# BEARING CAPACITY OF SANDY SOIL AND
# LATERAL EARTH PRESSURE DURING EARTHQUAKES

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## PREFACE

There are many examples of damage to structures caused by the change of the stress distribution in the ground or the change of the mechanical properties of the soil during earthquakes. Such examples include sliding of quay walls, settlement of embankments and building foundations and landslides. In Japan, the intensity of earthquakes are classified into one of seven grades in the order of the extent of damages. Earthquakes of intensity V are classified as "Very Strong" and cause cracking of brick and plaster walls, damage to chimneys and damage to warehouse structures. Earthquakes of intensity VI are classified as "Disastrous" and cause the destruction of over one percent of all wood frame houses, landslides, and fissuring of level ground accompanied by the emission of mud and water. Earthquakes of intensity VII (maximum) are classified as "Ruinous" and cause destruction of over 30 percent of all wood frame houses.

In other words, the superstructural damage caused by earthquakes of intensity V is considerably increased by earthquakes of intensity VI due to the greater damage suffered by the substructure. As earthquakes of intensity V have an average acceleration of 150 gals and those of intensity VI an average acceleration of 450 gals, it may be postulated that the shearing strength of average soils is reduced considerably when the earthquake acceleration exceeds 300 gals approximately. Though not a confirmed fact, indications are that the characteristics of "Disastrous" earthquakes are different from those of lesser intensites. Two earthquakes, both having the same characteristics except for the level of acceleration, could have different resultant damage aspects due to differing soil response.

Facts indicate that the properties of soils during vibrations are correlated closely to structural damage. Structural engineers should endeavor to develop definite methods of dealing with aseismic soil structures. However, we have at present very little knowledge as to the properties of soil structures during earthquakes. A few results of research on the fundamental properties of soil, the lateral pressure and the bearing capacity during vibrations have been utilized in the aseismic design of embankments, retaining walls, quay walls, bridge foundations and other soil structures. This paper outlines some results of the research on soils mechanics for earthquake engineering in Japan.

## MECHANICAL PROPERTIES OF VIBRATING SOIL

In Japan, Dr. Takeo Mogami was the first to study the behavior of sandy and clayey soils subjected to vertical harmonic vibrations. He found that the shearing strength or the yield value of the soil is diminished considerably by increasing the acceleration of the vibrations

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(1 and 2). In his series of experiments, a shear box was filled with sandy soil and vibrated vertically on a shaking table. Direct-shear tests were performed and the shearing strengths measured under varying vibrations. The experiments indicated that during vibrations of the soil the shearing strength of the soil was correlated more to the acceleration than to any other factor. A typical relationship between acceleration and shearing strength of sandy soils is shown in Fig. 1.

It is of interest to note that at an acceleration above 300 gals, the shearing strength of the sandy soil is reduced considerably to a state noted by Dr Mogami as "liquification". Shear tests were performed also on the soil called "Kanto Loam", and on disturbed and undisturbed clay samples obtained from the bottom of Yokohama harbor. In the case of the shear tests on clayey soil, the specimens began to yield at a shearing force defined as the yield value of this soil. Fig. 2 shows the ratio of the yield values of clay for the case of dynamic testing and static testing against the accelerations of the vibrations. Fig. 3 shows the yield values of Kanto loam during vibrations with various water contents.

It is noted from these results that the yield value of the soil diminishes with the increasing acceleration of vibrations and the amount of the decrease is dependent upon the moisture content. Further, these results show that the amount of the decrease of the yield value is largest at the water content nearest the optimum moisture content. As the optimum moisture content of clay while vibrating changes in value from that at rest, it may be inferred that a soil structure compacted while near the statical optimum is not always the most resistant possible to earthquake forces.

The lowering of the shearing strength of soil due to vibrations seems to be related closely to the large damage suffered by structures from earthquakes of intensity VI. It is of interest to note that the critical value of acceleration causing liquification of soil approximately coincides with the acceleration value of a disastrous earthquake. It is difficult to escape earthquake damage resulting directly from liquification of the soil. Therefore it is the usual practice in Japan, in the regions where disastrous earthquakes can be expected, to place important structures on safer soils or to design them for the effects to be expected if they must be placed on suspect soils. This consists, for example, of extending building foundations to diluvial formation. Also, reinforced concrete piers are used instead of quay walls.

