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AN EXPERIMENTAL STUDY OF MOCK-UP PILE FOUNDATIONS PART 1 AGING EFFECTS ON DYNAMIC SOIL SPRINGS

Yukio Shimomura[1)], Yoshio Ikeda[2)], Hiromi Adachi[3)], Mitsukazu Nakanishi[3)], Yoshiya Ogushi[3)],

Masatoshi Nakamura[3)], Toshiaki Arai[4)]

1) Department of Construction, Junior College of Nihon University, Japan
2) Nuclear Facilities Department, Taisei Corporation, Japan
3) College of Science and Technology, Nihon University, Japan
4) Research Institute, Nishimatsu Construction Co. Ltd., Japan

ABSTRACT

Massive structures, that is, reactor, turbine and other structures, have been closely constructed in reactor plants. Therefore, the consideration of the dynamic interaction effects not only between soil and structure but also among adjacent structures is indispensable to understanding the response of these buildings in detail. In some studies on dynamic cross interaction the adjacent structures have normally rested on the soil surface, and studies related on dynamic cross interaction of structures supported on pile foundation have been rarely seen except the authors' paper. We had conducted twice forced vibration tests of five mock-up pile foundations at Funabashi Campus in Chiba Prefecture in 1995 and 1996. After six years, we repeated forced vibration test of the same models. The forced vibration test was carried out to confirm the secular variation of the dynamic characteristics of the pile foundations and to obtain essential features of the dynamic cross interaction among the adjacent foundations. In this paper, we introduce an outline of the forced vibration tests and describe the results of the comparative analysis study with the tests.

KEY WORDS: pile foundation, forced vibration test, adjacent structures, sway and rocking springs, simulation analysis, dynamic cross interaction, aging effects.

INTRODUCTION

In order to understand the effects of dynamic interaction between soil and structure supported on piles, studies based on vibration generator examination tests have been carried out, and a number of precious findings have been presented [1], [2]. These studies focus to the influence of the material nonlinearity (strain dependency of soil rigidity) and geometrical nonlinearity (separation between pile and its surrounding soil) on the dynamic characteristics of pile foundation and group effects of mock-up piles. We have also conducted vibration tests of five mock-up pile foundations. Experimental facilities of the pile-foundation system were installed at Funabashi Campus of Nihon University as a part of the research program promoted by Nihon University and Nishimatsu Construction Co. Ltd. These facilities are composed of four different-sized foundation models supported on mock-up piles and an embedded raft foundation model. After installing the models, the research group carried out forced vibration tests twice in 1995 and 1996 [3]. After six years, we repeated another forced vibration test of the same models in 2001. This test was conducted to obtain a confirmation of secular variation of dynamic characteristic of the pile foundation models and basic characteristics of dynamic cross interaction among pile-supported structures.

In some studies on dynamic cross interaction the adjacent structures have normally rested on the soil surface [4], and studies related on dynamic cross interaction of structures supported on pile foundation have been rarely seen except the authors' paper [5]. This study has the following two original contents. First, we performed the forced vibration test using the same pile models after about six years again. Second, we investigate on effects of dynamic cross interaction among the pile foundations from both sides of experiment and analysis.

The study is composed of this paper (Part 1) and the companion one (Part 2). In this paper, we firstly introduce the soil properties of the experiment site and the feature of the pile foundation model group. Secondly, we present an outline of the past and last year's forced vibration tests, and describe the analysis result presumed in a preparation stage of the test. Next, we present the displacement resonance curve and sway and rocking springs calculated from the test results. Finally, we discuss the secular change of the dynamic characteristics of the pile foundation models. In the companion paper, we discuss the test results and the analytical results considering with the influence of adjacent foundations supported on piles in detail.

OUTLINE OF TEST FACILITIES

Ground characteristics of experiment site

Funabashi Campus of Nihon University is located in the middle west region of Chiba Prefecture, central region of Japan, and is on a diluvium plateau. The foundation model group has been constructed at the north side of reinforced concrete structure C schoolhouse as shown in Fig. 1. Each accelerometer having three components has been installed at depth of G.L.-156 m, -80 m, -44 m, -15 m, -6.5 m and soil surface of point A located in the northwest side of the experiment yard shown in Fig. 1. Moreover, at B point located in front side of the observation monitor hut, accelerometers have been installed at the depth of G.L.-25 m, -6.5 m and -3.5 m. Fig. 2 shows soil profile, that is, distribution of density and elasticity wave velocity from the surface to the depth of G.L.-80 m at point A. The shear wave velocity of the soil layer in which the tips of piles penetrate near the depth of G.L.-25 m probably exceeds 400 m/s. The supporting layer consists of a steady fine sand and its N value of the standard penetration test exceeds 50. Furthermore, it has confirmed that the soil structure including the experiment yard is almost stratified horizontally.

