; however, its use is generally prohibited by its cost. Laboratory tests indicate that gravel backfill does not undergo liquefaction during vibration. It may therefore be used without compaction. Quarry run material is also a good backfill material when it contains less than 5% clay; however, it is rarely available at a competitive cost.

Blast furnace slag may also be used beneficially for backfill whenever available in the immediate area. Steel plants are generally on the lookout for methods of disposing of blast furnace slag, and it may therefore be a competitive backfill material. There are three types of blast furnace slag of which two types can be beneficially used for backfill of quay walls. They are air cooled slag with a nominal size of 1 1/2" to 3/8" and expanded slag with a nominal size of 3/8" to #4 sieve. The third type of blast furnace slag is granulated slag; however, it compacts to a unit weight similar to that of sand and thereby offers no particular advantage over sand. On the other hand, air cooled slag weighs between 70 to 85 pounds per cubic foot while expanded blast furnace slag weighs between 45 and 65 pounds per cubic foot for fine aggregate and from 25 to 45 pounds per cubic foot for course aggregate. Both types have the physical characteristics of crushed rock, and thereby have a high angle of internal friction. They do not disolve in water, and they under go an increase in shear strength after placement

due to a cementation between particles. Blast furnace slag is purported to be non-corrosive by the National Slag Association. However, these claims are based on reinforcing steel in blast furnace slag aggregate concrete and in blast furnace slag cement concrete. (66) Further investigation of the corrosive effects of slag used as a backfill with steel, both above and below the water line, is recommended prior to its extensive use. The combined benefits of large angle of internal friction and small unit weight permits bulkheads of greater depth, even under conditions of seismic loading, than other type backfill material.

Clay backfills should be avoided where ever possible. When obtained through dredging, they exert a lateral earth pressure equal that of a fluid of their specific gravity. Upon consolidation, this lateral earth pressure is reduced to the neutral condition of lateral earth pressure. The consolidation of clay backfill has an adverse effect on the tie rods by inducing bending. At the same time, consolidation increases the cohesive strength of the material and reduces the chance of slide failure. It is often necessary to use dredge material containing large quantities of clay for backfill in large reclamation projects. In such cases, a dyke of clean granular material should be placed directly behind the quay wall and laid back on its natural angle of repose. As previously mentioned, this will reduce the lateral earth pressure to that of a non-cohesive material. (101)

4.26 Graving Docks

Graving docks enjoy a favorable site position in respect to earthquake motion due to their location below the surface of the ground. Surface waves are the most destructive element of earthquake motion. They have the largest amplitudes and highest accelerations; their strength decreases rapidly with depth below the surface of the ground. However, the foundation conditions of graving docks may well be a source of earthquake hazard. Rock is the ideal foundation material for earthquake resistant graving docks, as it is for normal design considerations. The amplitude of rock vibrations is necessarily slight, otherwise the rock would fracture thereby utilizing the energy vice transmitting it, and reliable bearing is assured. Deep strata of saturated, unconsolidated alluvium is the least desirable graving dock foundation material. Conservative values of bearing pressure are required as amplified ground motion must be withstood by the structure and its foundation. Sites composed of sandy foundation material fall somewhere between the extremes of rock and unconsolidated clay in desirability for locating a graving dock.

The walls of a graving dock, acting as a structural frame along with the floor section, must be able to withstand the increased lateral earth pressure of seismic vibration for all conditions of utilization, i. e. during construction, empty without ship, empty with ship, and full of water. As a graving dock is a rigid structure, it must be capable of withstanding

the full force of earthquake acceleration. The Mononobe-Okabe formula for lateral earth pressure adequately expresses the conditions of seismic loading of the graving dock walls.

The most important consideration in the seismic design of graving docks is the determination of the seismic factor, This is principally predicated on the seismicity of the immediate area and the foundation characteristics of the site. (16) The cost of the structure is significantly affected by this determination.

