Resistance of Foundation Ground against Overturning Forces.

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Synopsis: This is a summary of experimental studies on stability of structural foundations against eccentric, inclined loads and on stresses induced thereby in the foundation bodies.

Prototype bridge piers, caissons and numerous model foundations were tested under eccentric, inclined loads statically applied, and experimental formulas for computing soil reaction, stresses in the foundation bodies, and limit resistance of the ground were established on the basis of the measurements made during the tests. As a result of those static tests, some foundamental fallacies in conventional stability computations were revealed and corrected.

Several bridge piers and caissons were tested dinamically, and it is noticed that the damping factor of vibration is remarkably high due to friction and/or adhesion of soil acting at the wall and the base of the foundation body.

I. Foundamental Studies

1.1. Requirements for stability of foundations

For safety and stability of structural foundations, the following two conditions are required to be satisfied:

(i) There is no failure of the foundation bodies.

(ii) There is neither failure nor excessive yielding of the foundation ground.

Excessive deformation of foundations is also unfavorable, but the problem of deformation may be considered in connection with the condition (i) or (ii) above. The deformation will be tolerably small where the stresses in the foundation body or in the ground is not excessively high, except in some extraordinary cases.

There were many cases where the foundation bodies failed under earthquake forces. Similar failure' were often observed by lateral loading tests on cylindrical foundations or on piles where the foundation body is not strong enough against bending. Therefore, it is very much important to examine the condition (i) above.

There were relatively few cases where the foundation ground failed or yielded excessively under earthquake forces. But, failure of the foundation ground is possible when the dimension of the foundation body, especially the burried depth, is too small and the soil is weak. Therefore, it is also important to examine the condition (ii) above.

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In connection with this point, it must be noticed that the maximum bending moment of the foundation body increases as its burried depth increase. In other words, over-size design require over-strong foundation body and it makes the design doubly uneconomical.

On the other hand, conventional methods of stability computations call for over-size designs mainly because of the theoretical assumption that the lateral resistance of the ground could not exceed Rankines passive pressure acting normal to a vertical wall face. Actual lateral resistance of the foundation ground is much more larger than the assumed value as the mechanics of the ground resistance is much more complicate than that of Rankin's hypothesis and it involves additional important factors of resistance that had not been considered in his theory.

1.2. Soil reactions and stresses in the foundation bodies

Soil reactions are not usually normal to the surface of the foundation bodies under the loads V_n , H_o and $M_o = hH_o$ (See Fig. 1). In the case of a cylindrical foundation, their tangential components act in the direction opposit to the rotational displacement of the foundation body. Those tangential components will take their maximum value of friction and/or adhesion f at very small displacement.

At the vertical face $\overline{12}$ or $\overline{34}$,

$$f = p_h \tan\delta + C_a \cong p_h \tan\phi + C \qquad \text{............ (1)}$$

where p_h, δ and C_a are the horizontal reaction, the angle of wall friction and the wall adhesion respectively. Usually, δ is nearly equal to the angle of internal friction of the soil ϕ and C_a is nearly equal to the cohesion of the soil C.

At the base $\overline{24}$,

$$f_{bu} = p_v \tan\delta_b + C_b \qquad \text{............ (2)}$$

where p_v , δ_b and C_b are the vertical reaction, the angle of base friction and the base adhesion respectively. Here again, δ_b is nearly equal to the angle of internal friction of the soil under the base ϕ_b , and C_b to the cohesion C .

Distribution of the horizontal reaction p_h changes as the distribution of coefficient of horizontal reaction K_h varies. The value of K_h and p_h at the depth z are generally expressed in the following form:

$$K_h = K_{h1}\, z^r \qquad \text{............ (3)}$$

where K_{h1} is the value of K_h at unit depth and r is a power index which changes with the condition of the ground $(0 \leq r \leq 1)$.

$$p_h = \delta_h k_h = \left(1 - \frac{z}{l_1}\right) \delta_{oh} \, k_{h1} \, z^r \qquad \cdots\cdots\cdots\cdots \quad (4)$$

where l_1 is the depth of the center of rotation, δ_h the horizontal displacement at the depth z , and δ_{oh} the value of δ_h at the ground surface. In the above equation, the foundation body is assumed to be perfectly rigid.