Below a certain limiting value of acceleration, the mechanical properties of soil do not change, practically speaking. Therefore, the stability of soil structures may be investigated analytically by analyzing the change of stress due to vibrations without considering the change in the mechanical properties. In the next two portions of this paper the lateral pressure and the bearing capacity of soil will be discussed for earthquakes with acceleration intensities not sufficient to cause soil liquification.

## LATERAL PRESSURE OF SOIL DURING VIBRATIONS

Dr. Nagaho Mononobe and Dr. Saburo Okabe were the first to study the lateral earth pressure exerted on a retaining wall during an

earthquake and proposed a procedure to estimate the value (3 and 4). In their treatises, superstructures and foundations are assumed to be acted upon statically by the resultant of the gravitational force and the maximum earthquake force. The ground surface is regarded, therefore, as inclined at an angle to be calculated by use of equation 1.

$$i = \tan^{-1} \frac{\alpha_h}{g \mp \alpha_v} \qquad \ldots\ldots\ldots (1)$$

Then the design earth pressure on a retaining wall, shown in Fig. 4a, is derived by the theory of statical earth pressure by use of equations 2 and 3.

$$P = C \left(1 - \frac{\alpha_v}{g}\right) \left\{ \frac{p}{2} H^2 + p_0 H \frac{\cos \psi}{\cos(\psi - \theta_0)} \right\} \qquad \ldots\ldots\ldots (2)$$

where

$$C = \frac{\cos^2(\varphi - i - \psi)}{\cos i \cos^2 \psi \cos(\delta + \psi + i)} \left\{ 1 + \sqrt{\frac{\sin(\varphi + \delta)\sin(\varphi - i - \theta_0)}{\cos(\delta + \psi + i)\cos(\theta_0 - \psi)}} \right\}^{-1} \ldots\ldots\ldots (3)$$

The point of application of the earth pressure caused by the surcharge is located at an elevation of $H/2$ and that caused by the weight of the soil at an elevation of $H/3$ above the base of the wall. The values of C are shown in Table 1 when $\psi$ and $\delta$ both have a value of zero.

These theoretical results were studied by Dr. Haruo Matsuo experimentally (5). A box having dimensions of 0.4 m x 0.4 m x 1.1 m and filled with dry sand was shaken horizontally on a shaking table and the lateral pressure on a wall of the box was measured by pressure gages attached to the wall. The period of vibration was 0.75 to 0.90 sec and the maximum acceleration was 400 gals. Furthermore, similar experiments were repeated by the same author using larger sized boxes and all results agreed in general. The results are shown in Fig. 5. It is seen from these results that the increase of the pressure due to vibrations is much larger near the surface of the ground than at lower depths. This is in contradiction to Mononobe's assumption of linearly distributed pressure.

In 1953 members of the Transportation Technical Research Institute of the Ministry of Transportation built a large size vibration generator and constructed a large model retaining wall at a distance of 10.6 m from the generator (6). The wall is made of concrete with a height and length of 3.0 m and 5.0 m, respectively. The vibration generator consists of a large elliptical concrete bowl buried in the ground with principal diameters of 6.0 m and 4.4 m and a depth of 3.0 m. In this bowl is placed a mechanical vibrator which consists of two sets of unbalanced locomotive wheels. As the wheels are revolved, the centrifugal force due to their unbalanced moments set the bowl in violent oscillation. The surrounding ground rocks similar to an earthquake with the vibrations being felt to a distance of one kilometer from the generator. The maximum angular velocity is 5.7 cps and the maximum vibrating force is 3.15 tons. The maximum horizontal acceleration measured at the top edge of the bowl is about 500 gals. The maximum horizontal acceleration recorded on the ground surface at a point 5.0 m from the edge of the bowl is about 370 gals.

# EARTHQUAKE EFFECTS ON SOILS AND FOUNDATIONS

The study on lateral earth pressure on the model retaining wall using this vibration generator has not been completed as of this date, however, the data so far obtained shows that the distribution of earth pressure on the wall during the vibration tests generally coincides with the results obtained by Matsuo's small scale experiments.