Feature of foundation model group

The foundation model group is composed of five models as shown in Fig. 1. Table 1 shows physical constants of piles, and Table 2 displays profiles of each foundation model. Fig. 3 shows the cross section of each foundation model. All piles consist of steel pipes that have the same section and length, and all foundation blocks are made of reinforced concrete. Model I, Model II and Model III were constructed to investigate basic characteristics of dynamic interaction between soil and piles supporting structures. We have set about 0.5 m spaces between the bottom surfaces of these three foundation blocks and soil surface, to satisfy the condition that stress transmission from the blocks to soil is entirely done through piles. Increase of the spaces, about 0.25 m, between surfaces of foundation blocks and ground surface has been observed. This might be caused by erosion etc. Model I was constructed to investigate dynamic characteristics of a single pile, while Model II and Model III were installed to examine those of pile-groups. The former has four piles and the latter possess four or nine piles that were arranged with 1.0 m intervals, *i.e.* about 2.5 times as much as the diameter of the piles. Embedment depths of Model IV and Model V are about 0.6 m, and these block sizes are the same as Model II. Model V is a simple embedded raft foundation without piles, while Model IV is supported by four piles. Furthermore, it was recognized that their embedment depths had become partially lower. Pile arrangements of all pile foundations are in the square array. We employed Model III as the foundation on which the vibration generator was set for the forced vibration tests.

OUTLINES OF TEST20 (PAST FORCED VIBRATION TESTS)

Forced vibration tests in 1995 and 1996 were twice carried out by using the same model group as the experiment in 2001. Two types of vibration generator were employed in the past experiments. One was an eccentric mass type vibration generator whose eccentric moment is able to be consecutively changeable and another is an oil pressure servo type one. In order to compare with the results of the last year’s experiment, only these of the past experiments by the eccentric mass type vibration generator are chosen. Here, we name the past experiment that utilized the eccentric mass type vibration generator as TEST20.

The step steady state sinusoidal forced vibration whose frequency range is from 2.0 Hz to 25 Hz with frequency interval of about 0.1 Hz was performed in the past test. Table 3 shows predominant frequencies and equivalent viscous damping ratios obtained by curve fitting of the displacement resonance curves of each excitation level of three models [3]. Furthermore, Fig. 4 shows the displacement resonance curve of Model III. Fig. 4 displays the amplitude (absolute value) and phase angle per unit excitation force whose target exciting force P*=9.8 kN(1 tonf). The predominant frequency, about 8.6 Hz, is obviously found from not the amplitude function but the phase angle one. Moreover, the result shown in Table 3 obtained from the curve fitting is 8.64 Hz, and both results have a good agreement.

OUTLINES OF TEST21 (REPEATED FORCED VIBRATION TEST)

Purpose of the test

The forced vibration test in 2001 (here, it is named TEST21) was conducted to investigate following three items as a main purpose.

(a) Dynamic characteristics of Model III

The change of dynamic characteristics of Model III by differences of excitation force level and foundation mass is investigated qualitatively and quantitatively.

(b) Dynamic behaviors of adjacent foundation models when Model III is excited

Obtaining basic characteristics about dynamic cross interaction of pile supported foundation group, the measurement system that has sensors of total 300 components is employed.

(c) Secular variation concerning dynamic characteristic of pile foundation model

Secular variation of dynamic stiffness of soil around piles is investigated through the comparison with the past test results.

Prediction analysis (simulation analysis of TEST20)

A sweep excitation method that gradually increases exciting frequencies is adopted because of the restriction of the performance of the vibration generator and the experiment period in TEST21. The excitation force of TEST21 is

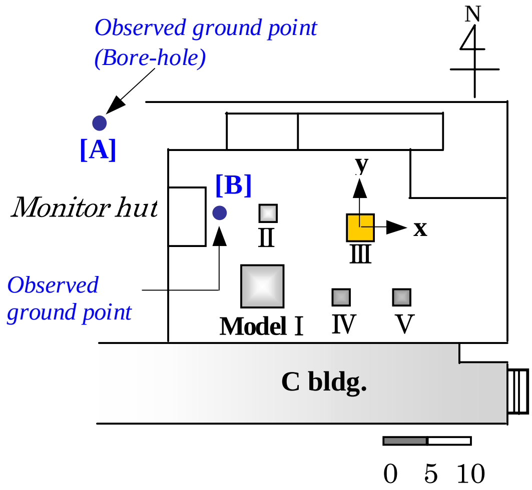


Fig. 1 Illustration of test field

Table 1 Pile profile

Variety	Steel
Length	26.6 m
Diameter (D)	406.4 mm
Thickness	9.5 mm
Section area	118.5 cm^2
Mass per unit length	88.2 kg / m
Moment of inertia	2.334×10^4 cm^4
Young's modulus	2.06×10^7 N / cm^2

