Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Vibration test of bridge pier with large-scale group-pile foundation and analysis

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ABSTRACT : The porpose of this study is to verify the adequacy of an FEM model taking into account soil-structure interaction applied to earthquake response analysis of a bridge pier with a large-scale group-pile foundation. Principally, this paper describes the results of dynamic FEM analyses compared with those of in-situ vibration tests using an exciter.
It was found from the experiments and analyses that ① coupling resonant phenomena between the ground and the pier are conspicuously observed at the first and second natural frequencies, ② dynamic axial strain which occurs at the tops of the piles is transmitted to the ends of the piles, and ③ the experimental results can be simulated by the two-dimensional analysis model, FLUSH.

1 INTRODUCTION

Seismic safety design of pile foundations is generally evaluated in Japan by means of the seismic intensity method. However, in cases of important structures, parameters obtained by the above mentioned method are generally verified by the earthquake response analysis method such as dynamic FEM method, etc.

Meanwhile, there is a scarcity of in-situ measuring data on interaction between a large-scale group pile foundation and ground, and the analytical method is far from being considered adequate from the point of view of verification, which in turn remains 2 a very important subject.

In this grasp, the authors conducted a forced vibration test on a real large size bridge and a simulation analysis by FEM method in order to study the dynamic behavior of the pile ground and ground interaction, as well as verifying the validity of the analytical model used for the earthquake response analysis.

The foundation of the pier which used for the test was constructed with group of piles comprised of 56 steel pipe piles that measures 1500 mm in diameter and 50 m in length. The footing and the pier stud are of composite structure made of steel plate and cast-in-place concrete. (The steel structure was prefabricated at yard and transported to the site.)

2 OUTLINE OF GROUND AND PIER FOUNDATION

The ground condition around the P11 pier of the access bridge for the Kansai International Airport, which is the experimental site in this study, is shown in Fig. 1, in which firstly soft alluvial clay is situated under the ground surface, secondly a sedimentary gravel bed, and finally a diluvial layer and thin sand layers form alternatie strata thereunder. The pile end is penetrated into the alluvial clay to about 55m deep. The pier is of portal bracing structure as shown in Fig.2 and the pier stud has a sectional area of 5 x 5m and a height of 34.4 m. The footing measures 32.5 x 27.5 m, comprised of 56 driven steel piles with ø1500 mm. The said steel-made pier is entirely manufactured in the factory, then transported and installed at the site. After installation, concrete is poured into the footing to form a composite structure. Concrete is poured up to a level of 15 m below the top of the pier.

3 EXPERIMENTAL METHOD

A vibration generator of 20 tons power was , installed on the top of the pier. Exciting moment was set to about 20 tons in a range of 2.8 Hz to 15 Hz. Frequency setting was in steps of 0.2 Hz on the average but the steps become more minutely divided near the resonance frequency. The excitation direction was in the longitudinal direction. As sensors, accelerometers, strain meters and reinforcing-bar stress transducers were put into use, as shown in Fig. 1 and Fig.2. Twenty five accelerometers were installed in total, of which 9 were at the pier stud (10 points), 6 at the footing (7 points), 6 in A5 pile (11 points), 1 in G5 pile (1 element), and 3 on the ground (6 points). The measuring point on the ground was chosen at 10 m, 20 m, and 30 m points (2 - 3 m in depth) in the exciting direction from the footing. Twelve strain meters (12 points) were installed on the wall of the pier stud, another 18 (18 points) in A4 pile, and more 18 (18 points) in G4 pile, and besides, 8 reinforcing-bar stress transducers were installed in the concrete of the pier.

Signals sent from each sensor were subjected to amplification by the amplifier and recorded on the digital data recorder together with the signal of the phase corresponding to the excitation force. A pen recorder and a spectrum analyzer were utilized as monitor for the purpose of observing the recording conditions and the amplified ratio. At the primary measuring points, amplitude was read off with the analyzer, and resonance curves were drawn up with the personal computer. Based on the above results, the resonance frequency was obtained, consequently determining the excitation step. The measuring results were subjected to the computer analysis to read off the number of vibrations, amplitude, and phase.