Distribution of the vertical pressure p_v at the base may be assumed to be linier as the error in this assumption has minor influence.

$$p_v = \delta_v k_v \qquad k_v = \kappa \, k_{h1} \, l^r \qquad \delta_v = \frac{b \, \delta_{oh}}{2 l_1}$$

$$\text{or} \quad p_v = \left| \frac{b \, \kappa \, p_{hl}}{2(l_1 - l)} \right| \qquad \cdots\cdots\cdots\cdots \quad (5)$$

From the conditions of equilibrium of the loads V_n, H_o and M_o to those reactions f_1, f_2, f_b and p_h , the following equations are obtained;

$$\frac{l_1}{l} = \frac{(r+1)\left[(r+3)M_o + (r+2) l H_o - l F_b - (r+3)\frac{b}{2} c d l + (r+3)\frac{b}{2}(H_o + F_b)\left\{\tan\delta + (r+2)\frac{\kappa W_b}{d l^2}\right\}\right]}{(r+3)\left[(r+2)M_o + (r+1) l H_o - l F_b - (r+2)\frac{b}{2} c d l + (r+2)\frac{b}{2}(H_o + F_b)\tan\delta\left\{1 - \frac{2}{r+2}\left(\frac{l}{l}\right)^{r+1}\right\}\right]}$$

W_b = section modulus $\qquad \cdots\cdots\cdots\cdots$ (6)

where d is the width of foundation perpendicular to the direction of p_h and F_b is the resultant of f_b which is equal to the smaller value of the following two;

$$F_b' = \frac{\frac{M_o}{l} + H_o - \frac{b}{2} c d}{\frac{2}{r+3} + \frac{b}{2l}\tan\delta + \frac{(r+1)(r+2) b \kappa W_b}{2 d l^3}} - H_o - \alpha \qquad \cdots\cdots\cdots\cdots \quad (7)$$

α = coefficient of deformation

$$F_{bu} = \frac{V_n - H_o \tan\delta - c d (2 l_1 - l) + c_b b d \cot\delta_b}{\tan\delta + \cot\delta_b} \qquad \cdots\cdots\cdots\cdots \quad (7)'$$

Then, the horizontal reaction;

$$p_h = \frac{(r+1)(r+2)(H_o + F_b) z^r}{\left\{r+2 - (r+1)\frac{l}{l_1}\right\} d l^{r+1}} \left(1 - \frac{z}{l_1}\right) \qquad \cdots\cdots\cdots\cdots \quad (8)$$

The bending moment of the foundation body at the depth z ;

$$M_z = M_o + H_o z - \frac{(r+1)(H_o + F_b) z^{r+1}}{\left\{r+2 - (r+1)\frac{l}{l_1}\right\} l^{r+1}} \left\{\frac{z}{r+3}\left(\frac{r+3}{r+1} - \frac{z}{l_1}\right) + \frac{b}{2}\left(\frac{r+2}{r+1} - \frac{z}{l_1}\right)\tan\delta\right\} - \frac{b}{2} c d z \qquad \cdots\cdots\cdots\cdots \quad (9)$$

And, the vertical stress at the depth z ;

$$V_z = V_n - \frac{(r+1)(H_o + F_b) z^{r+1}}{\left\{r+2 - (r+1)\frac{l}{l_1}\right\} l^{r+1}} \left(\frac{r+2}{r+1} - \frac{z}{l_1}\right)\tan\delta - c d z \qquad \cdots\cdots\cdots\cdots \quad (10)$$

When $F_b' < F_{bu}$ and the supporting stratum is stiff enough as compared to the overlaying soil, the horizontal displacement of the base of the foundation body is neglegibly small. Then, $\ell_1 \cong \ell$

$$F_b \cong F_b'' = \frac{\frac{M_o}{\ell} + H_o - \frac{b}{2}cd}{\frac{2}{r+3} + \frac{b}{2\ell}\tan\delta + \frac{(r+1)(r+2)bkW_b}{2d\ell^3}} - H_o \quad \text{..............(7)''}$$

And,

$$p_z \cong (r+1)(r+2)(H_o + F_b)\frac{z^r}{d\ell^{r+1}}\left(1 - \frac{z}{\ell}\right) \quad \text{..............(8)'}$$

$$M_z \cong M_o + H_o z - \frac{(r+1)(H_o+F_b)z^{r+1}}{\ell^{r+1}}\left\{\frac{z}{r+3}\left(\frac{r+3}{r+1} - \frac{z}{\ell}\right) + \frac{b}{2}\left(\frac{r+2}{r+1} - \frac{z}{\ell}\right)\tan\delta\right\} - \frac{b}{2}cdz \quad \text{..............(9)'}$$

$$V_z \cong V_n - (r+1)(H_o + F_b)\frac{z^{r+1}}{\ell^{r+1}}\left(\frac{r+2}{r+1} - \frac{z}{\ell}\right)\tan\delta - cdz \quad \text{.........(10)'}$$

The condition $\ell_1 \cong \ell$ is satiafied often at the foundations of heavy structures resting on hard strata. In this case, the center of rotation comes down to the level of the foundation base.

