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# Dynamic characteristics of steel pipe piled well foundations

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ABSTRACT:This study aims at clarifing the dynamic characteristics of the steel pipe piled well foundations. A model test was carried out to confirm the basic characteristics, the modeling method of a foundation and a soil by FEM. The interaction between the foundation and soil under the earthquake excitation and the amount of inertial force of the constrained soil within the well foundation were the main enlightened aspects. The necessity to include interaction as part of the mathematical model is verified from the experimental model. Finally, a comparison of static loading test result of two prototype foundations with those of the FEM calculations was carried out.

## 1. INTRODUCTION

A steel pipe piled well foundation has an intermediate characteristics between a caisson and a pile group foundation. This foundation type is a useful technique for underwater and soft ground foundations, because of its application to the cofferdam. The static mechanical properties have been studied in the past, with few studies about its dynamic characteristics. The aseismic design standard as yet have not been established.

This study aims to clarify the dynamic characteristics of the steel pipe piled well foundations. A small model test was carried out to confirm the basic characteristics and the modeling method of a foundation and soil by FEM. It became clear through test results and the calculations, that a steel pipe piled well foundation vibrated at its first natural frecuncy of the ground, then the soil pressure acting inside the foundation is almost $30 \sim 50\%$ of total inside soil inertial force. It is also verified that the numerical analysis must use the soil-foundataion interaction model. The results of FEM calculation for the prototype foundation can be compare with the model test results.

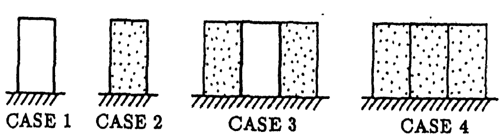



Fig.1 Test cases

## 2. MODEL TEST

Two kinds of model tests were carried out. The first test is to clarify the vibration characteristics of the model foundation and splice fixity. The other test is observation and analysis of the response characteristics of soil-foundation system. Fig.1 shows all test cases.

A view of the experimental model is shown in Fig. 2. Foundation is made by 16 poly vinyl chloride pipes (outside diameter 60mm, thickness 2mm), and splices were pipes(outside diameter 20mm) and silicon resin adhesive. Inside of sand box is filled with sand (average diameter 1.0mm), a model foundation is fixed to the bottom of the box. The experiment was performed on shaking table, TEST A is vibration by shaking table excitation and TEST B is oscillation at the top of the model by a small oscillator. Accel-

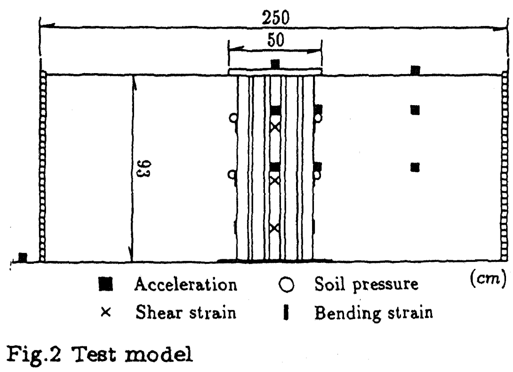



Fig.2 Test model

erations, soil pressures and strains of pipes were measured at several points as shown in Fig.2.

## 3. RESULTS OF MODEL TEST

Table 1 shows the natural frequency of foundation model. In CASE2, the inside of the foundation is filled with sand, the natural frequency increased in spite of mass increase, indicating large splice fixity. The resonance curves of CASE4 were shown in Fig.3 and Fig.4. Fig.3 is the results of TEST A and Fig.4 is TEST B. In Fig.3, a resonance curve of the soil and the foundation are shown to have the same tendency, then the founation was vibrated by the soil at its first natural frequency. However, vibration at the top of the foundation shows different tendency between the three curves in Fig.4. At the top of the foundation, there are peaks at $33Hz$ and $40Hz$. Here, $33Hz$ is the natural frequency of shaking table-

Table 1 Natural frequency of foundation model

| | CASE1-A | CASE2-A |
|---|---|---|
| Input | 70gal | 50gal |
| Eigen Frequency | 10.2Hz | 13.3Hz |
| Respons Acc. | 326gal | 339gal |

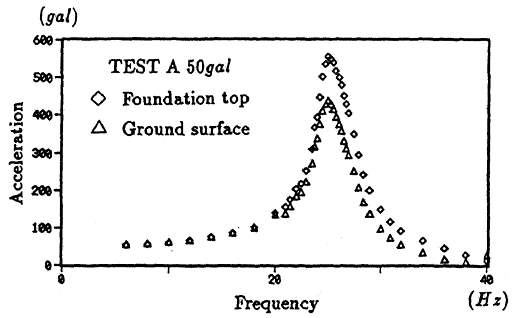



Fig.3 Resonance curve of CASE 4-A

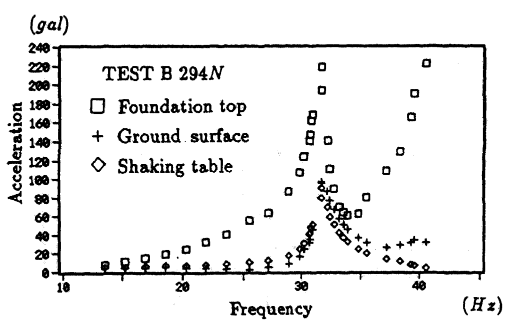



Fig.4 Resonance curve of CASE 4-B

actuator system. From these results, the natural frequency of model foundation will be over $40Hz$, this frequency could not be found because of osscilater's capacity. Therefore, the dynamic characteristics of foundation in soil are different when vibrated at the top of the pile, from the one vibrated at the bottom of the foundation. The dynamic analyses was then carried by the soil-foundation finite element model, a spring-mass model seems inadequate.