4.27 Piers

Piers extending out over the water on pile foundations have a good record of earthquake resistance. The relative flexibility of this type structure due to long unsupported piling allows them to ride out earthquake motion with nondestructive displacements. Their design for ship impact and resulting absorption of energy results in earthquake resistance characteristics. The principal problem of seismic design of piers is the connection to the relatively rigid shore. Earthquake field observations have repeatedly evidenced damage at this connection. It is extremely difficult, if not impossible, to design a connection between a relatively rigid land mass and relatively flexible pier structure which would resist, or adapt to, all conceivable ordinates of earthquake dislocation. Accordingly, the designer would be well advised to install an expendable transition section. This section would be sufficiently

weak as not to damage abutting portions of the structure while being adaptable for rapid and convenient replacement. Utility lines running between the shore and the pier must be capable of undergoing large displacements without failure at this transition section.

A more sophisticated analysis would be in order for piers with a large superstructure mass. Such a structure would have a longer natural period of vibration than most waterfront structures, and would be adversely affected by long period, large amplitude ground motion as found in deep strata of unconsolidated, saturated alluvium common to harbor location. Such a structure should be analyzed by the methods of structural dynamics. The depth of fixity of the piling below the dredge line and the relative fixity of the piling cap may be determined by conventional methods to arrive at the piling length for column analogy. The natural period of the pier may then be calculated, and the earthquake force analyzed by standard response spectrum technique. (7, 102) Design utilizing this type analysis should be based on substantial factors of safety as present response spectrum data has been collected for, and is applicable to, superior foundation conditions than are found at waterfront installations. Development of standard response spectrums for waterfront geological conditions, which will probably reveal maximum accelerations at larger natural periods of vibration than for good foundation conditions, will make this approach feasible.

4.28 Breakwaters

Breakwaters and similar structures, such as groins, are little affected by earthquake force. Their normal design requires them to withstand lateral ocean wave forces during storm conditions much greater than earthquake loads. Subsidance or uplift of the sea bottom due to the mechanics of earthquake crustal dislocation would certainly cause permanent deformation of a breakwater. Accordingly, a rubble mound type flexible breakwater would be particularly adaptable to locations of severe earthquake occurrence. The relative ease of repairs to a rubble mound breakwater enhances its desirability for seismic location. Rigid type breakwaters of cut stone, concrete block, or caisson variety would be more difficult to repair after large earthquake deformation than a rubble mound type.

Rigid breakwaters may be analyzed for earthquake loading conditions by use of the dynamic water pressure techniques developed by Westergaard as described in the first part of this chapter. Twice the dynamic water pressure force should be used to represent the worst condition of effects on both sides of the structure. The seismic analysis should include bearing pressure at the toe of the rigid section. A sand and stone foundation mat should be laid prior to placement of a gravity breakwater. A sand blanket is required on the bottom to prevent the remoulding effect of stone movement.

Chapter 5

TSUNAMI

5.01 Introduction

Tsunami, or seismic sea waves, are a related manifestation of the seismicity of the earth which is of concern to waterfront engineering. The occurrence of destructive tsunami is less frequent than destructive earthquakes; however, as its destructive action is restricted to the seacoast, it is incumbent upon waterfront engineers to consider its effects in the design of waterfront installations. This chapter will describe the origins and characteristics of tsunami, their effects on engineered structures, and appropriate design considerations.

5.02 Origin and Characteristics

The exact mechanism of origin of tsunami is not known other than it is a product of the structural adjustment of the earth's crust, or an earthquake. It was originally believed that tsunami was caused by faulting on the surface of the ocean floor. Current investigation and rationalization offer two other explanations, either one of which, or a combination of both, offer a reasonable explanation. The first theory explains tsunami as being caused by subsidance or uplift over large areas of the ocean bottom resulting from crustal adjustment. Such occurrence was observed after the tsunami producing Tokyo earthquake of 1923 and Chile earthquake of 1960, as setforth in Chapter 2, although it cannot be concluded that this was the

sole or principal cause. The second theory explains tsunami as the product of underwater landslides or mudslides triggered by an earthquake along the steep submarine slopes of the continental shelf. This latter explanation accounts for the lack of tsunami originating along the highly seismic area of the western United States. The essential combination of steep submarine slopes and great bodies of soft sediment appear to be lacking. Contrasted to this are Japan and Chile, areas of frequent tsunami origin, where deep ocean trenches with steep slopes are found adjacent to areas of high seismicity. The relatively recent geological origin of these areas and their steep coastal mountains would indicate the presence of expansive masses of soft sediment. (16)