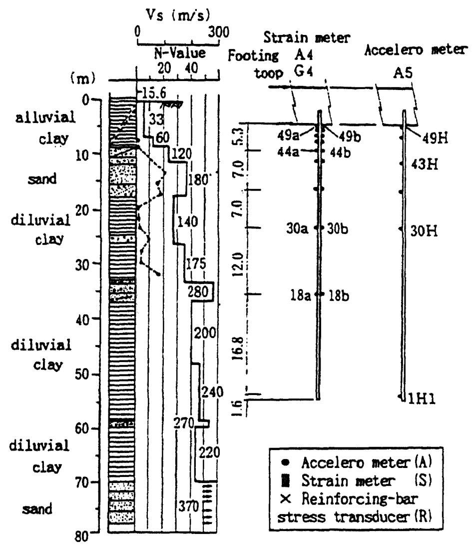


Fig.1 Soil-profile And Observation Point (Pile)

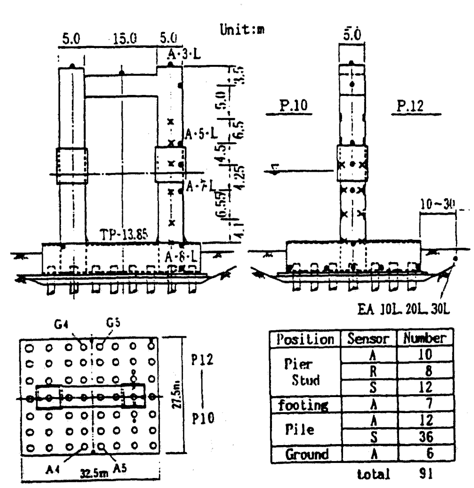


Fig.2 Pier And Observation Point

4 RESULTS OF EXPERIMENT

4.1 Resonance and phase curves

Resonance and phase curves at the representative measuring points of the pier stud, footing and the adjacent ground are illustrated in Fig. 3 to Fig. 6. Displacement was obtained by integrating the acceleration data two times. Bending strain is the mean value averaging the strain value measured at the opposite side of the same depth. Resonance curves were obtained by standardizing the above value per excitation force of 1 ton. It is found from the figure that the primary resonance points are at three points, 1.4 Hz, 3.2 Hz, and 11.7 Hz. The amplitude of the pier stud shows two clear peaks in Fig. 3, at which the phase indicates 90°, whereas the amplitude of the footing and the pile is shown in Fig.4, in which another peak comes at around 1.4 Hz in addition to the above two peaks of the pier stud. The amplitude of the footing is as small as 1/10 of that of the pier top. The peak at 1.4 Hz is also noticed in the bending stress of the adjacent ground and piles, particularly it is conspicuous at EA30L (at the point of the ground 30 m away from the footing).

According to these resonance and phase curves, damping constant will be calculated to 20 %, using the resonance-time amplitude both in the case of the pier stud and pile at the second peak of 3.2 Hz and 2 - 3 % at the third peak. At the first peak of 1.4 Hz it is too big to read off.

4.2 Vibration mode

Vibration modes at the resonance point are shown in Fig. 7. These figures depict distribution of displacement when the displacement at the top of the pier attains the max. The mode at 1.4 Hz is likely to be affected by the coupled movement of the horizontal vibration of the ground and the sway vibration of the footing, and the mode at 3.2 Hz by the primary bending vibration of the pier stud and the rocking vibration of footing. And the case of 11.7 Hz is likely to be attributable to the fact that the secondary bending of the pier stud is predominant. Damping constant, as mentioned above, becomes smaller at 1st, 2nd, and 3rd peaks in this order, and it is found from the characteristics of the vibration mode that damping increases as the vibration energy ratio of the adjacent ground rises.