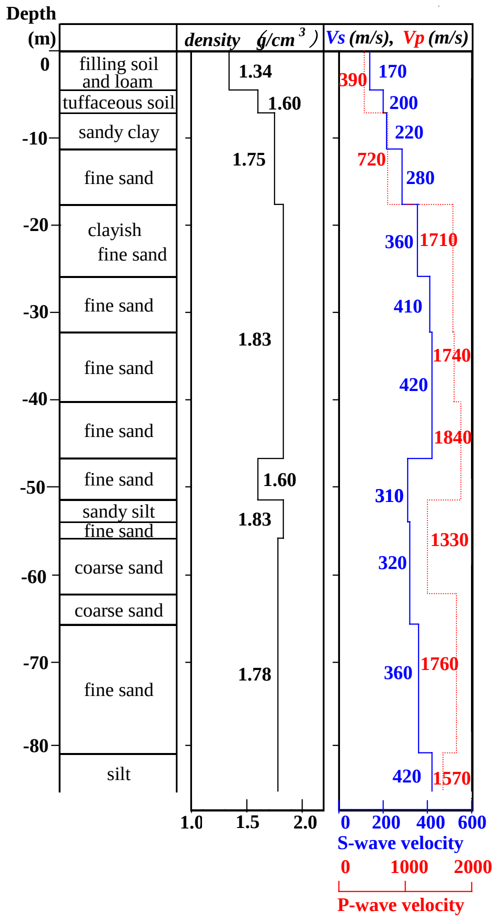


Fig. 2 Soil profile

Table 2 Model profile

MODEL	Dimension (m)	Mass (ton)	Pile number	Pile space	Remarks
Model I	5x5x1	60	4	10D	-
Model II	2x2x1.2	11.5	4	2.5D	-
Model III	3x3x1.4	30.2	9	2.5D	-
Model IV	2x2x1.2	11.5	4	2.5D	Embedded
Model V	2x2x1.2	11.5	None	-	Embedded raft

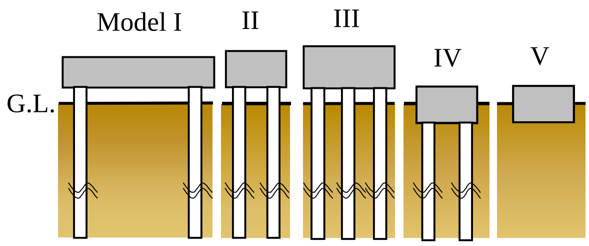


Fig. 3 Model sections

Table 3. Predominant frequencies and damping ratios of three models in TEST20

MODEL	Excitation dimension	Excitation force(kN)	Predominant freq.(Hz)	Damping ratio(%)
M.I	x (EW)	9.8	4.60	5.5
		19.6	4.34	7.7
		29.4	4.30	14.0
M.II	x (EW)	2.45	8.73	11.8
		4.9	8.59	9.9
		3.68	8.59	11.0
M.III	x (EW)	4.9	8.83	17.1
		9.8	8.64	16.8
		7.35	8.67	17.8

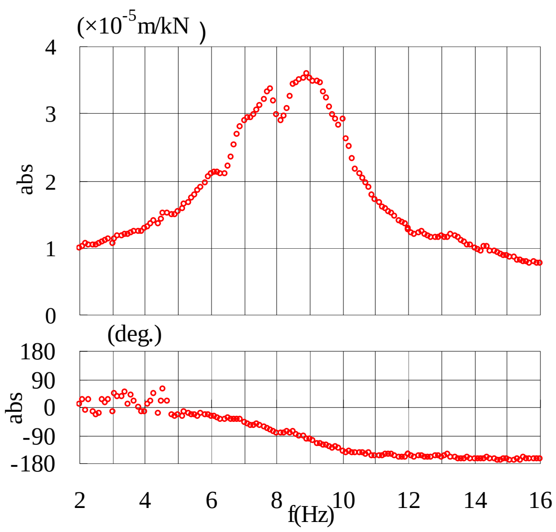
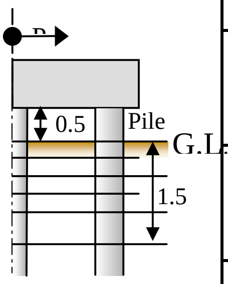