Tsunami is made up of a series of long period gravity waves. The characteristics of propagation and motion of tsunami conforms to water wave theory. Their wave length is much longer than ocean storm waves and shorter than normal semi-diurnal tide waves caused by gravitational pull of the moon and sun. The height of tsunami waves in the open ocean is very slight; they are unnoticable to ships at sea. They travel with a celerity of between 300 to 800 miles per hour.

Observations in Japan have indicated that tsunami with nearby origin have a natural wave period in the neighborhood of 12 minutes. It was also observed that the tsunami which originated in Chile during the 1960 earthquake had a wave period of approximately 1 hour. The wave periods of this tsunami

observed in San Diego, California and Hawaii fell between these values. Wave height, while frequently distorted by local geographical conditions, diminishes with distance from the point of origin. It is concluded from these observations, as would be anticipated from wave theory, that tsunami wave period increases with distance from point of origin while wave height decreases in a manner similar to wind generated waves. As a result the long period of tsunami waves, they are substantially affected by all depths of ocean. Accordingly, their celerity may be computed by the mathematical expression for very shallow depths as follows:

$$C = \quad gd$$

where

C - celerity

g - acceleration of gravity

d - depth of water

Tsunami waves undergo refraction throughout their travel as a result of their shallow depth characteristic. The 1960 tsunami which originated in Chile was refracted around both sides of Hawaii and converged on Northern Honshu, Japan to wraught extraordinary damage. (49) Plate XXI illustrates a typical example of the propagation of tsunami and the aggravated refraction along the upslope to the continental shelf. (68)

Tsunami may affect any coast line in the world as a result of its ability to propagate great distances with little decay. However, the coast lines of the Pacific Ocean are most affected.