4.3 Axial strain and bending strain of pile

Axial strain and bending strain of the pile at three peak are illustrated in Fig. 8, in which bold line data (-) was compiled from the strain meter, and the dotted line (---) by the difference between the vertical and horizontal displacements after being subjected to integration of the acceleration values. It seems from these figures that the axial strain and the bending strain show almost the same tendency as that by the strain meter, and the

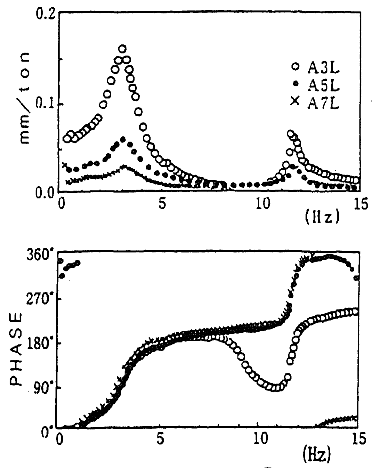


Fig.3 Resonanse Curves of the Pier stud

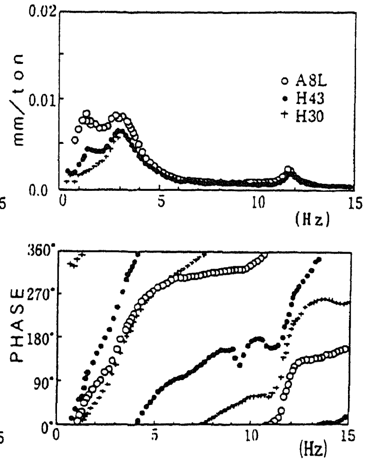


Fig.4 Resonanse Curves of the Footing & Pile

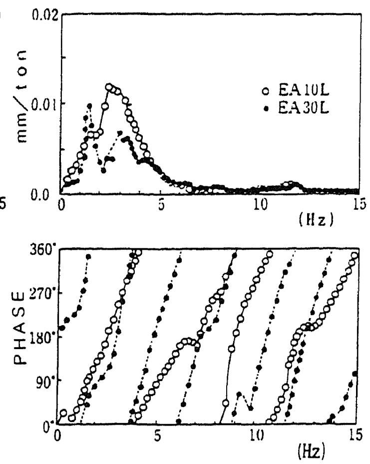


Fig.5 Resonanse Curves of the Ground

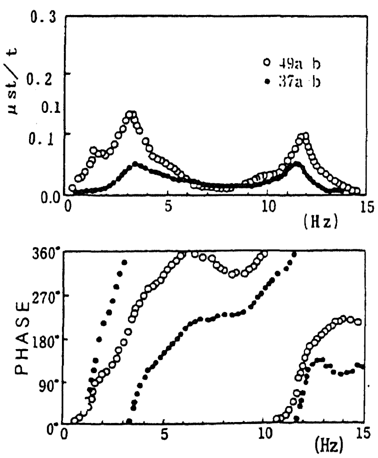


Fig.6 Resonanse Curves of the Pile (Bending strain)