Fig. 4 Displacement resonance curves of Model III of TEST20

Table 4 Soil profile for numerical analysis

Depth (m)	Vs (m/sec)	Mass density (ton/m^3)	Poisson's ratio	Damping ratio
3.15	130	1.40	0.420	0.01
6.95	150	1.50	0.488	0.01
19.10	255	1.85	0.496	0.01
26.00	350	1.85	0.476	0.01
52.00	400	1.85	0.469	0.01
Semi-Infinite	400	1.85	0.469	0.01
	modeled by dashpot-mats			



	Thickness (m)	Vs (m/s)	Damping ratio
Case1	0.50	130	0.01
	0.25	130	0.01
	0.25	130	0.01
	0.50	130	0.01
Case2	0.50	80	0.03
	0.25	90	0.02
	0.25	100	0.02
	0.50	100	0.02
Case3	0.25	50	0.02
	0.25	50	0.01
	0.50	60	0.01
	0.50	90	0.01

Fig. 5 Prediction analysis cases

proportional to the quadratic of exciting frequency because the excitation is carried out with the constant eccentric moment in each sub-test. Therefore, displacement component is the most appropriate value to obtain accurately measurement data in a wide frequency range. Because lots of sensors that have already been installed are accelerometers, appropriate gain setting of each sensor is indispensable to obtain good data during measurement. A prediction analysis of response value of displacement, velocity and acceleration at each measurement point of the model subjected to sinusoidal excitations was carried out to obtain basic information for the gain setting.

We conducted the simulation analysis of TEST20 substituting prediction analysis of TEST21. It assumed that Model III was constructed alone at the test yard and the simulation analysis was performed by 3-dimensional thin layer approach [6]. The basic soil profile utilized for numerical analyses is shown in Table 4. The soil profile is based on the PS logging test results of Fig. 2 in consideration of the material nonlinearity of surface soil with about 3 m thickness. Then, it assumed that the layers lying under pile tip are a semi-infinite homogeneous elastic medium and they are modeled by a dashpot-mat. Furthermore, taking into account not only material but also geometrical nonlinearity, we refined the basic soil profile to three prediction analysis cases indicated in Fig. 5. In the numerical calculation, the mass of the vibration generator and the height of exciting point (about 0.3 m from the upper surface of the block) were assumed to be the same as TEST21.

The Horizontal displacement resonance curves at the center position on the block of Model III in x-direction (EW component) for three analysis cases are shown in Fig. 6. The result of TEST20 is also displayed in the figure. The prediction analysis result of Case 3 that took appreciably both decreasing in the rigidity of soil and increasing in the damping into account agrees well with the past test results. Therefore, it is recognized that the properties of soil surrounding pile foundation that is subjected to forced vibration are strongly affected by the material nonlinearity.

Test schedule

In TEST21, an eccentric mass type vibration generator (maximum excitation force performance: 29.4 kN(3 tonf)) was installed at the center position on the upper surface of the block of Model III. We conducted two cases

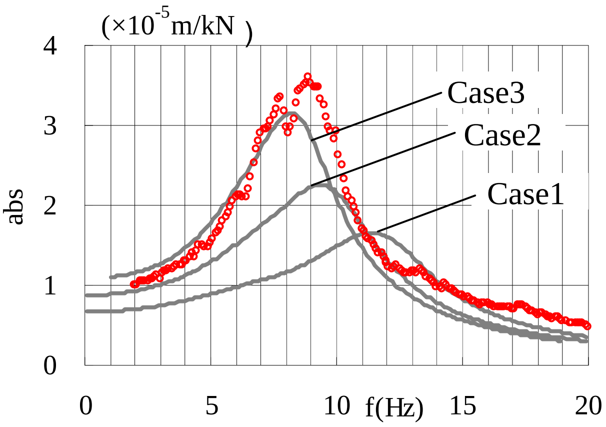


Fig. 6 Displacement resonance curves (U_T/P) by prediction analysis

Table 5 Test schedule

Test name	Exciting direction	Exciting force level	Added weight
NXS	x	small	-
NYS	y	small	-
Added weight installation			
EYS	y	small	○
EXS	x	small	○
EXM	x	medium	○
EXL	x	large	○
Added weight removal			
NXM	x	medium	-
NXL	x	large	-

