CONVENIENT ASEISMIC DESIGN OF PILE FOUNDATION

BY
YASUYUKI Esashi[I], Yasuo Yoshida[II]

SUMMARY

In recent year, many pile foundation have been used to bear the structures on the thick poor ground, but those aseismic design methods are not established even now that become more complex and more difficult because of complicate structure of pile foundations. Therefore, there are much uneasiness for stability of structure beared with piles, if strong earthquake were felt.

This paper describes for improvement and simplification of aseismic design of pile foundation, through the following studies (see Table 1).

(1) Observation about the prototype steel piles at the Matushiro earthquake swarm in Japan 1966.
(2) Some filed vibration tests for the piles.
(3) Simulation of the observation for piles.
(4) Development for a in-situ test method of the dynamic properties of ground.

OBSERVATION FOR THE PROTOTYPE STEEL PILES AT THE MATUSHIRO EARTHQUAKE SWARM AND AT THE VIBRATION TEST.

For investigation of behavior of pile foundation during earthquake and vibration, acceleration and strain of two piles were observed at the Matushiro earthquake swarm and at the vibration tests, and then the acceleration response of the piles and strain response were gotton (see Fig. 1.2).

From these datam, the followings were studied.

(1) Comparison of amplitude between piles and ground.
(2) Comparison of acceleration-period curve and prominant period at the earthquake between the piles and surrounding ground.
(3) Comparison between the prominant period of piles and the resonanse period by the vibration test.
(4) Comparison of acceleration power-spectrum response between the piles and surrounding ground at same three depths (top of pile: surface ground, Gl-5m, pin of pile: Gl-10m).
(5) Strain distribution of the piles at the earthquake, at the vibration test and static loading test.

The results of these studies: (1) The acceleration ratio and the displacement ratio of the piles to surrounding ground were almost more than one at near the pile top, but at more deep part these ratios were almost equal to one (see Fig. 3,4). These phenomena were also recognized for the comparison of the acceleration-period curve of the piles and the surrounding ground and for power spectrum response (see Fig. 5).

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I: Fellow Research Engineer, Manager of Engineering Seicmology Section, Foundation & Earthquake Engineering Department Laboratory. CRIEPI, Japan.
II: Senior Research Engineer.

It was remarkable that the prominant periods of the piles were almost same to the resonanse period by the vibration tests (see Fig. 5).
(2) Other hand, the distributions of bending stress, shear stress and coefficient of soil reaction at main seismic wave were like to those at the vibration test and at the static loading test (see Fig. 6), and then the maximum stress concentrations occured near the top of the piles by the case of fixed condition of pile head. The correlation between these stress of the pile head and the displacement amplitude were better than the correlation between the stress and acceleration amplitude of them (see Fig. 7).
(3) From these phenomena of the piles, it is suggested that, for the convenient aseismic design of pile foundation, one lumped-mass system can be able to indicate the dynamic behavior of pile foundations.

THE RESULTS OF THE OTHER VIBRATION TEST

(1) It was recognized that the horizontal spring coefficients of single piles were larger than those of group piles, of which distance were under four times of the diameter. From the datum of acceleration for the piles and the ground which was enclosed by the piles, it was shown that the dynamic effects of pile groups were caused by dynamic behavior of the ground which was almost same amplitude and phase to the piles, and then it was shown that the apprisment as virtual mass of the ground enclosed by piles and (lm) was effective (see Fig. 8). (lm): meaning depth of maximum bending moment.
(2) Deformation reliance of horizontal spring was very similar to strain reliance of the soil dynamic deformation modulus, and namely the horizontal spring coefficient decrease in proportion to the increase of deformation amplitude of the piles (see Fig. 8). But in contrast to this, deformation reliance of damping coefficient of piles was not similar to the soil viscous damping and the damping coefficient also decrease in proportion to the increase of the deformation amplitude (see Fig. 9). It is seemed that these show the non-linear hysteresis type spring of pile foundation being very distinct in comparison with other and these phenomena are caused by the residual displacement of the surrounding ground of the pile top.

SIMULATION FOR THE PILE FOUNDATION BY USING ONE LUMPED MASS MODEL

From the above mentioned results, it was seemed effective for the correct simulation to use the one lumped mass model and necessary to consider the effect of pile group and non-linear of dynamic coefficients. Then it was tried to simulate the acceleration of the pile top at the earthquake, by using one lumped mass model and by equivalent linear method with experimental formulas which indicated non linear of the dynamic coefficient depend on displacement amplitude (see Fig. 10). As shown in Figure 11,12, the results of the simulation are almost coincide with the observed seismic waves, but it is necessary for the convenient aseismic desing to investigate the nonlinear dynamic ground coefficient more easily and exactly.