The seismic earth pressure of saturated soils on a retaining wall was studied by Dr. Matsuo and Mr. Sukeo Ohara by model tests. They determined the existence of two kinds of water pressure produced by the pore-water in the soil (7). They filled a box with saturated sand, vibrated it horizontally on a shaking table and measured the lateral pressure on a wall of the box. The distribution pressure of the saturated sand and water pressure on a retaining wall during vibration tests are shown in Fig. 6. From these results, it is noted that a seismic hydraulic pressure exists and oscillates with the period of the exciting vibration force.

When the acceleration is increased gradually and exceeds about 200 gals, another kind of water pressure is noted. The soil suffers consolidation with a sudden increase of the pore-water pressure. This is of practical importance as the sudden increase of pore-water pressure due to vibrations could be the cause of landslides and also of the spouting of sand and water during earthquakes.

The Japan Harbor Association has specified the pressure for quay wall design based on the results of Matsuo's experiments. The specification provides that the earth pressure and the water pressure on a quay wall for earthquake resistant design be determined by adding the following pressure to the normal earth pressure:

1. Earth pressure. The earth pressure to vary linearly from zero at the ground surface to 6 K (t/m$^2$) at the water level in the backfill and again to vary linearly from this level to zero at 6 m depth below the ground surface. When the height, H, of the wall is less than 6 m, the pressure at the water level in the backfill is taken as HK (t/m$^2$) instead of 6K (t/m$^2$) (see Fig. 4b).

2. Water pressure. Normal water pressure acting on the seaward surface of the wall is reduced during earthquakes. This reduced pressure varies in value linearly from $P_2$ at sea level to $P_3$ at the base of the wall, where $P_2$ and $P_3$ are given by equations 4 and 5.

$$P_2 = K \,( 0.26 \, H_1 + 0.70 ) \qquad \ldots\ldots\ldots (4)$$

$$P_3 = K \,( 0.89 \, H_2 + 0.70 ) \qquad \ldots\ldots\ldots (5)$$

where $H_2$ is the distance from the water level in the backfill to the base of the wall (see Fig. 4b).

Harbor structures are now designed generally on the basis of this standard set by the Harbor Association. However, most other types of structures of a similar nature, such as bridge abutments and water works retaining walls, are designed following Mononobe's formula.

# BEARING CAPACITY OF SOIL DURING EARTHQUAKES

The difficulty of determining the bearing capacity of the soil by field tests during an earthquake is apparent. Therefore, it is necessary to ascertain the value from the normal bearing capacity value determined by tests and calculations. For several years, a study on the effects of vibrations on the bearing capacity of sandy soil has been progressing in the applied mechanics laboratory at our Institute (8 and 9).

The test consists of a box filled with sandy soil placed on a shaking table and shaken horizontally. A test plate, loaded vertically, is placed on the surface of the sand. As the box is shaken horizontally, the vertically loaded plate settles into the sand. When the plate is loaded obliquely instead of vertically, the plate slides due to shear failure of the soil beneath the plate before settlement begins. The settlement of vertically loaded plates under similar conditions has already been studied experimentally by other authors, such as Professor G.P. Tschebotarioff (10, 11 and 12). In this paper, the results of experiments on the shearing failure of sandy soil beneath obliquely loaded test plates will be reported.

Dimensions of the test box were 15 cm width x 60 cm length x 30 cm depth. The test plate dimensions were 6.1 cm x 15.0 cm. The sand had a fineness modulus of 1.76, an average dry weight of 1.56 $gm/cm^3$ in the fairly dense state and the angle of internal friction ranged from 38 degrees to 50 degrees, according to the density. The shaking table period of vibration ranged from 1.35 to 0.33 secs, amplitudes ranged from 10 to 25 mm and the maximum acceleration was 380 gals.

It was found impossible to determine the exact bearing capacity when the acceleration of the vibration was more than 350 gals for dry sand and about 300 gals for saturated sand due to the violent motions near the surface of the sand. Up to these limits of acceleration, it was found that the acceleration was the most decisive factor to the bearing capacity.

The relationship between the acceleration of the vibration and the bearing capacity of dry sand is shown in Fig. 10a. This same relationship for saturated sand is shown in Fig. 10b. The angle of inclination of the obliquely applied loads ranges from 15 degrees to 25 degrees from the vertical. From those results, it is noted that the value of the bearing capacity diminishes linearly with increasing accelerations. Typical examples of the line of failure in dry and saturated sand are shown Fig. 8.