1.3. Limit amount of eccentricity of an eccentric, vertical load.

The limit amount of eccentricity of an eccentric, vertical load V_n may be expressed by the following equations.

$$\text{sand;} \quad e_n = \frac{b}{2}\left\{X_1\left(1 - \sqrt[n]{\frac{V_n}{V_{ou}}}\right) + X_2\left(1 - \frac{V_n^3}{V_{ou}^3}\right) + X_3\left(\frac{V_{ou}}{V_n} - 1\right)\right\} \text{......(11)}$$

$$\text{clay;} \quad e_n = \frac{b}{2}\left\{X_1'\left(1 - \frac{V_n X_1'}{V_{ou} - c(d+b)\ell}\right) + (X_2' + X_3')\left(\frac{V_{ou}}{V_n} - 1\right)\right\} \text{......(12)}$$

where the foundation body is a parallelepiped which is placed on, or burried into, the uniform ground with a horizontal surface. And,

V_{ou} = the limit value of centric, vertical load.

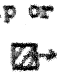
$$X_1 = \frac{b}{b+4\ell} \qquad X_1' = \left(1 - \frac{\ell}{bN_c}\right)$$

$$X_2 = \frac{1.16\ell}{b+\ell}\sqrt{\frac{\ell}{d}} \qquad X_2' = \frac{1}{N_c}\left(1 + \frac{\sqrt{b^2+\ell^2}}{2d}\right)\frac{\ell}{b}$$

$$X_3 = 0.047\frac{\ell^2}{b^2}\left(1 - \frac{2\frac{d}{\ell}}{2 + \frac{d^3}{\ell^3}}\right) \qquad X_3' = \frac{1}{N_c}\left(\pi - 1 + \frac{\frac{N_{co}}{2} - \pi + 1}{1 + 2\frac{d}{\ell}}\right)\frac{\ell^2}{b^2}$$

N_c = coefficient of bearing capacity of the foundation in consideration.
N_{co} = coefficient of bearing capacity of a strip foundation d in width.

The values of n are as follows:

Shape	loose sand	dense sand	clay
Strip or wall-like	2.0	2.45	1.0
	1.6	2.0	1.0
	2.0	2.45	1.0
	2.45	3.0	1.0

The terms containing $\frac{V_{ou}}{V_n}$ represent the factor of lateral resistance, while the other terms represent the factor of resistance by vertical reactions.

The skeltons of the above equations are formed by qualitative considerations on the mechanics of ground resistance. But, coefficient X, X' etc. are determined on the basis of experimental results.[1), 2)]

The computed values of $\frac{2}{b}e_n$ are shown in Fig. 2.

The limit values of eccentric loads may be analized by a failure-surface method in a way similar to the one used in the bearing-capacity theories. However, the problems here are non-symetric, three-dimensional ones, and the shape of the failure-surface changes remarkably with loading conditions, shape of the foundation body, condition of the ground, etc. Therefore, the failure-surface method may give very complicate solutions inadequate for practical application, except in such a simple case as the one of a strip foundation on the uniform ground with a horizontal surface.

The failure-surface methods as applied to the strip foundations mentioned above, irrespective of the shape of the failure-surface, gives answers identical to the ones obtained from the equation (11) or (12) above, if n is assumed to be 2.0 for sand or 1.0 for clay.

Values of V_{ou} obtained by loading tests on sand are usually larger than theoretical values published by Terzaghi[3)] or by Meyerhof.[4)] Cause of the difference may possibly be an influence of the compaction of sand during the loading tests. The experimental values on dense sand are nearly twice as much as the theoretical values.

On the other hand, observed values of V_{ou} of foundations burried into cohesive soil are smaller than theoretical values. Cause of the deficency is considered to be two fold. Except the case of clay without internal friction, the theoretical values of bearing capacity of the foundation base are too high as the spread of failure-surfaces assumed in the theories is too large. The wall adhesion C_a is evaluated also too high in the theories. C_a takes its maximum value C only where the lateral pressure acting to the wall is sufficiently large and is much smaller than C at shallow depth. Under a limit eccentric load, however, C_a can take its maximum value at the wall on compression side. As the compression takes place on one side only, the maximum adhesion may be assumed to be effective on one-half of the entire area of the walls. And, for computing the equation (12), V_{ou} may be assumed to include that much adhesion, although actual values of V_{ou} would be smaller than the values assumed above.

1.4. Limit inclination of a centric load

The limit angle of inclination of a centric load α_{ou} on a surface foundation $(\ell=0)$ is expressed by the following equations:

$$\text{sand;} \quad \alpha_{ou} = 1.3\phi\left(1 - \sqrt[3]{\frac{V_n}{V_{ou}}}\right) \quad \text{when} \quad 0 \leq \alpha_{ou} \leq \phi$$

$$\cdots\cdots\cdots\cdots (13)$$

$$\text{clay;} \quad \alpha_{ou} = \psi_o\left(1 - \frac{V_n}{V_{ou}}\right) \quad \text{when} \quad 0 \leq \alpha_{ou} < \psi$$

$$\alpha_{ou} = \cot^{-1}\frac{V_n}{cA} \qquad \text{when} \quad \psi \leq \alpha_{ou} \leq \frac{\pi}{2} \qquad \text{............ (14)}$$

except the case of a strip foundation on sand where $0 < \alpha_{ou} \leq \frac{3}{8}\phi$, and

$$\alpha_{ou} = \frac{\phi}{2}\left(1 - \frac{V_n}{V_{ou}}\right) \qquad \text{............ (13)'}$$

In the above equations, ϕ is the angle of internal friction, c the cohesion, A the area of the foundation base, and $\psi_0 \cong 38°$, $\psi = 19°$(strip foundation) or $28°$(square foundation).