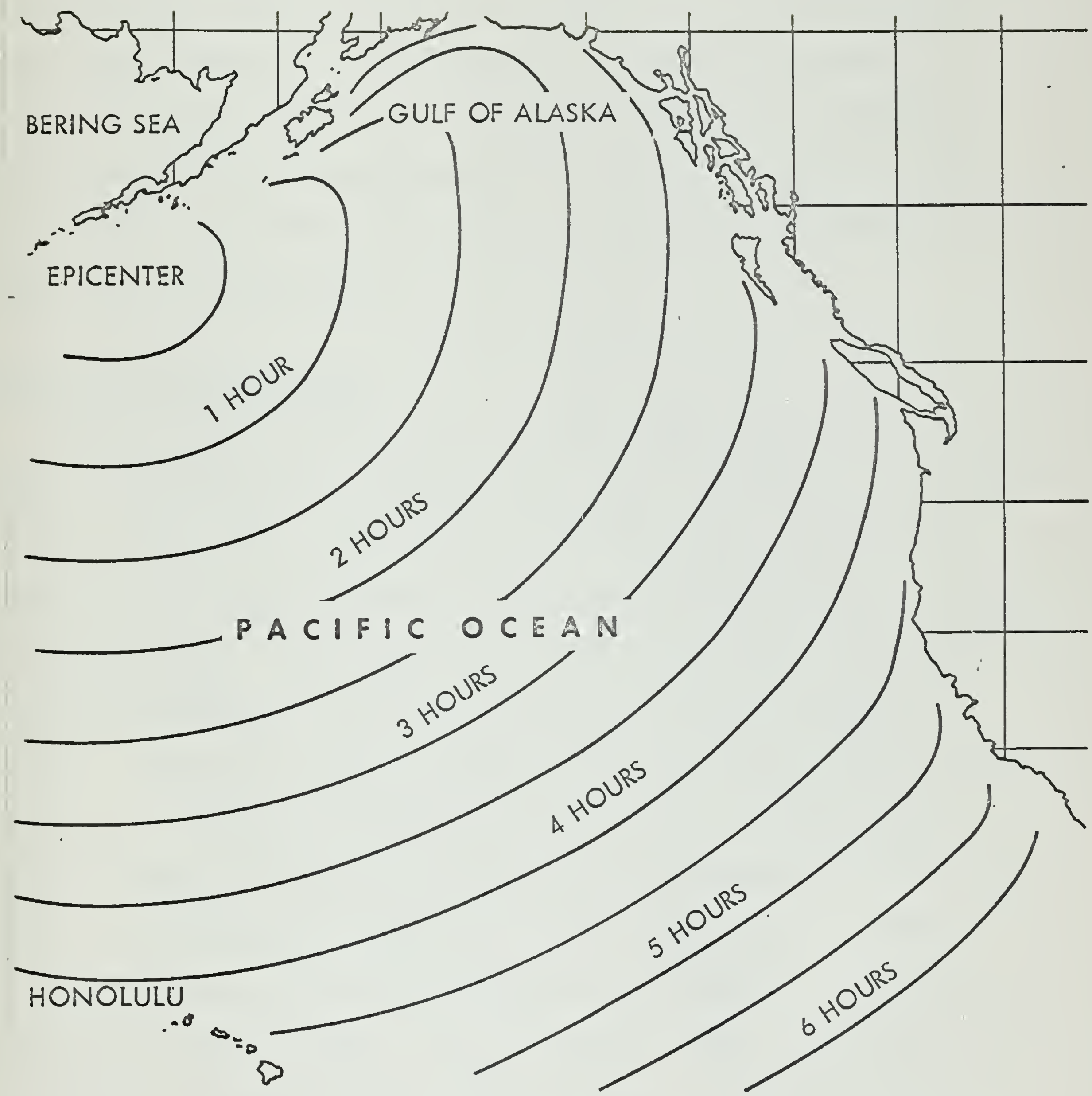


FIGURE 12
Typical Seismic Sea Wave Pattern
(From BUDOCKS, Reference 68)

PLATE XXI

This is a result of the circumpacific belt of greatest seismic activity which surrounds this area. The incident direction of tsunami is largely independent of the exposure of a bay due to its very large wave period. A network of detection stations which issue warnings and predict time of arrival is now in existance in the Pacific. Despite the great speed at which tsunami travels, these warnings greatly reduces loss of life and permit a short time to place a waterfront installation in a condition of readiness.

5.03 Effects of Tsunami

Tsunami waves do not roll in and break as popularly conceived in fiction, but perform more like a frequent flowing and ebbing of the tide. The great damage caused by tsunami is a result of the extraordinarily high water levels which occur in bays and inlets. The great wave length and long wave period of tsunami transports a great mass of water in its slight wave height. This great mass of water accumulates in coastal inlets to result in exceptionally high water levels. Tsunami flow stage as high as 55 feet has been observed; however, the occurrence of such extreme heights are rare. (68) Severe damage results from much lower tsunami where sea walls are overtoped, structures near the shore line are washed away, and boats are set adrift to collide with other objects. Proportionally low water levels occur during the tsunami ebb stage.

It is a popular belief that a tsunami is always preceeded

by a drawdown of the water level. This is not necessarily true. Several tsunami have been observed where the first indication was an increase in water level. The phase of the initial wave front is dependent on a locations' geographical relationship to the submarine landslide or subsidance. The first wave front to arrive, either flow but usually ebb, is of a lesser magnitude than the wave fronts to follow. This phenomenon affords some warning to people near to, but not affected by, the initial change in water level.

Tsunami, like normal tide varation, is greatly affected by geography. It is believed that when the wave period of tsunami exceeds the natural period of oscillation of a bay, the wave height within the bay attenuates as it travels across the bay. On the other hand, when the wave period of the tsunami is less than the natural period of oscillation of the bay, the height of the tsunami becomes magnified as it advances. These effects have been observed in Japan through comparing heights of short period tsunami of local origin with long period tsunami which originated in Chile. Occasionally, tsunami causes a bore in rivers flowing into ocean bays. (49)