To generalize the experimental results, some theoretical considerations are necessary. As mentioned previously, the mechanical properties of sand for all practical considerations can be considered not to change up to a certain limit of acceleration. Therefore, it is assumed that the change of stress in the soil due to the vibrations can be analyzed without consideration of the mechanical properties. During an earthquake, some additional stresses are induced and the value of the bearing capacity reduced. The frequency of earthquake vibration is so relatively slow that it may be possible to analyze statically the resulting phenomena. On this assumption, it may be possible to investigate the stability of

foundations taking only the gravitational and maximum seismic forces into consideration. The resultant of these forces inclines to the vertical at the angle calculated by use of equation 6.

$$i = \tan^{-1} \frac{\alpha_h}{g} \qquad \cdots \cdots (6)$$

It is apparent, therefore, that a horizontal ground surface inclined at a slope angle equal to i can be considered.

The bearing capacity of the sloping ground is calculated using the following assumptions:

1.  Shearing failure occurs along a failure line shown in Fig. 9.

2.  The zone of plastic equilibrium, represented in Fig. 9 by the area ADEC, can be sub-divided into three zones: namely, 1. A wedge shaped zone located beneath the loaded plate, 2. A zone of radial shear emanating from the outer edge of the loaded strip and, 3. A passive Rankine zone.

3.  On account of the existence of frictional resistance between the soil and the base of the plate, the soil located immediately beneath the base of the plate remains permanently in a state of elastic equilibrium and behaves as if it were a part of the sinking footing. Then it is assumed that the soil located just below point D (see Fig. 9) moves parallel to the direction of the external load, and the surface of sliding through point D will start from a tangent in the same direction.

From assumption 3, equation 7 is obtained.

$$\frac{\beta}{2} = \varphi + \varepsilon \qquad \cdots \cdots (7)$$

Another angle, $\beta_1$ , and the reactions on two sides, AD and BD, are determined by the equilibrium conditions of forces.

$$\beta_1 = \varphi - \varepsilon \qquad \cdots \cdots (8)$$

$$F_1 = \frac{pB}{2}(1 + \cot \varphi \tan \varepsilon) \qquad \cdots \cdots (9)$$

$$F_2 = \frac{pB}{2}(1 - \cot \varphi \tan \varepsilon) \qquad \cdots \cdots (10)$$

The equation of the failure line in the zone of radial shear is given by equation 11.

$$\gamma = \gamma_0 e^{\theta \tan \varphi} \qquad \cdots \cdots (11)$$

From Rankine's theory, equations 12 and 13 are obtained.

$$\alpha_1 = \frac{\pi}{4} - \frac{\varphi}{2} + \frac{\gamma - i}{2} \qquad \cdots \cdots (12)$$

$$\alpha_2 = \frac{\pi}{4} - \frac{\varphi}{2} - \frac{\gamma - i}{2} \qquad \cdots \cdots (13)$$

The earth pressure on the side BE is given by equation 14.

$$\frac{\rho \gamma_0^2}{2} e^{2\delta \tan \varphi} \sin \alpha_1 \sqrt{\frac{\sin(\gamma+i)}{\sin(\gamma-i)}} \qquad \ldots\ldots\ldots (14)$$

Setting the sum of the moments of forces acting on the zone of radial shear about point B equal to zero, equation 15 is obtained.

$$\frac{p}{\rho B} = \frac{2 \cos \varepsilon \, \sin(\varphi-\varepsilon)}{3 \cos^2 \varphi \, \sin 2\varphi} \left[ \frac{1}{1+9\tan^2\varphi} \left[ e^{3\delta \tan \varphi} \cos(\alpha_1+i)\{3\tan\varphi - \tan(\alpha_1+i)\} + \cos(\beta_2-i)\{3\tan\varphi + \tan(\beta_2-i)\} \right] + e^{3\delta \tan \varphi} \sin \alpha_1 \cos \varphi \sqrt{\frac{\sin(\gamma+i)}{\sin(\gamma-i)}} \right] \qquad \ldots\ldots\ldots (15)$$

This relationship gives the theoretical bearing capacity of soil during an earthquake.