The equations (13) and (14) above were obtained by analysing experimental results [2] (See Fig. 3 and Fig. 4).

The limit inclination $\tan\alpha_{ou}$ where the burried depth of foundation $\ell > 0$ is expressed by the following equations:

$$\tan\alpha_{ou} = \frac{e_n}{\ell + e_n \cot\left\{\left(1+\frac{\ell}{b}\right)\alpha_{ouo}\right\}} \qquad \text{when} \quad \left(1+\frac{\ell}{b}\right)\alpha_{ouo} < \frac{\pi}{2} \qquad \text{............ (15)}$$

$$\tan\alpha_{ou} = \frac{e_n}{\ell} \qquad \text{when} \quad \left(1+\frac{\ell}{b}\right)\alpha_{ouo} \geq \frac{\pi}{2} \qquad \text{............ (16)}$$

where e_n is the limit amount of eccentricity of a vertical load V_n which is equal to the vertical component of the inclined load in consideration, and α_{ouo} is the limit angle of inclination of a surface foundation $(\ell = 0)$ with identical base area (See equations (13) or (14)).

The equations (15) and (16) above were induced by comparing the soil reactions (See equation (5), (8), etc.) due to limit eccentric, vertical loads with the ones due to centric, inclined loads. Limit condition is assumed to exist where the horizontal reaction R_h by the latter is identical to the former.

1.5. Limit inclination of an eccentric, inclined load

The limit inclination $\tan\alpha_u$ of an eccentric inclined load may be expressed approximately by the following equation:

$$\tan\alpha_u \cong \left(1 - \frac{e}{e_n}\right)\tan\alpha_{ou} \qquad \text{............ (17)}$$

where e is the amount of eccentricity, e_n is the limit amount of eccentricty of a vertical load which is equal to the vertical component V_n of the eccentric, inclined load in consideration. And, $\tan\alpha_{ou}$ is the limit inclination of a centric inclined load the vertical component of which is also equal to V_n .

The equation (17) above may be obtained by a way similar to the one used in inducing equation (15) and (16) where $\ell \gg 0$ and $V_n \ll V_{ou}$. It has not been proved theoretically, but the equation (17) seems to hold where $\ell \geqq 0$ and/or $V_n \not\ll V_{ou}$

If the equation (17) holds at any values of ℓ and V_n , the ratio of the horizontal component of an eccentric, inclined load H_u to the vertical V_n component V_n may be expressed in the following simple form irrespective of the value of ℓ or V_n .

$$\frac{H_u}{V_n} = \frac{e_n}{h + \ell + e_n \cot\left\{\left(1+\frac{\ell}{b}\right)\alpha_{ouo}\right\}} \quad \text{where } \left(1+\frac{\ell}{b}\right)\alpha_{ouo} < \frac{\pi}{2}$$

$$\frac{H_u}{V_n} = \frac{e_n}{h + \ell} \quad \text{where } \left(1+\frac{\ell}{b}\right)\alpha_{ouo} \geq \frac{\pi}{2}$$

In the case when the resistance of the foundation ground against earthquake forces is examined statically, it may be taken to be regarded as a necessary condition of stability that $\frac{H_u}{V_n}$ as given by the equation (18) or (19) is not less than design earthquake coefficient k , and there should be a sufficient margin of safety between $\frac{H_u}{V_n}$ and k .

II. Experiments

2.1. Outline of experiments

The experiments shown in Table 1 were made to observe soil reactions, stresses in the foundation bodies or limit values of eccentric loads and eccentric, inclined loads.

The applied pull was measured with a proving ring or a spring ballance, and the thrust with a load cell in which electric-resistance type strain gages had been attached.

The measurement of displacement was made by reading milimeter scales with transits, or by reading dialgages. Other types of preceise instruments such as microscopic comparators, solenoid-type displacement gages, electro-magnetic vibration meters, oscillographs etc. were also used where they were required.

The normal component of soil reactions was measured by soil pressure cells.

2.2. Soil reactions and stresses in foundation bodies

A typical example of the observed soil pressure is shown in Fig. 5 wherein the horizontal reaction is assumed to be the sum of pressure increase on the compression side and pressure decrease on the tension side, friction on the side wall not being added to it.