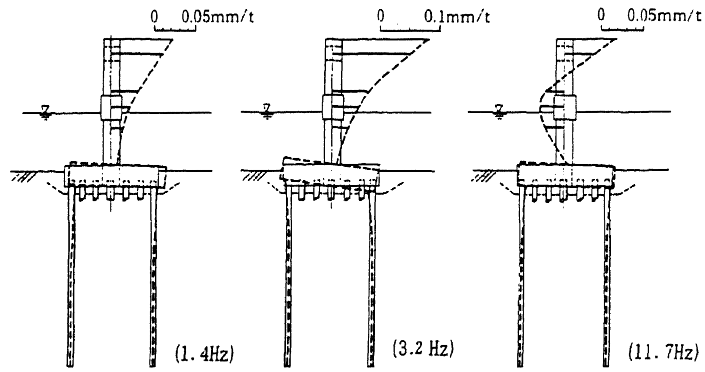


Fig.7 Mode Shape

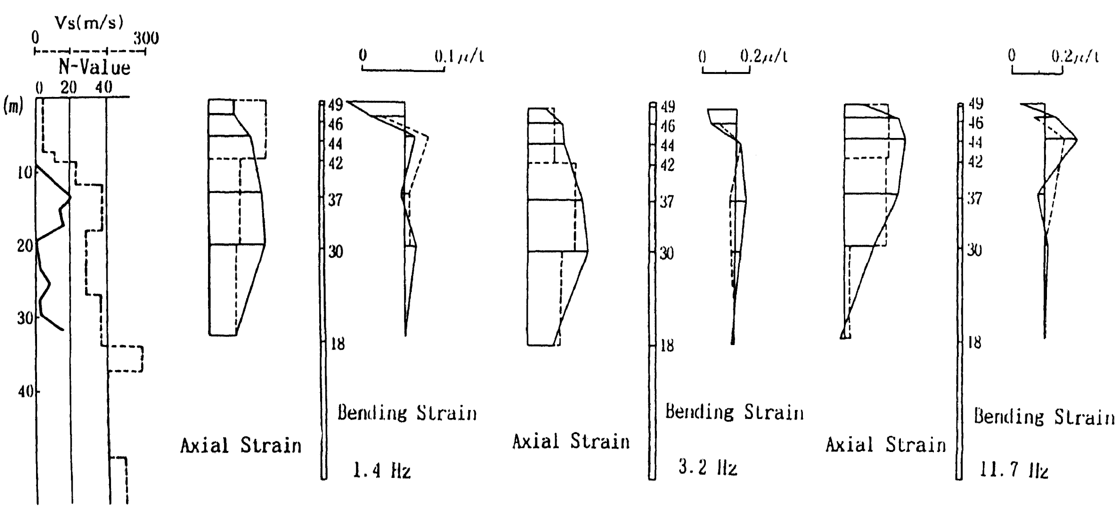
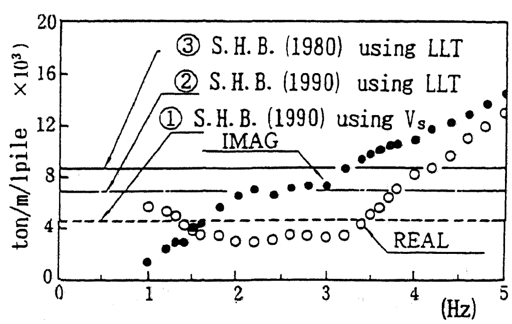


Fig.8 Strain of Pile

bending strain attains the max. at the pile top, at which the pile top is regarded as almost fixed. On the other hand, the axial strain does not come to the peak at the pile top but at the middle point near measuring point 30. *This is attributed to the fact that since the area up to* 2 m depth from the pile top was filled with concrete and the pile wall thickness up to 13 m depth from the pile top is 19 mm but decreases to 16 mm at the deeper level, the area located nearer the pile top is believed to have greater section rigidity, thus hardly causing strain. It is also because the layer at a depth of 20 m to 30 m is a softer layer (Vs = 140 m/s) than that located above or beneath, therefore the ground itself is easily subject to strain.

4.4 Horizontal spring constant of the pile head

First, the external force which affects on the pile top is calculated from the acceleration amplitude resulted on the experiment, the mass of the pier and the excitation force, and then divided by the vertical displacement on the pile top to obtain vertical spring constant, the results of which are illustrated in Fig. 9. It is shown from the said figure that the spring constant varies in a complicated pattern depending on frequency, however, the real part tends to decrease to the minimum value at around 2 - 3 Hz, whereas the imaginary part tends to increase in a range larger than 3 Hz as the frequency increases. Since the resonance point where the ground and the footing vibrations are coupled is clearly observed at around 3Hz on the resonance and phase curves, the aforesaid variation by frequency is, therefore, seemed to have been affected by the effect of the above resonance point. The spring constant obtained from the experiment is almost coincident to the value listed in the specifications of the notes in the figure as far as it is concerned with the mean value of 1 to 5 Hz. The reason why it was coincident despite the fact that the pile displacement value, on which the design manual stands, is as large as 100 times of that occurred in the vibration test is explained as follows. That is, the design manual value does not take into consideration the effect of the pile group whereas the vibration test



S.H.B.: Specifications of Highway Bridges in Japan

Fig. 9 Spring Constant

result was subjected to the above effect. Therefore, the experimental result happened to be coincident with that of the design manual for the reason that the increase effect on the spring due to the lower displacement of *pile was offset with the decrease effect due to the pile* group effect.