AN EXPLORATORY METHOD FOR DYNAMIC PROPERTIES OF GROUND THROUGH BOREHOLE WALL

Author have tried to develop the in-situ exploratory equipment of ground through borehole wall (1).

The equipment is composed of dynamic loading part through the rubber tube, a driving and controlling part adjoining the loading part, an oil power supply, a load setting instrument and a recorder part, as shown in Fig. 13 and Fig. 14. The driving and controlling part are composed of a piston cylinder and a servo valve.

The performance of the equipment is illustrated in table 2 . Kh, Ed and "he" are defined by the loop of pressure and displacement of the wall. Namely kh, ED and "he" are given by following equations.

$$kh = \frac{r}{p}\ ,\ \mathrm{Ed} = (1+\nu)\cdot \mathrm{ro}\cdot kh,\ \text{"}he\text{"} = \frac{1}{2}\ \frac{\text{energy loss of one cycle}}{\text{elastic energy of one cycle}}$$

From the comparison of the test result by this equipment with the vibration test result for the piles at same place, it is shown that conversion kh by Vesic-Kishida's conversion formula(2) is almost coinced with kh by the vibration test (see Fig. 15) and "he" with this equipment is smaller than "he" by the vibration test (see Fig,916).

CONCLUSION

The following results were obtained by these studies.

(1) It was shown by the correlation between main dynamic behavior and bending stress distribution of piles during the earthquake that a expression for the pile foundation as one lumped-mass system is effective. Also, it is shown that it is possible to simulate correctly the main dynamic behavior of the pile foundation during the earthquake by using the one lumped-mass system and by computation with equivalent linear method.

(2) By the result of the vibration tests, dynamic effects of pile group were clearly recognized for the pile groups of which pile distance were about under four times of the diameter. The most distinct dynamic pile group effect compared with single pile is the decrease in horizontal spring coefficient, and it is shown that appraisement as virtual mass of ground enclosed by piles and (1m) is effective.

(3) Deformation reliance of horizontal spring is very similar to strain reliance of soil dynamic deformation modulus, and namely the horizontal spring coefficient decrease in proportion to the increase of deformation amplitude of pile. But in contrast to this, deformation reliance of damping coefficient of pile is not similar to the soil viscous damping, and the damping coefficient also decreases in proportion to the increase of the deformation amplitude. It is seemed that these show the non-linear hysteresis type spring of pile foundation being very distinct in comparison with other.

(4) The new equipment for in-situ test has been developed, which can generate dynamic pressure on borehole wall through rubber tube at arbitarary depth, and the test results were compared with the pile vibration test results.

Based on the results obtained, the convenient aseismic design of pile foundation which is composed of the one-lumped mass system, the equivalent linear system,the pile group effect and the in-situ test is proposed.

ACKNOWLEDGEMENTS

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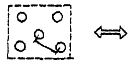
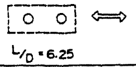
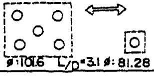
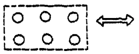
REFERENCE

(1) Esashi, Y & Yoshida, Y (1977): AN EXPLORATORY METHOD FOR DYNAMIC PROPERTIES OF GROUND THROUGH BOREHOLE WALL. Sixth W.C.E.E

(2) Kishida, H and Nakai, S (1977): Nonlinear Relation between Coefficient of Reaction and Displacement. Soil and Foundation Vol.25, No.8, Ser. No. 234 (in Japanese)

Table-1 The outline of the experiments

Number of experiment	Ground condition	Size of piles	Disposition of pile L : Space of piles D: diameter of pile ⟺: direction of vibration	Kinds of experimet
ExpNoI	very poor back fill ground	diameter ø60.96cm thickness : 9mm length l : 24 m quality : steel ss41	 L/D ≑ 2.7	vibration test
Exp No II	poor cohesive soil qround	ø : 40.64 cm t : 6.4 mm l : 10 m steel SS 41	 L/D = 6.25	Observation of Matushiro Earthquake swarm Vibration test Horizontal loading test
Exp No III	Sandy ground	ø : 101.6, 81.28 cm t : 12.7 mm l : 40 m Steel SS 41	 ø:101.6 L/D=3.1 ø: 81.28	Vibratien test Horizontal loading test
Exp No IV	Sandy model ground	ø : 2.5 cm t : 6.4 mm l : 4.5 m Steel SS 41	 L/D = 2.3, 3.7, 5.6	Vibration test Horizontal loading test