The observed reactions seems to approximate the values computed by the equations (8) or (8)' wherein the values of r are taken as follows:

		Value of r
Caissons in very soft soil:		0.5
Caissons in soft soil overlaid with denser soil:		0
Pole foundations in brittle Kanto loam		0.25
Model foundation	(in very loose sand:	1.0
	(medium sand:	0.7
	(dense sand:	0.25

Fig. 6 is a presentation of the friction (adhesion of soil may be included) and the normal forces on the wall on the compression side and the base observed with a set of special arrangements. The observed value of F_{bu} or P_{IV} as shown in Fig. 6 is roughly equal to the value computed by the equations (7)' or (1), provided the experimental error in measuring P_{Ih} is eliminated.

Those frictions P_{IV} and F_{bu} have a remarkable influence to reduce the horizontal reaction P_h and to increase the bending moment in the foundation body M_x (see equations (8) and (9)). Furthermore, the influence of those frictions to reduce the vertical stress in the foundation body V_x (see equation (10)) often results in a very high tensile stress on the tension side. Those high tensile stresses were recorded elsewhere at several steel models of caissons.

2.3. Limit values of eccentric, inclined loads

The values of limit horizontal pull H_u observed during the tests on railway bridge piers which does not fail by tension of pier bodies coincide well with the computed values of equations (10) and (12) as shown in Table 1. The foundation ground of the site is bouldery sand. The value of V_{ou} as shown in Table 1 are computed by the following equation.

$$V_{ou} = N_\gamma \left(1 - 0.3 \frac{b}{d}\right) \frac{\gamma}{2} B d b \qquad B = \text{smaller one of d or b}$$

where N_γ is the coefficient of bearing capacity (estimated to be 500 on the basis of plate-loading tests), γ the unit weight of the sand (1.8 t/m^3).

The limit value of overturning moment M_{ou} as observed during the tests on twenty four experimental pole foundations in the natural ground with a horizontal surface show close approximation to the values computed by the equation (11) and the modified form of equation (13).

$$M_{ou} = \frac{e_n V_u}{l + h}$$

Those values of M_{ou} are shown in Table 2. The difference between the average observed values and the computed values is less than 10%, except where the foundation body failed first (*). Considering the dilicate nature and nonuniformity of the soils, the error of 10% may well be said small.

The values of V_{ou} in Table 2 was computed by the following equation.

$$V_{ou} = c b d N_c + \gamma b d B \left(\frac{N_\gamma}{2} + N_q \frac{l}{B}\right)$$

where N_c , N_γ and N_q are the coefficients of bearing capacity (assumed $N_c = 30 \sim 31$, $N_\gamma = 1.95$ and $N_q = 6.5$ on the basis of soil test data; $\phi = 23°$). The measured values of cohesion scatter in a wide range from 2.6 t/m^2 to 6.7 t/m^2 . Considering the disturbance of soil due to the deformation during the tests, the value of 3.0 t/m^2 which is close to the lower limit is chosen to be used in the computations. The value of limit overturning moment M'_{ou} of the foundation near the top of the slope (1:1.5) may be expressed approximately by the following equation.

$$M'_{ou} = \left(0.70 + 0.15 \frac{L}{d}\right) M_{ou} \qquad \frac{L}{d} < 2$$

where M_{ou} is the value on horizontal ground, L the distance between the center of the foundation and the top of the slope.

The values of M_{ou} observed at twelve foundations placed in the filled soil decrease approximately in linier proportion with the decrease of penetrating resistance of WES-type cone-penetrometers. They are less than 25% of the ones observed on the natural ground at a compacted fill, and less than 19% at a loose fill.

2.4. Tests on models

The values of M_{ou} or H_{ou} observed during the model tests made on Kanto loam scatter widely in a way similar to the scattering of the values of cohesion c. But, they seems to approximate to the computed values if the scattering of soil strengths were eliminated. Cause of the scattering is considered to be in non-uniformity of the soil, although it did not affect the prototype tests where the scattered weak spots were much smaller than the size of the foundation.

The limit values of eccentric, vertical loads observed during the model tests on dry sand and on soft clay show a good approximation to the computed values of the equation (10) and (11) as shown on Fig. 1. The scattering of the values observed on clay is due to the imperfect loading arrangements and to the local variety of soil strengths.

2.5. Dynamic tests

Fig. 7 is an example of records taken during the dynamic tests on a caisson which was made by cutting the horizontal pull or thrust suddenly. Similar records were obtained on bridge piers where the burried depth of foundation l and the displacement of the foundation before cutting was relatively large. As shown in Fig. 7 damping of the free oscillation was so remarkable that it is close to the state of critical damping.

On the records of vibration produced by vibration generators, similar tendency are observed. The peaks of resonance on the resonance curves are so low that it is difficult to identify those peaks from forced oscillations.

The large damping is considered to be due to the friction on the vertical wall on the compression side and on the base. The friction changes its direction as soon as the return displacement begins.