When sand is saturated with water, the bearing capacity is reduced further. This may be caused by the decrease of the apparent specific gravity due to bouyancy and the relative increase of the ratio of seismic force to the gravitational force. The increase of the pore-water pressure due to vibrations has been determined by Matsuo, as mentioned previously, but its effect on the bearing capacity has not been quantitatively ascertained. Calculating the apparent specific gravity of the saturated sand by equation 16,

$$\rho' = \rho + \upsilon - 1 \qquad \ldots\ldots\ldots (16)$$

the resultant force of the gravity, bouyancy and maximum seismic forces is inclined to the vertical at an angle given by equation 17.

$$i' = \tan^{-1} \frac{\rho \alpha_h}{\rho' g} \qquad \ldots\ldots\ldots (17)$$

It is assumed that the horizontal ground surface is inclined at an angle equal to i' during an earthquake. The solid lines in Fig. 10 represent the theoretical values of the bearing capacity of dry and saturated sand for angles of $\varphi = 42$ degrees and $\varphi = 43$ degrees. The agreement between the calculated and measured bearing capacities is believed to be well within the limit of accuracy usually required for engineering application. It is of particular importance that the theoretical value of the bearing capacity decreases with increasing acceleration in almost the same ratio as the experimentally determined value. These findings support the assumption that the bearing capacity of sandy soil during an earthquake may be estimated from the normal value. For angles of internal friction ranging from 30 degrees to 45 degrees, the values of $p/\rho B$ have been calculated and are shown in Fig. 11.

It is to be noted further that the bearing capacity value shows a phenomenal decrease due to acceleration above about 350 gals for dry sand and about 300 gals for saturated sand. It may be presumed that this phenomenon is due to the liquification of the sand and directly results in the remarkable increase of damage noted for "Disastrous" earthquakes over

damage noted for "Strong" earthquakes. Recently T. Tateishi has computed the bearing capacity of clay stratum for inclined loads during an earthquake by assuming a circular sliding surface analogous to Fellenius's surfaces. He found that four types of failure could appear, namely:

      a. Shallow circle failure in the same and opposite directions with the inclined loads ( $N_1$ and $N_2$ ).

      b. Deep base failure similar to that predicated by Taylor's computations ( $N_3$ ).

      c. Sliding along the base of the structure ( $N_4$ ).

The reasons for failure and the findings of his investigations are:

      1. Sliding along the base is the most frequent condition of failure.

      2. The influence of the mass force of foundation soils on the bearing capacity in an earthquake is negligible except in cases of deep base failure. Therefore, the increase of load inclination and eccentricity changes by earthquake motion are the main causes of decreasing the bearing capacity.

      3. Deep base failures occur very seldom.

      Fig. 12 shows the $N_b$-values in Tateishi's equation $W = cBN_b$ as the function of $\alpha$, $\beta$, and $e$.

## CONCLUSION

      Using experimental results, design procedures for soil structures resistant to earthquake force have been presented. They are far from being perfect, at present, but from the results of their use the damage to soil structures due to earthquakes can be explained and the forces acting on a structure during an earthquake can be estimated approximately. There remains many facets of the problems discussed herein which need future study in order that design procedures may become increasingly more reliable. In the bibliography of this paper are listed some papers published in Japan on the problems of soil characteristics during earthquakes which have not been discussed in this paper.

      I wish to express my appreciation to the Earthquake Engineering Research Institute for allowing me to present this paper. I wish to thank Dr. Takeo Mogami, Dr. Haruo Matsuo, Mr. Sukeo Ohara, Dr. Masao Kondo, Mr. Tetsuyo Tateichi, and Mr. Matsuhei Ichihara for permission to mention results of their studies.