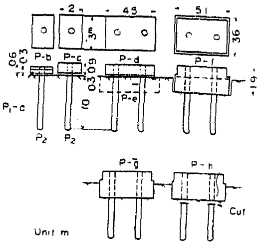


Fig. 1 Pile head Cndition used to observation of the Earthquake and vibration test (Exp No II)

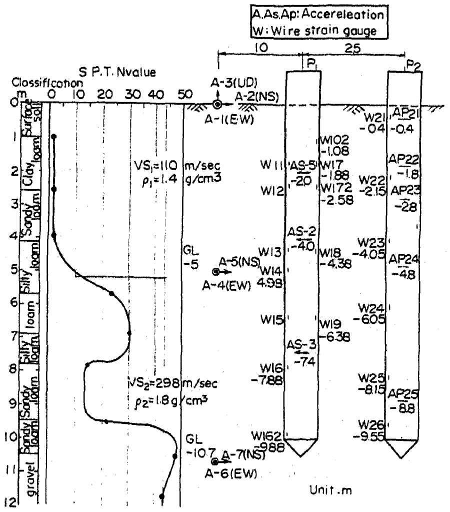


Fig.2 Columnar Section and Position of gauge for the earthquake observation. (Exp No. II)

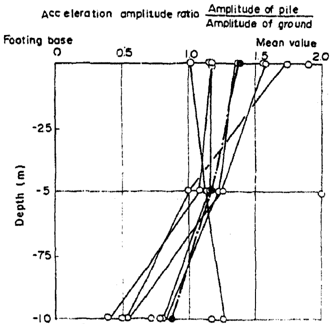


Fig.3 Comparison of horizontal acceleration amplitude between pile and ground (Exp No II. pile head condition p-g)

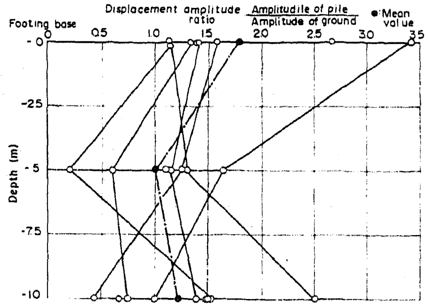


Fig. 4 Comparison of horizontal accelerotio amplitude between pile and ground (Exp No II. pile head Condition P-g)

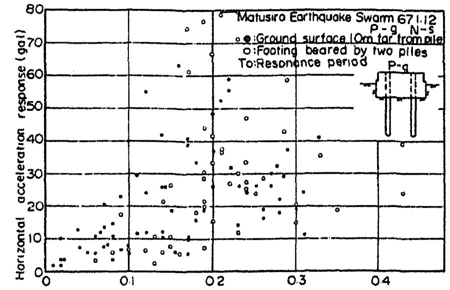


Fig a Comparison of acceleration between pile head and ground surface

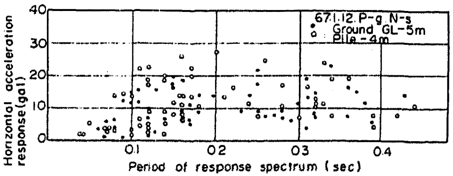


Fig-b Comparison of acceleration between pile -4m and ground GL-5m

Fig 5 Comparison of horizontal acceleration between pile and ground

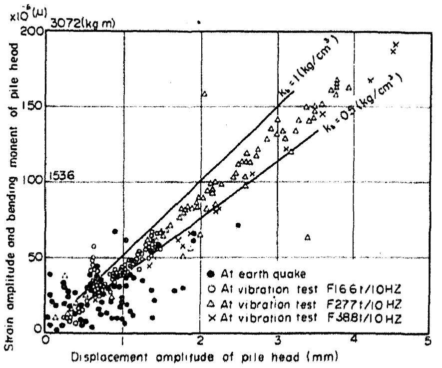


Fig 7 Correlation between displacement of pile head and bending stress at earthquake and horizontal vibration