Another interesting phenomena was observed by the sudden increase of lateral pull on pole foundations. The displacement of the foundation is identical to the one observed during slow tests only when the pull is small ($H \leq \frac{H_u}{2}$), and the difference between the dynamic displacement and the static one grows over four times as large as the former. There is little yielding of soil during the dynamic tests even when the pull reaches its statical limit

Conclusion

Of the important facts revealed by the foundamental studies and experiments as have been mentioned in the preceeding articles, the most important one is that the weak points of the foundations lay usually in the foundation body except extraordinary cases. The lateral resistance of the ground is much larger than the values computed by the conventional formulas except the case of surface foundation ($l = 0$). And, the friction between the foundation body and the soil has a remarkable influence upon stress distribution and vibration characteristics of the foundations.

Although foundamental points of those problems were believed to be clarified, many things about those problems are left for studies in the future.

References

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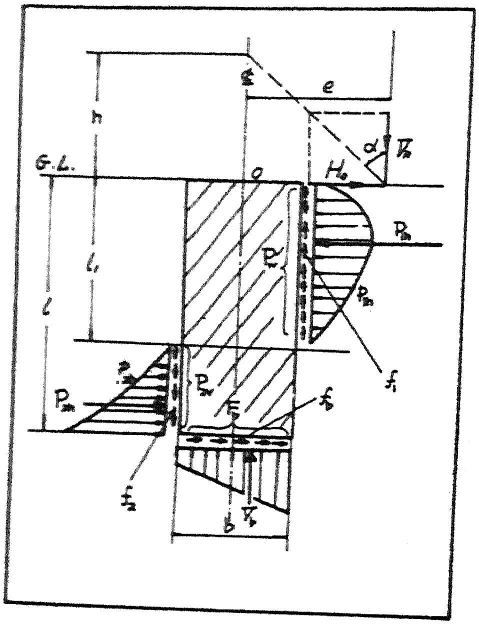

Fig. 1. Reactions on a cylindrical foundation.

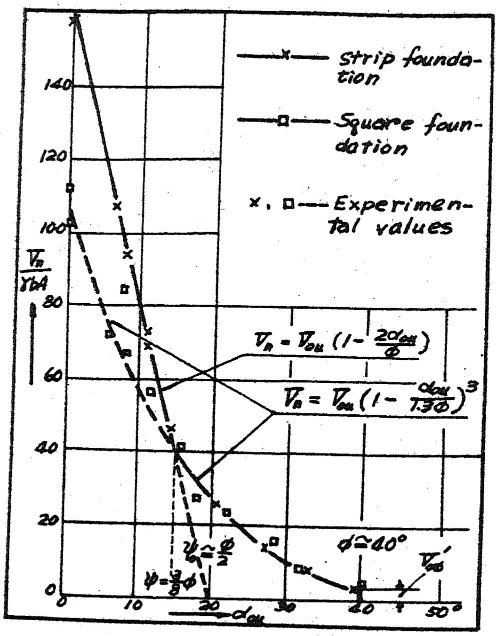

Fig. 3. Relation between V_n and α_{ou} of centric, inclined loads (sand).

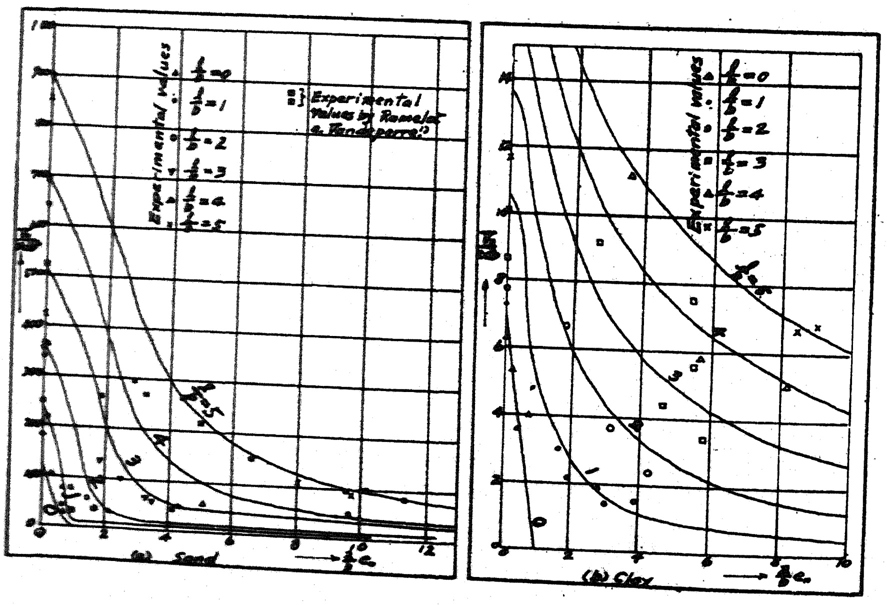

Fig. 2. Relation between V_n and e_n of eccentric, vertical loads.