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## NOMENCLATURE

|  |  | Unit |
|---|---|---|
| $\alpha_h =$ | Maximum horizontal acceleration of an earthquake | cm/sec$^2$ |
| $\alpha_v =$ | Maximum vertical acceleration of an earthquake | cm/sec$^2$ |
| $K = \dfrac{\alpha_h}{g}$ | | None |
| $i = \tan^{-1}\dfrac{\alpha_h}{g \mp \alpha_v}$ or $\tan^{-1}\dfrac{\alpha_h}{g}$ | | Degree |
| $i' = \tan^{-1}\dfrac{\rho\alpha_h}{\rho'g}$ | | Degree |
| $C =$ | Coefficient of active earth pressure during an earthquake | None |
| $\rho =$ | Unit weight of soil | kg/cm$^3$ |
| $\varphi =$ | Angle of internal friction | Degree |
| $\theta_0 =$ | Angle of slope of the back fill | Degree |
| $\psi =$ | Slope of the inner face of the wall to the vertical | Degree |
| $\delta =$ | Angle of wall friction | Degree |
| $P_0 =$ | Intensity of surcharge | kg/cm$^2$ |
| $H =$ | Height of retaining wall or caisson | cm |
| $H_2 =$ | Height of back fill water level measured from the base of the caisson | cm |
| $P_2 =$ | Reduced pressure of sea water during an earthquake | kg/cm$^2$ |

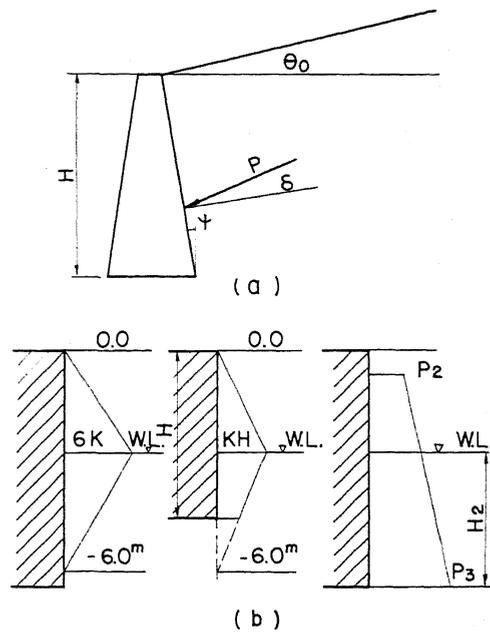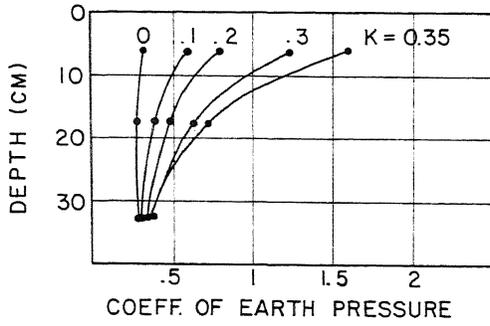| | | Unit |
|---|---|---|
| $P_3$ = | Reduced pressure of water at the base of the wall during an earthquake | $kg/cm^2$ |
| $\varepsilon$ = | Angle of the direction of the external load to the vertical (see Fig. 9) | Degree |
| $\gamma$ = | Angle (see Fig. 9) | Degree |
| $\gamma_0$ = | Length of the side BD (see Fig. 9) | cm |
| $\gamma$ = | Radius vector of the curve DE from the point B (see Fig. 9) | cm |
| $\theta$ = | Angle between $\gamma$ and $\gamma_0$ | Degree |
| $\alpha_1$ = | Angle (see Fig. 9) | Degree |
| $\alpha_2$ = | Angle (see Fig. 9) | Degree |
| $\beta_1$ = | Angle (see Fig. 9) | Degree |
| $\beta_2$ = | Angle (see Fig. 9) | Degree |
| $p$ = | Bearing capacity per unit area | $kg/cm^2$ |
| $B$ = | Width of the test plate | cm |
| $\rho'$ = | Submerged unit weight of soil | $kg/cm^3$ |
| $v$ = | Void ratio of the soil | None |
| $\gamma = \sin^{-1}\dfrac{\sin i}{\sin \varphi}$ | | Degree |
| $N_b$ = | Coefficient of bearing capacity (see Fig. 12) | None |
| $c$ = | Cohesive strength of soil | $kg/cm^2$ |
| $e_0$ = | Eccentricity of load (see Fig. 12) | None |
| $\alpha = \tan \varepsilon$ (see Fig. 12) | | None |
| $\beta = K\rho B/2c$ | | None |
| $e = 1 + 2e_0/B$ | | None |
| $W$ = | External load (see Fig. 12) | kg |

# EARTHQUAKE EFFECTS ON SOILS AND FOUNDATIONS

## FIGURE CAPTIONS

Fig. 1    Shearing strength of Soma sand during vibrations

       $\triangle$ = 48 cps.   ● = 35 cps.   ○ = 28 cps.   X = 23 cps.