5 ANALYTICAL METHOD

Two dimensional FEM of the advanced FLUSH version covering the viscous bottom boundary was applied for the simulation analysis, Fig. 10 shows the analytical model (1/2 model), which can represent 3-dimensional effect using viscous boundary attached to each nodes taking the thickness of the structure into account transmitting elements were used as side boundary. The depth is designed to be the same as of the footing width of 32.5 m. The pier stud was assumed to be a board, in modelling, having the same depth, and 8 piles to be one board and modelled into 7 boards as beam element. The bottom of the pier stud and the footing are of compound structure made of steel and concrete, and the pile inside is filled with concrete up to the level of 2 m under the footing. Therefore, these concrete parts were evaluated as all sectional area being effective. Parameters used with reqard to the ground are shear modulus and density based on the elastic wave test. The effect of sea water was taken into account as added mass to the pier stud submerged in the sea.

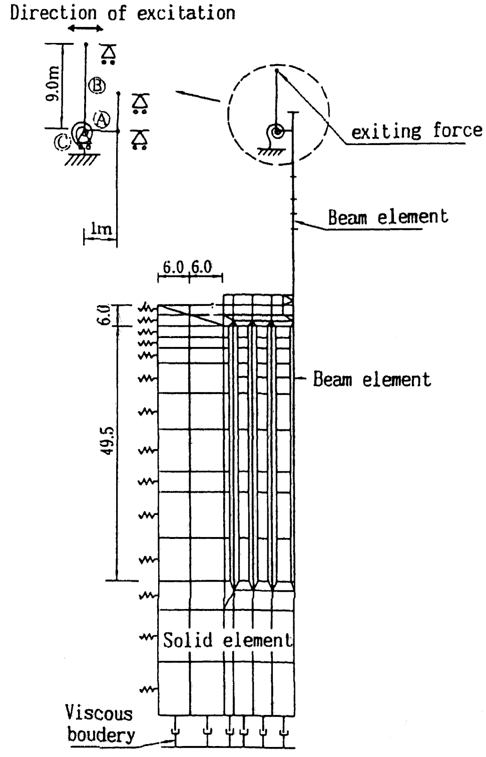


Fig. 10 F E M Model

Damping of structure on the pier stud, footing, pile, etc. was set to 2% and all layers of the ground equally to 5%. In this analysis, non-linear effect was not evaluated because the analysis was limited in the very small strain domain. The pier stud was designed to be of such a structure that the top of two pier studs was connected with the cross beam, and a vibration generator is installed at the center of the cross beam, which generates excitation force in the axial direction of the pier. Therefore, the cross beam section was made into a model as shown in the attached drawing of Fig. 10, of which it was placed vertically in order to simulate a three-dimensional structure with the beam of two dimension structure having rotational spring and rotational inertia at the bottom. The top of the beam was restricted to rotate but free to move vertically. The length is as long as the center point of the cross beam (position of the vibration generator). In the figure, A is assumed to be a rigid beam element which only transmits the horizontal axial force, B to be a cantilever with the flexural rigidity of as much as two times that of the cross beam whose the top is restricted to rotate, and C to be a rotation spring provided with the rotation inertia of the pier stud at the center, which simulates the torsional rigidity of the pier stud. The excitation point is at the top of beam B, in which sine wave proportional to the output of the vibration generator was entered to analyze the response.