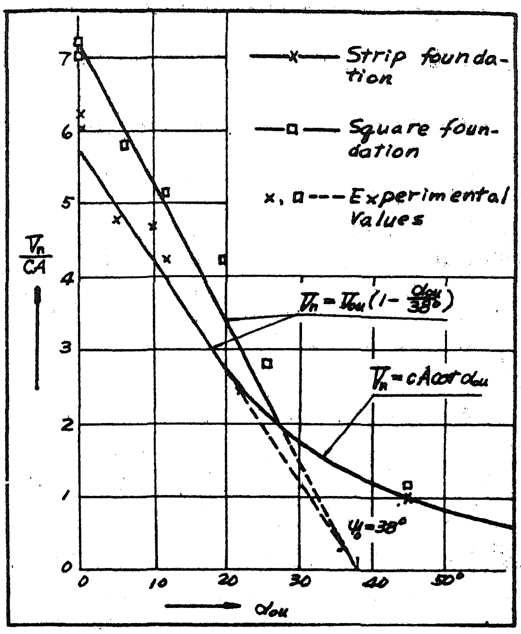

Fig. 4. Relation between V_n and α_{ou} of centric, inclined loads (clay).

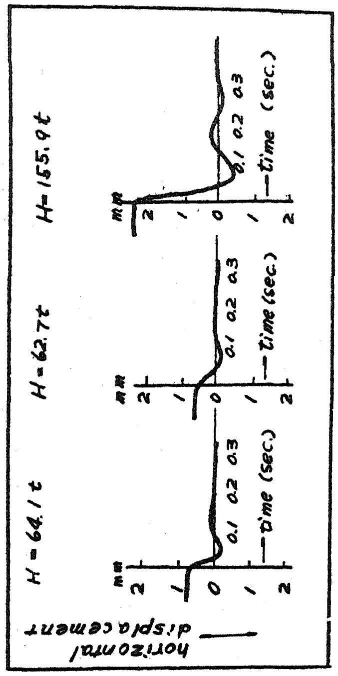

Fig. 7. Records of damped oscillation of a caisson 32.1 m. deep in soft silty ground.

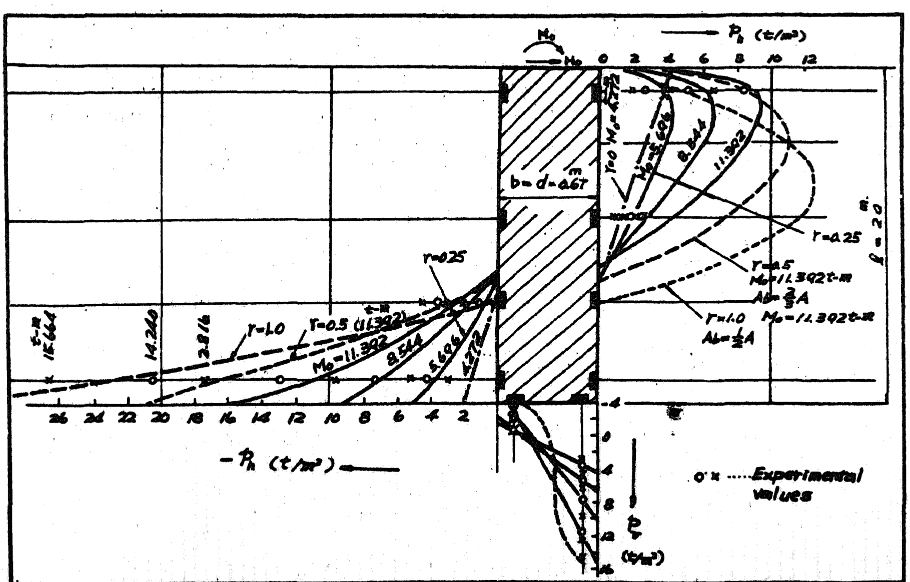

Fig. 5. Distribution of normal reactions on a pole foundation in Kanto loam.