Fig.a Bending moment distribution mode

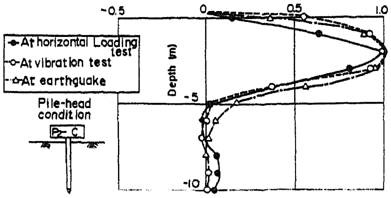


Fig b shear stress distribution mode

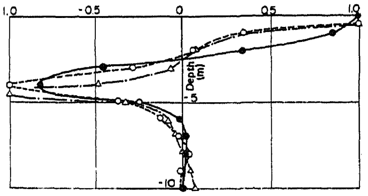


Fig b

Fig c soil reaction distribution mode

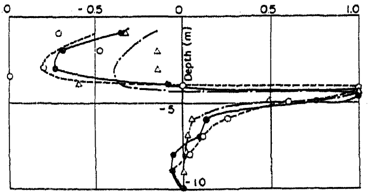


Fig.6 Comparison example of pile behavior at earthquake horizontal loading and vibration (Exp No.II pile head condition P₂-c, Earthquake 66.10.19)

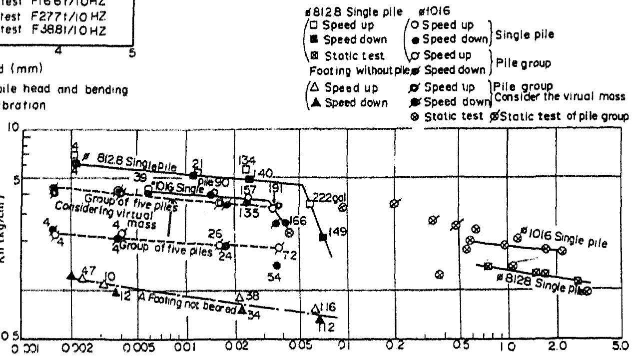


Fig.8 Relationship between horizontal coefficient of soil reactron and displacement ampli tude

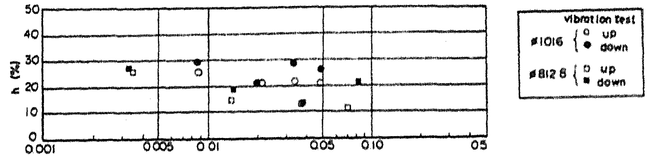


Fig. 9 Relation between damping ratio of pile and resonant displace ment amplitude

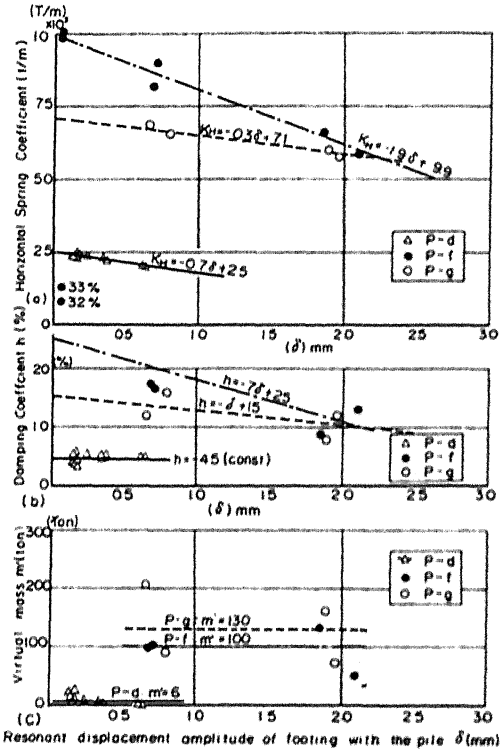
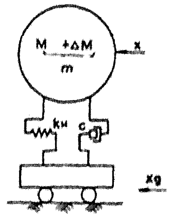


Fig 10-a Dynamic coefficients for simulation which are investigated by the vibration test



a lumped mass model

$$m(\ddot{x} + \ddot{x}_g) + c\dot{x} + k_H x = 0$$

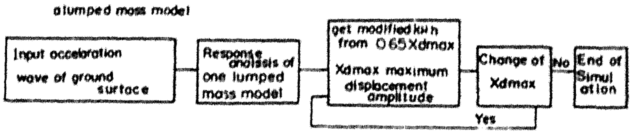


Fig.10 One lumped mass model and flow of Simulation

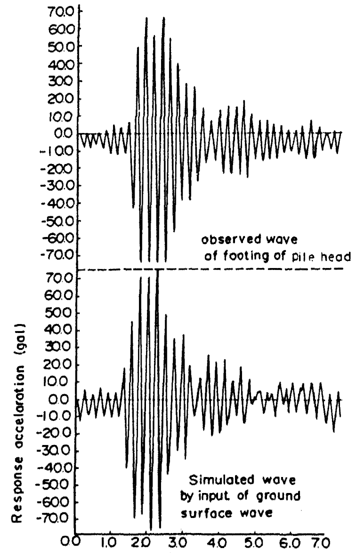


Fig.11
Comparison between observed wave of Mastushiro Earthquake wave and simulated wave
(pile head condition p-d)