6 ANALYTICAL RESULTS

6.1 Resonance and phase curves

Fig. 11 shows the comparison with the experimental results measured at the representative points. It seems from these figures that the resonance curves at each measuring point of the pier stud, footing, pile, etc. are very much coincident with each other respectively, which implies that the model simulated the vibration characteristics very well. However, it is noted that the peak appearing at around 12 Hz is slightly different in the case of the pier stud. This is attributable to the fact that since at this peak, torsion and flexural vibration occur at the pier stud, modelling has a limit as far as two-dimension structure is used even if the rotation spring and beam element are set vertically as illustrated in Fig. 10. The smaller amplitude of the peak at the pile section than the experimental value is regarded as ascribed to the reason that the element division at the bottom of the ground was too large and high frequency could not be simulated.

6.2 Vibration mode

Vibration modes of each resonant frequency is charted in Fig. 12. The respective mode shows at the moment when the horizontal displacement of the top of the pier

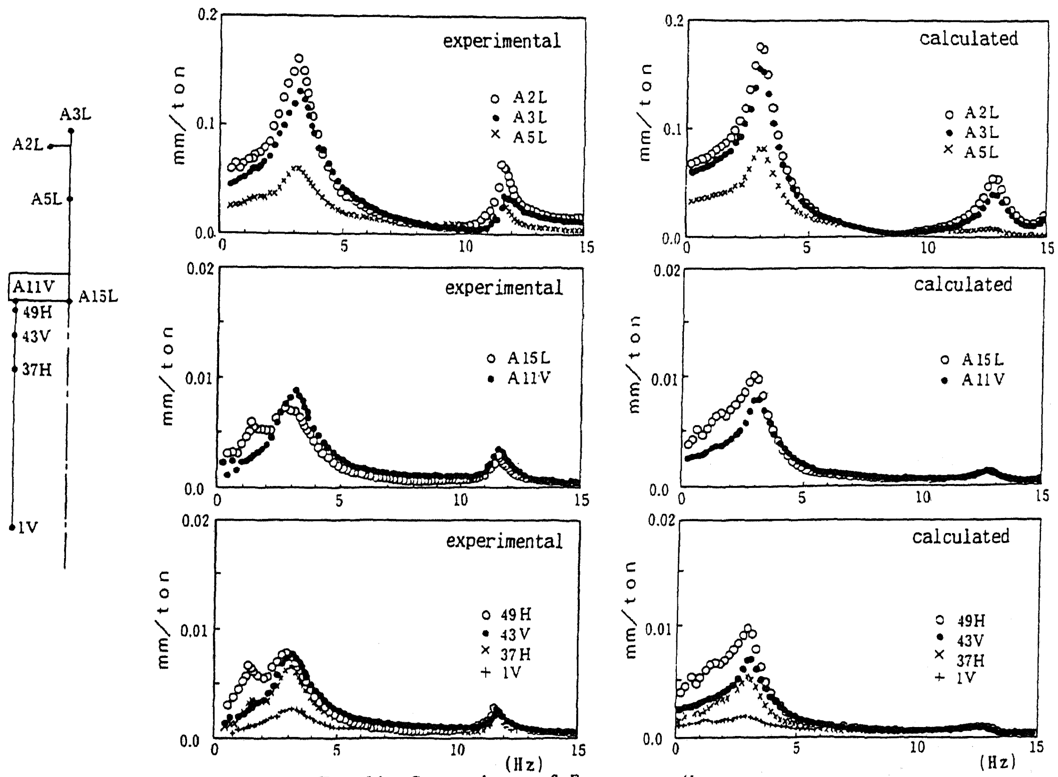


Fig. 11 Comparison of Resonance Curves

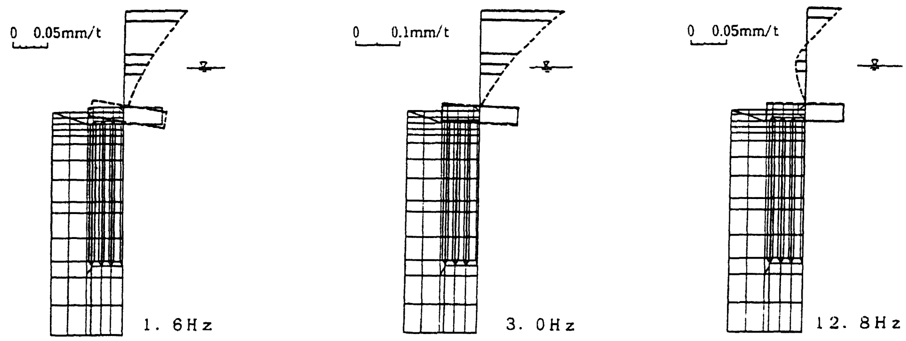


Fig. 12 Model Shape (Calculated)