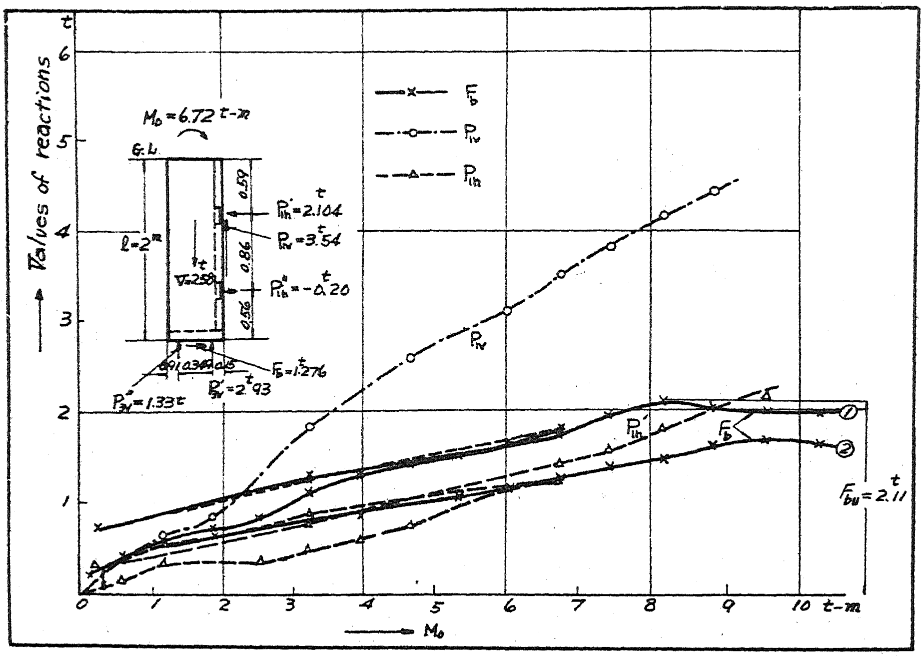

Fig. 6. Observed values of normal reactions and tangential reactions on a pole foundation in Kanto loam.

Table 1. Outline of experiment

Classification		Loading condition	Number of tests	Values observed
Prototype	Plain concrete, railway bridge piers (overturned by failure of soil) $\ell = 0$	eccentric, inclind; H by pull	4	Limit value of H, horizontal displacements, deformation of pier bodies, damping factor of vibration (soil reactions)
	Railway bridge piers (overturned by failure of brick seams.) $0 \leq \ell \leq 3.0$ m.	eccentric, inclined; H by pull	8 (2)	
	Caissons(not tested until failure took place) 22.5 m. $\leq \ell \leq 33.1$ m.	centric, inclined; H by thrust of 200 t. in maximum	5 (2)	Horizontal displacements, deformation of caisson bodies, soil reactions, (damping factor of vibrations)
	Reinforced concrete foundation of electrification poles	eccentric, inclined H by pull	77 (2)	Limit values of H, rotation and horizontal displacements, (soil reactions)
Model	Steel models on Kanto loam	eccentric inclined; centric inclined	51	Limit values of rotation of models, horizontal displacement (stresses in foundation bodies)
	Steel models on dry sand	eccentric, vertical; centric, inclined;	125 (8)	
	Steel models on soft clay	eccentric, inclined,	150 (2)	

Note: Values in (   ) were observed in a number of tests shown in (   ). ℓ = burried depth, H = horizontal pull.

Table 2. Values of H_u (Tests on bridge piers)

Number	1P	3P	4P	K1P
Material Shape of bearing area.	Stone	and	concrete	concrete
Dimension of bearing area	d = 2.61 b = 2.85	3.10 2.52	3.20 2.50	4.2 4.2
Vertical load V_n	85.0	89.1	101.8	193.2
Value of V_{ou}	7862	6730	6350	27538
Height of pull h	5.25	3.80	5.95	6.90
Observed value of H_u	34	23	18	51
Computed value of H_u	36.1	24.3	19.5	48.9

Note: Unit of force = metric ton, unit of length = meter.

Table 3. Values of (Tests on pole foundations)

Number	Dimension of foundation body	Vertical load V_n	Value of V_{ou}	$e_n V_n$	Computed value of M_{ou}	Observed value of M_{ou} average	max.	min.
1	0.75 ϕ x 2.0	2.63	63.2	31.9	24.8	25.7	27.4	24.0
2	0.6 ϕ x 2.0	1.90	43.7	26.3	20.3	22.0	23.7	20.2
3	0.5 ϕ x 2.0	1.50	31.4	22.1	17.2	15.6	16.8	14.3
4	0.4 ϕ x 2.0	1.35	22.1	18.3	14.2	14.0	15.0	13.0
5	0.3 ϕ x 2.0	1.02	14.1	13.7	10.7	12.3	12.5	12.0
6	0.2 ϕ x 2.0	0.79	7.7	9.7	7.5	7.1	7.7	6.4
7	0.6 x 0.6 x 2.0	2.26	52.7	29.4	22.9	24.8	24.8	-
8	0.6 x 0.8 x 2.0	2.81	68.3	36.2	28.1	26.8	28.5	25.3
9	0.6 x 1.0 x 2.0	3.36	87.7	42.0	32.7	31.7	32.5	30.8
10	0.6 ϕ x 1.6	1.64	39.3	16.4	13.4	10.5	-	10.5*
11	0.6 ϕ x 2.4	2.16	46.6	39.5	29.4	30.0	30.0	30.0
12	0.3 ϕ x 1.6	0.97	12.5	8.5	6.9	7.1	7.7	6.6
13	0.3 ϕ x 2.4	1.08	15.6	20.4	15.2	14.0	14.4	14.0

Note: Unit of force = metric ton, unit of length=meter, unit of moment = ton-meter.