Fig. 2    Ratio of the yield value to varying values of acceleration for clay in dynamic and static cases

Fig. 3    Shearing strength of clay with various moisture contents.

    ● = at rest
    X = frequency of 48 cps, amplitude of 0.1 mm and acceleration of 980 gals.
    $\triangle$ = frequency of 34 cps, amplitude of 0.1 mm and acceleration of 490 gals.
    ○ = frequency of 48 cps, amplitude of 0.05 mm and acceleration of 490 gals.

Fig. 4a   Figure showing a retaining wall

Fig. 4b   Figures showing the distribution of the design earth and water pressure on a caisson during earthquakes

Fig. 5    Oscillating pressures and the coefficients of oscilating pressure of dry sand on a wall during vibrations

Fig. 6    Oscillating pressures and oscillating water pressures of saturated sand on a wall during vibrations

Fig. 7    Concrete bowl container for the vibration generator

Fig. 8    Failure lines beneath a loaded plate in dry (a) and saturated (b) sand during vibrations

Fig. 9    Failure line in sand

Fig. 10   Bearing capacity of dry (a) and saturated (b) sand during vibrations

    ○ = $\mathcal{E}$ = 15° ,    ● = $\mathcal{E}$ = 20° ,    $\triangle$ = $\mathcal{E}$ = 25°

Fig. 11   Graphs showing the bearing capacity of obliquely loaded sand during vibrations

Fig. 12   Bearing capacity of clay in an earthquake.

    (a) = sliding surfaces
    (b), (c) and (d) = coefficients of the bearing capacity

FIG. 1



FIG. 2



FIG. 3



FIG. 4

FIG. 5



FIG. 6



FIG. 7



(a)

(b)

FIG. 8

FIG. 9



(a)



MAX. ACCELERATION (GAL)

(b)
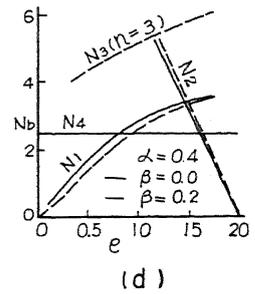
FIG. 10



FIG. 11



FIG. 12

# EARTHQUAKE EFFECTS ON SOILS AND FOUNDATIONS

COEFFICIENTS OF ACTIVE EARTH PRESSURE DURING EARTHQUAKES.

| $\varphi$ | $\theta$ \ K | .00 | .10 | .15 | .20 | .25 | .30 | .35 | .40 |
|---|---|---|---|---|---|---|---|---|---|
| 45 | 0 | .172 | .217. | .243 | .271 | .301 | .334 | .370 | .409 |
|  | 10 | .186 | .238 | .269 | .303 | .342 | .383 | .431 | .485 |
|  | 20 | .204 | .269 | .309 | .355 | .408 | .473 | .555 | .667 |
|  | 30 | .234 | .325 | .388 | .476 | .553 |  |  |  |
| 40 | 0 | .217 | .269 | .298 | .329 | .363 | .401 | .444 | .495 |
|  | 10 | .238 | .299 | .335 | .375 | .420 | .473 | .534 | .604 |
|  | 20 | .267 | .349 | .397 | .457 | .532 | .639 |  |  |
|  | 30 | .318 | .452 | .574 |  |  |  |  |  |
| 35 | 0 | .271 | .328 | .352 | .396 | .434 | .477 | .527 | .580 |
|  | 10 | .300 | .370 | .404 | .461 | .517 | .584 | .669 |  |
|  | 20 | .345 | .445 | .502 |  |  |  |  |  |
| 30 | 0 | .330 | .396 | .434 | .474 | .519 | .570 | .627 | .699 |
|  | 10 | .371 | .456 | .511 | .571 | .647 |  |  |  |
|  | 20 | .441 | .582 | .649 |  |  |  |  |  |
| 25 | 0 | .405 | .476 | .517 | .564 | .616 | .680 |  |  |
|  | 10 | .463 | .564 | .631 |  |  |  |  |  |

# TABLE I