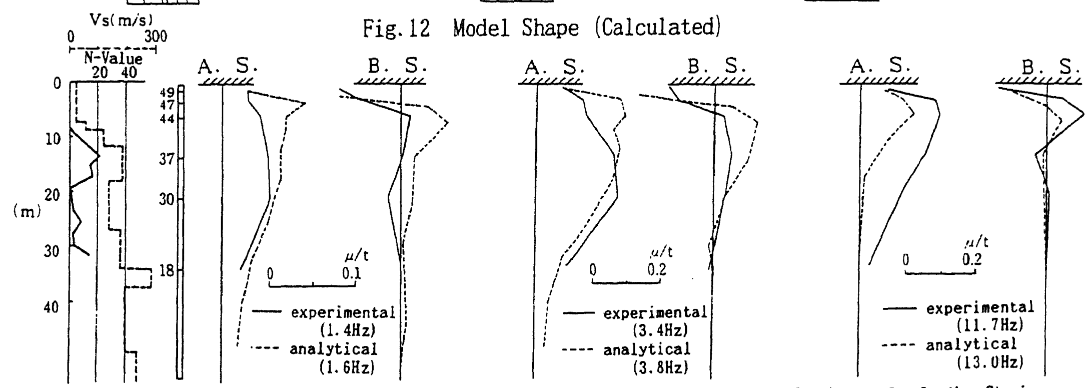


Fig. 13 Comparison of Pile Strains

A.S.: Axial Strain B.S.: Bending Strain

stud becomes the max. It is found from the figure that the mode at each resonance frequency is very much coincident with that of the experimental result.

6.3 Axial and bending strain of the pile

Fig. 13 illustrates the distribution of the axial and bending strain that occurred in the pile for each measurement point. It is apparent from these figures that the analytical results of the strain distribution of the pile are almost coincident with the experimental results. However, the analytical results of the axial and bending strain at the point 5 to 6 m from the pile top (measuring point 47, 44) is about 1.7 times larger than those of the experiment. This is likely to be explained by the reason that as concrete was poured into the pile deeper than expected, making it larger than the rigidity of the pile in the analytical model, and in the analysis the pile is modelled into a board but in reality 8 piles are lined in parallel.

7 CONCLUSIONS

It can be concluded as follows according to the results of experiment and analysis.

1. In the comparison of the resonance mode, the peaks which have a larger interaction with ground are the first and second resonant frequencies. In the first resonant frequencies the horizontal vibration of the ground and the vertical vibration of the footing were coupled, at the second resonant frequencies the primary bending vibration of the pier stud and the rocking vibration of the footing. Also, the third resonant frequencies appeared and this is due to the secondary bending vibration of the pier stud (Fig. 7). It is found from the relationship between the mode and damping that damping becomes larger as the vibration energy ratio of the adjacent ground in the mode increases.
2. Dynamic axial strain subjected to the pile might be transmitted to the deep part. The bending strain indicates the maximum value. at the pile top, besides the pile top seems to be almost constrained (Fig. 8).
3. The imaginary part of the spring constant of the pile top obtained from the experiment is tends to increase along with the increase of the frequency over around the 2nd resonant frequency and the real part shows the minimum value around this frequency. The spring constant in a range of 1 to 4 Hz is seemingly coincident with that obtained from the design manual (Fig. 9). This is likely to be attributed to the fact that since the group effect was effective despite small ground strain, these effects were offset.
4. The numesical using the two-dimensional FEM model, provided with appropriate treatment on the bottom and side boundary very successfully simulated the experimental results without adding any special adjustment to the analytical parameters.

References

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