Table 6 Sub-test for small excitation level

Sub-test No.	Mr(kg・m)	fs(Hz)	fe(Hz)	Pe(kN)
1	2	1.0	12.0	11.37
2	4	1.0	9.0	12.79
3	6	1.0	7.0	11.60
4	8	1.0	6.0	11.37
5	10	1.0	5.5	11.95

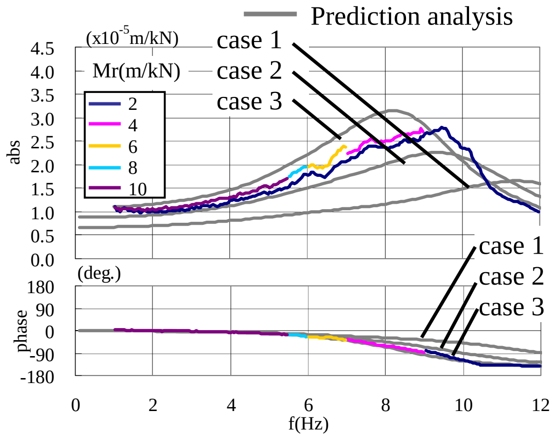


Fig. 7 Comparison of displacement resonance curves (U_T/P) between prediction analyses and TEST21(no-added weight)

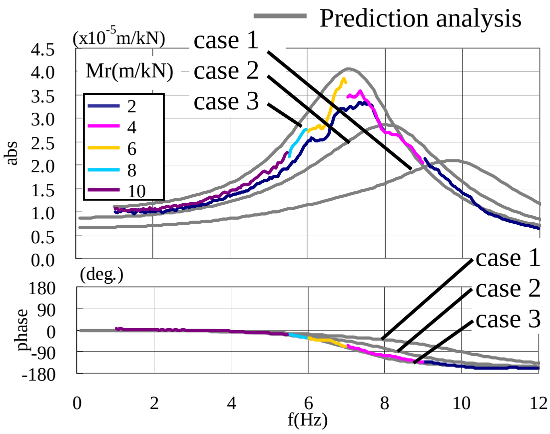


Fig. 8 Comparison of displacement resonance curves (U_T/P) between prediction analyses and TEST20(added weight)

of forced vibration tests to investigate the difference of dynamic characteristics of the pile foundation blocks caused by variation of the block mass. At the first test, the forced vibration test of Model III (total mass equals about 30 ton) without added mass was carried out. In another case, we excited Model III with the added lead mass (mass: 10 ton) whose total mass becomes 1.3 times. Moreover, we planed three excitation force levels as the excitation schedule to confirm presence of local nonlinearity that would be caused by strain dependency of soil rigidity and the separation between the pile surface and surrounding soil. Considering with the performance of the vibration generator, we decided the Small (target exciting force P*=9.8 kN), Medium (P*=19.6 kN) and Large (P*=29.4 kN) excitation levels indicated in Table 5. In TEST21, a sweep excitation method that gradually increases exciting frequencies was employed because of the restriction of the performance of vibration generator and the experiment period. The experiment on each excitation level shown in Table 5 was executed by dividing into some sub-tests due to the restriction of the data collection software. Table 6 shows one example of sub-tests for Small excitation level. In Table 6, fs and fe indicate the start and end frequency of sweep test, respectively. Monitoring frequency transfer functions that were computed in real time, we combined with each sub-test.

DYNAMIC CHARACTERISTICS OF MODEL III IN TEST21

Displacement resonance curve

The horizontal displacement resonance curves at the upper surface of Model III block in the exciting x-direction obtained from TEST21 and the prediction analysis of the three cases are shown in Figs. 7 and 8. In these figures, the resonance curve combined with several parts of each sub-test, whose exciting force was the closest to the target maximum exciting force (9.8 kN) of the Small excitation level (NXS), was drawn by five solid lines. We also illustrated all results in the case of Mr=2 kgm, which is the minimum eccentric moment of the applied vibration generator. We employed the Welch method with the Hanning window to calculate power and cross spectra for presuming frequency transfer functions from the measured data.

Fig. 7 shows the resonance curve with no-added mass in the case of Model III, and Fig. 8 displays that with added mass. Both figures indicate the resonance curve in terms of the absolute value and the phase angle (in degree) against frequencies. In those figures, some differences (gap) of the resonance curves at the boundary part of each eccentric moment are observed. It seems that the gaps are caused by evaluating transfer function at each sub-test and nonlinearity of soil.