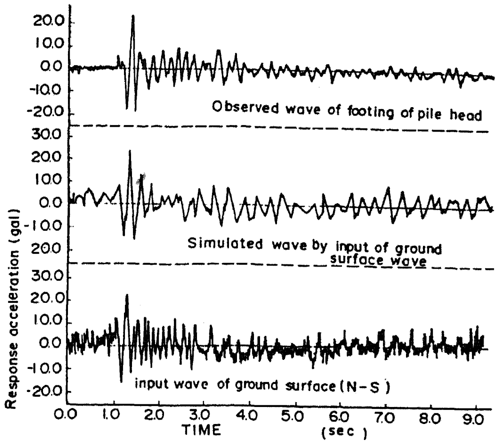


Fig.12 Collation of Simulated wave with observed wave of Matushiro Earthquake swarm (pile head condition)
:P-f

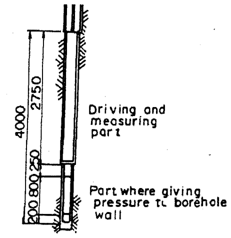


Fig.14 SIZE AND FORM OF PART WHERE GIVNG PRESSURE TO BOREHOLE WALL

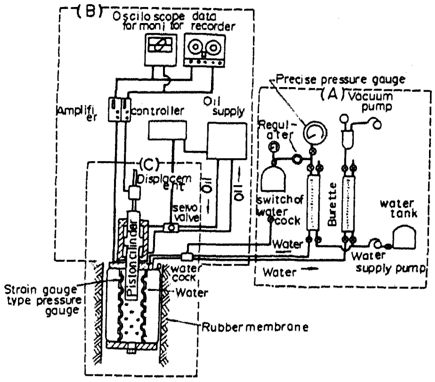


Fig.13 DYNAMIC GROUND MODULUS MEASUREMENT EQUIPMENT MECHANSM

TABLE - 2 EQUIPMENT PERFORMANCE

PRESSURE ABILITY (kg/cm²)	0 ~ 30
KINDS OF PRESSURE WAVES	SIN, TRAIANGLE, RECTANGLE
FREQUENCY ABILITY (Hz)	0.5 ~ 20
MAXIMUM DEPTH (M)	50
SIZE OF RUBBER MEMBRANE (M·M)	L TYPE φ80 × 800
	S TYPE φ80 × 400

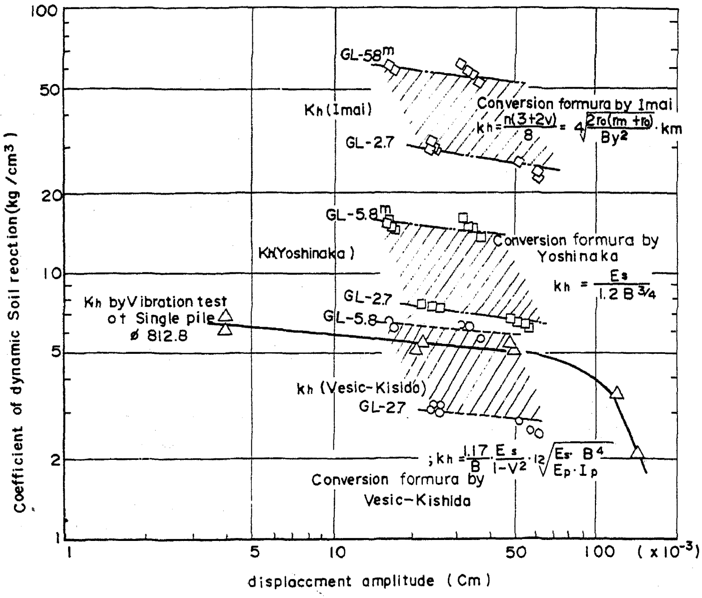


Fig.15 Comparison Conversion kh by the in-situ exploratory equipmrt and kh by vibration test (Exp NoIII ø 812.8)

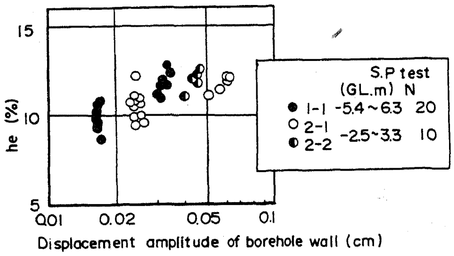


Fig.16 Relation between damping ratio by the in-Situ exploratory equipment and displacement amplitude at the same site of vibration test of pile