The action of tsunami in a harbor is generally a series of flowing and ebbing water motion, each cycle occurring over the time of one wave period, extending over a number of hours. The amplitude of each subsequent wave cycle gradually reduces after a series of maximum amplitude wave cycles. When conditions of harmonics are satisfied by the harbor and the tsunami,

standing waves may be set up which require many hours to settle out. The transport of large quantities of water in ebb and in flow every 12 to 16 minutes necessarily results in high currents. Spectacular currents resembling river rapids have been observed at restricted harbor entrances and between groins. Currents as high as 15 to 20 knots, reversing direction as often as every three minutes, may occur. Great turbulence, eddy currents, and whirlpools result. The tsunami may change the depth in a harbor by washing out certain areas and silting up others. Navigational aids and bouys may be carried away or moved out of position. Reduced currents are experienced in the bay proper; however, their velocity is greater than during maximum semi-diurnal tide flow.

5.04 Engineering Considerations

The initial engineering consideration of tsunami as a condition which may be encountered by a structure or installation is the limits of tsunami water level to be anticipated. The effects of tsunami vary significantly for various localities within a general area. Similar to the maximum intensity of earthquake force for design, the best source of data is the historical record for immediate area. The records of all tide gauges should be analysed for periods of past tsunami occurrence. Unfortunately, tide gauges have not generally performed well during tsunami due to mechanical failure or silted up equipage beyond the range of normal performance levels.

However, the available data extrapolated in accordance with water wave theory gives a reasonable estimate of maximum and minimum water levels. The account of eye witnesses and the study of locations where the extraordinary water levels caused damage are additional methods of obtaining useful data. All well protected harbors, inlets, and coves in areas of tsunami occurrence should be suspect. The current trent in areas of frequent tsunami is the installation of special tsunami gauges at selected locations. These should result in better data for future projects.

One of the most frequent sources of damage to civil engineering structures is the scour of foundations. The retreat of the water level from extraordinarily high levels to exceptionally low levels of drawdown produces strong currents in the great mass of retreating water which washes away fine foundation material. Very often the toe of structures which suffer scour were designed at levels below the L. L. W. level with no constructed means of resistance to flowing water. Seawalls are particularly vulnerable structures to this type action. This outflow of water which has overtopped a seawall, has been likened to hydraulic jetting of the fine foundation material. Design in areas of tsunami occurrence should require a rubble rock mound at the base of structures subject to tsunami scour to protect the fine foundation material.

It may often be infeasible to design a structure for the height of maximum tsunami when the probability of its occurrence

is very slight and the required height would detract from functional characteristics. The feasible design height would afford protection against a more probable tsunami water level. It then becomes advisable to pave the area directly behind the structures. Field observations of destructive tsunami have revealed that washout of backfill of an overtopped structure is a frequent type of damage. Each outflow of water removes additional backfill, and an increase in water pressure results from the substitution of hydraulic pressure for earth pressure. The mechanism of complete failure of seawalls and quay walls is the combination of removal of toe material by scour and increased lateral pressure through removal of backfill and substitution of water, especially after a slight tilt of the structure has occurred and the wall separates from the backfill. Means of rapid and direct drainage of surface water should also be incorporated in the design.

Tsunami introduces a case of unique loading of quay walls. At the time of maximum drawdown, the lateral earth pressure against the quay wall is greatly increased due to the removal of the counteracting water pressure. A substantial drawdown will result in lateral earth pressure in excess of any other condition of loading. Fortunately, tsunami occurs after the seismic loading condition and their effects need not be superimposed in design analysis. The period of time of low water level is too short to permit the beneficial effect of subgrade drainage, and is too long to be considered an instantaneous

condition of loading for increased strength of cohesive soil. Quay walls, and other structures undergoing lateral earth pressure, should be analysed for structural stability and circular slide failure, where applicable, for the condition of maximum low tsunami water level.

Vessels tied securely to any structure during tsunami may introduce unacceptable loads during both high and low water level. Vessels should be taken to sea upon tsunami warning or all lines loosened up.

A harbor should be surveyed immediately after a tsunami to identify locations of silt deposits. Locations of aids to navigation and mooring bouys should also be checked by survey immediately after tsunami to identify and permit correction of any movements.

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