At the experiment plan stage, it assumed that the nonlinearity of soil, e.g. strain dependency etc., hardly affect to dynamic subgrade reactions in Small exciting force level (NXS). It is recognized that the nonlinearity of soil might be occurred from the comparison of Mr=2 kgm exciting force level. The predominant frequency with no added mass of Model III, about 9.3 Hz can be estimated from phase. The numerical result of the prediction analysis Case 3 corresponds well with the test result. The resonance curve of the past test (target exciting force is 9.8 kN) is also shown in Fig. 9. Although the exciting force level of the test (NXS) is lower than that of the past test, the predominant frequency of TEST21 is a little higher than that of the past test. Therefore, it appears that the rigidity of soil surrounding piles has become higher than that in 1995 and 1996 due to secular variations.

Sway and rocking springs calculated from test results

The identified soil springs (sway spring K_H and rocking spring K_R) of two cases (no-added mass and with added mass cases) are calculated from the measurement data. The equation of the motion of a single foundation block subjected to sinusoidal horizontal exciting force $Pe^{i\omega t}$ can be written as,

$$\begin{Bmatrix} P \\ PH_P \end{Bmatrix} e^{i\omega t} = \left[-\omega^2 \begin{bmatrix} m & mH_G \\ mH_G & J_0 + mH_G{}^2 \end{bmatrix} + \begin{bmatrix} K_H & 0 \\ 0 & K_R \end{bmatrix} \right] \begin{Bmatrix} u_f \\ \theta_f \end{Bmatrix} e^{i\omega t} \tag{1}$$

where: ω : ciecular frequency of the exciting force,
P : amplitude of the exciting force,
H_p : distance of the exciting point from the bottom surface of the foundation block,
H_G : distance of the center of gravity of the block from its bottom surface,
u_f : displacement of the center of the bottom surface of the block in the exciting direction,
θ_f : angle of rotation about rotating axis of the block,
m : total mass of the block (including added-mass),
J_0 : moment inertia about rotating axis at the center of gravity of the block.

Then, using the known physical constants and the measured data, the sway and rocking spring can be derived as followings,

$$K_H = \frac{P + \omega^2 m u_G}{u_f} \tag{2}$$

$$K_R = \frac{PH_P + \omega^2 m u_G H_G + \omega^2 J_0 \theta_f}{\theta_f} \tag{3}$$

where, u_G :displacement of the center of gravity of the block in the exciting direction.

Figs. 10 and 11 show the identified results of sway spring, K_H, and rocking spring, K_R, of the two cases with the added mass and without the added mass, respectively. The figures display spring values of the minimum eccentric moment in the excitation case of Mr=2 kgm. Furthermore, the figures also show the springs calculated from the past test results. Little difference of two results of no-added mass case (NXS) and added mass case (EXS) is observed. Accordingly, it has been verified that a nonlinear influence degree in the case of the minimum eccentric moment Mr=2 kgm excitation is low. A remarkable difference of sway spring between both test results cannot be found, while it turned out that the real part (rigidity) of the rocking spring of the test is about 20 % or 30 % higher than that of the past test.

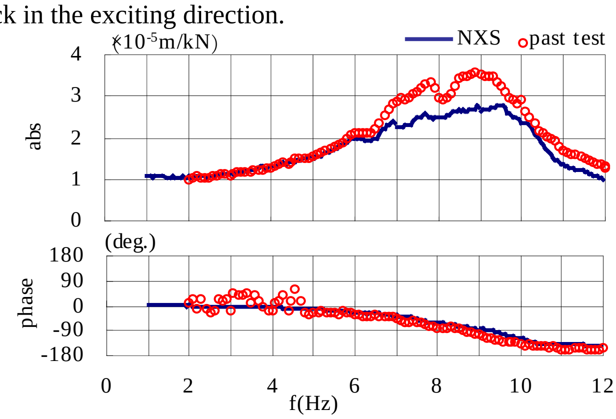


Fig. 9 Comparison of resonance curves (U_T/P) by past and TEST2001(no-added weight)

AGING EFFECTS ON DYNAMIC SOIL SPRINGS

Simulation analysis model considering subsidence of soil surface surrounding piles

According to the resonance curves and the impedance functions illustrated in Figs. 9, 10 and 11, it is recognized that the rigidity of soil around piles has been stiffer than that of TEST20. On the other hand, the soil subsidence surrounding piles under foundations, about 0.25 m, was found by a post measurement. Therefore we do a simulation analysis taking into account the secular variations of the ground condition, that is, the soil subsidence around piles and an increase of soil rigidity. As described above, we employ the three numerical models of Case A, B and C that have considered the subsidence of 0.25 m of the soil surface around piles and increase of soil rigidity of the upper part of 1.5 m depth. These three models are illustrated in Fig.12. Case A is a model that neglects completely the first soil layer with 0.25 m thicknesses. Case B is taking account of separation (no stress transmission) at the interface between the first soil layer and piles.

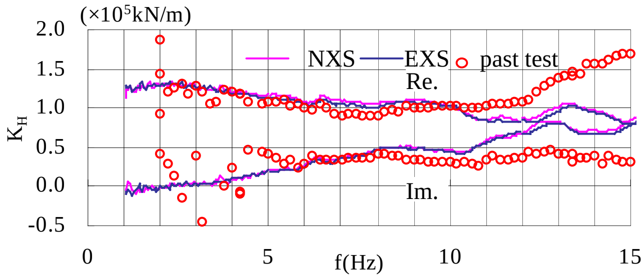


Fig. 10 Comparison of sway spring by TEST20 and TEST21

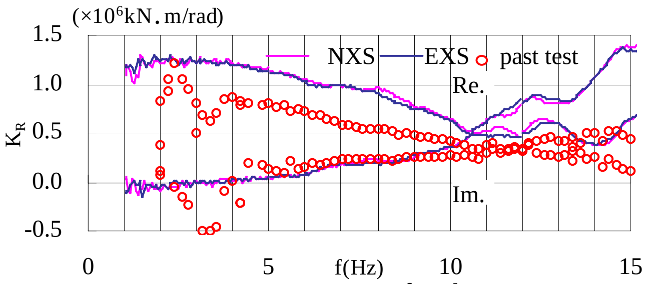


Fig. 11 Comparison of rocking spring by TEST20 and TEST21

Furthermore, Case C is adopted as a model for cross interaction analyses in the companion paper. This model has no separation, but is a compensated one in which the soil rigidity is lower and damping ratio is higher than Case B.

Displacement resonance curves

Fig.13 shows the resonance curves of the horizontal displacement on the upper surface of the block obtained from the simulation analysis and TEST21. The figure displays the absolute value and the phase angle (in degree) of the resonance curves. The result of Case B coincides with the test result better than that of Case A. According to the phase angle of the resonance curves, the predominant frequency of the numerical result for each model is almost 9.3 Hz and agrees well with the test result.

Impedance functions

Fig. 14 shows the sway and rocking impedance functions, K_H and K_R of the prediction analysis, *i.e.* Case3, comparing with the results of TEST20. Fig.15 illustrates the impedance functions of TEST21 that obtained by the simulation analyses Case A, B and C. Case 3 is not only the simulation analysis model against TEST20 but also applicable to TEST21. Fig. 14 indicates that a real part of rocking impedance of Case 3 is larger than that of TEST20. On the other hand, the simulation analysis results of all cases, *i.e.* Case A, B and C, show a good agreement with those of TEST21 below 9 Hz. In Fig. 15, the imaginary parts of sway impedance of Case B and C are corresponding especially well with that of TEST21. As results Case C is almost equivalent to Case B, and we employ Case C as the numerical model in the companion paper, Part 2. The real part of the rocking impedance of TEST20 was lower comparing to that of TEST21. It seems that the upper part of soil surrounding piles is still loose and the friction resistance (vertical subgrade reaction) at the interface between piles and soil does not work well due to the immediate performance of TEST20 after installation of the foundations.

CONCLUSIONS

In this paper, we first described the outline of the foundation models supported on mock-up piles and the vibration test that performed six years ago. Then, we introduced another forced vibration test utilizing the same models again. Investigating the secular variations of the dynamic characteristics of the pile foundation, we compared both test results of Model III. Through the comparison of both test results, it turned out that the dynamic stiffness of Model III has little difference. Increase in the rocking spring is more appreciable than that of the sway one. In the companion paper, we discuss the test results and the analytical results considering with the adjacent foundations in detail.

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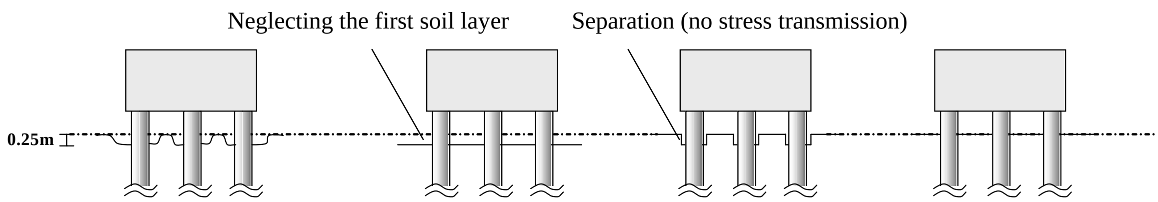

MODEL A

Thickness (m)	Vs (m/sec)	Damping ratio
1st layer removed		
0.25	70	0.02
0.25	70	0.01
0.25	90	0.01
0.5	110	0.01

MODEL B

Thickness (m)	Vs (m/sec)	Damping ratio
0.25	50	0.02
0.5	60	0.01
0.25	100	0.01
0.5	110	0.01

MODEL C

Thickness (m)	Vs (m/sec)	Damping ratio
0.3	50	0.03
0.3	80	0.03
0.4	80	0.03
0.5	100	0.02

Fig.12 Numerical analysis cases

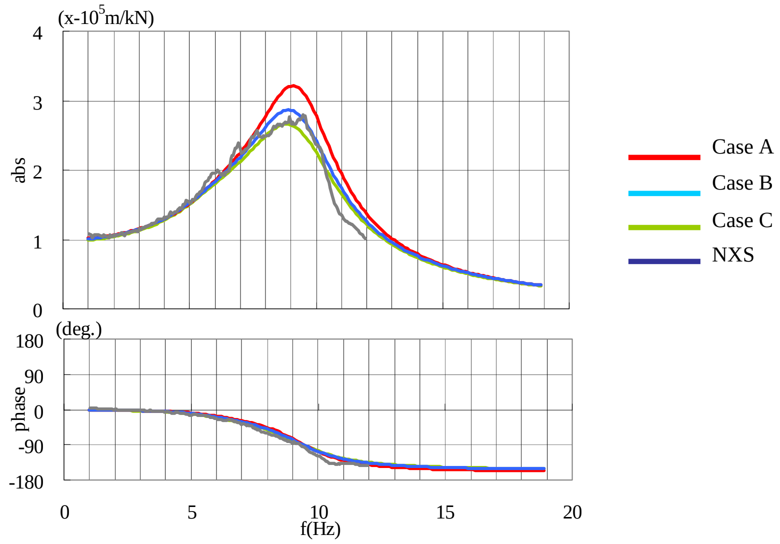

Fig.13 Comparison of displacement resonance curves (U_T/P) between numerical analysis and TEST21

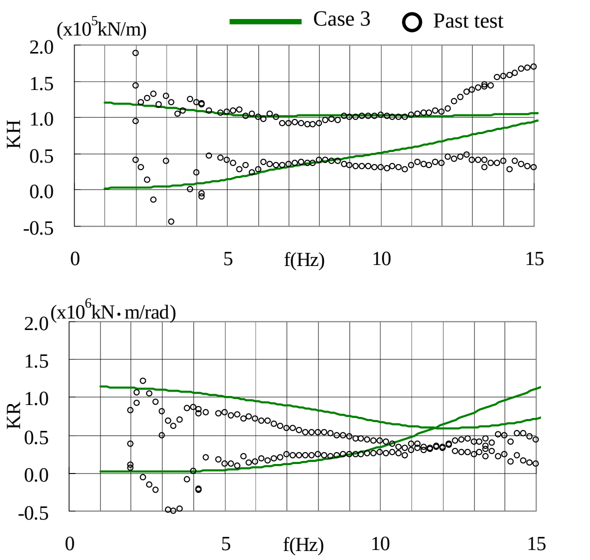

Fig.14 Comparison of sway and rocking springs between prediction analysis and TEST20

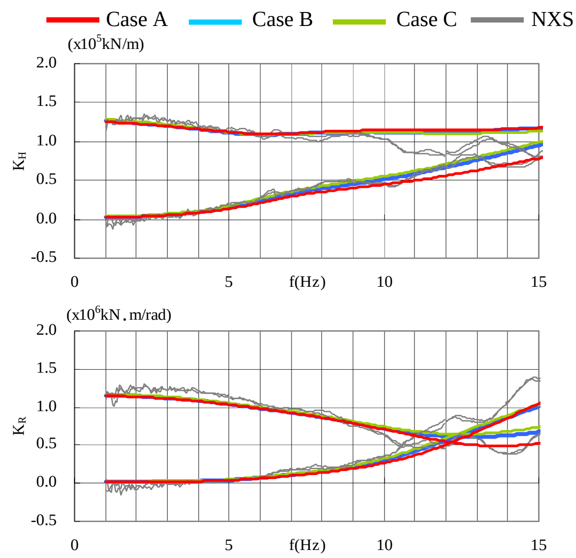

Fig.15 Comparison of sway and rocking springs between numerical analysis and TEST21