To consider the effect of the soil inside the foundation, Fig.5 shows a distribution of accelerations and soil pressures alternatively. In this figure, the maximum acceleration at the surface of the ground is normalized for $250gal$. The response accelerations inside of the foundation are rather smaller than at the surface. It can be assumed that foundation and soil vibrate at the same mode. Sand pressures indicates same tendency as accelerations, near the surface the soil pressures increase, outside pressures of CASE 4-A can be assumed as the sum of outside pressures of CASE 3-A and inside pressures of CASE 4-A. From this behavior, it can be said that pressure of inside soil is due to the inertial force obtained from the response acceleration of the inside soil. This value is 30~50% of the total inside soil inertial force. Then it can be concluded that all inside soil does not contribute to the total inertial force, because of the phase difference between the soil pressure and the inside soil response.

## 4. ANALYSIS OF MODEL

First, the foundation was modeled as two types shown in Fig.6. The natural frequency of these models is $3.4Hz$ and $7.0Hz$, respectivly. This, when compered with the result of the test at $7.25Hz$and splice fixity $\mu = 0$, MODEL B was used in calculation. The analysis of model was carried out by FLUSH program. The foundation was modeled by beam elements and soil as solid elements, three dimensional effect was considered in the calculations. A splice fixity $\mu$ and damping coefficients of foundation $h_F$ and soil $h_S$ were

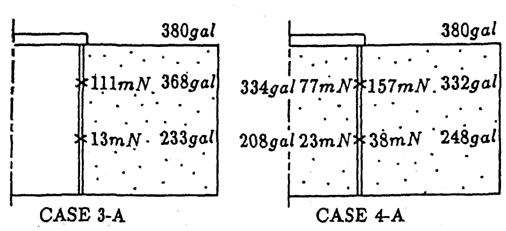



Fig.5 Distribution of the accelerations and soil pressures

observed by experiment, for CASE2 $\mu = 0.26$, $h_F = 0.104$ and $h_S = 0.066$ were used. The peak frequency and the response acceleration at the top of the foundation are shown in Table.2. Here, input wave is white noisee, its maximum value is 50 gal. The initial, a elastic wave velocity of sand $V_s$ was assumed at $90m/s$, that is observed by percussion test, then linear calculation was carried out. In the case, natural frequency was not equal to the test results. When $V_s = 100m/s$ these are equal, but at the experimental natural frequency response accelerations are larger than the test results. Then, nonlinear analysis using the Seed's strain - stiffness, damping curve at the peak frequency at $V_s = 116m/s$ was carried out, the response acceleration are then equal to the test results. The observed elastic wave velocity would be the velocity at the surface, so velocity at the bottom layer will be higher. Therefore, the average value is estimated as $V_s = 110{\sim}120m/s$. From Table 2, it is clear that a peak frequency of foundation equals that of soil, when the soil vibrates at the principal mode causing also the foundation to vibrate.

In this study, the law of similarity was ignored. An elastic modulus of poly vinyl chloride pipe is 1/80 that of steel and splice fixity is small, then to accomodate this negligence, the stiffness of foundation was changed as 100 times and 1000 times. The results of these calculations, a peak frequency of foundation was equal to soil. Therefore, a steel piped well foundation under the earthquake will oscillate at the soil's natural frequency. The result of model test is confirmed by the analysis.

## 5. ANALYSIS OF PROTOTYPE FOUNDATIONS

Two actual steel pipe piled well foundations shown in Fig.7 are considered for a comparison. These were constructed on soft ground. All piles of TYPE II are of the same length, however TYPE I has short and long piles alternatively. In the site of TYPE I, there is a thin bearing layer having $N > 50$ and the long piles extend below this layer. A static loading test was carried out for TYPE II foundation. These foundations were analyzed by the FLUSH program and modeled

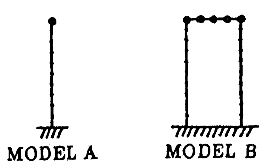


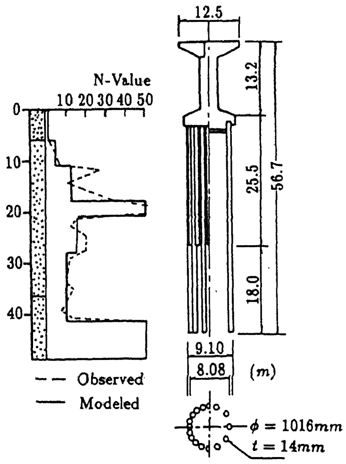

Fig.6 Foundation models

as an equivalent three dimensional model. In earthquake response analysis, the pile splice fixity was assumed as 0.5, and modified Tsugal Ohashi earthquake record of the Specifications for Highway Bridge of Japan for soft ground was used as excitation.

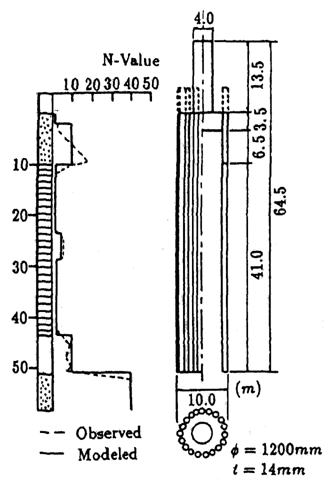
A frequency response functions at the top of the pier and at the groud surface of TYPE I

Table 2.

| Elastic Wave Velocity Vs (m/s) | Analysis | Input (gal) | Peak Frequency (Hz) CASE3-A Model | Ground | CASE4-A Model | Ground |
|---|---|---|---|---|---|---|
| 90 | Linear | 50 | 22.5 | 22.5 | 22.5 | 22.5 |
| 100 | Linear | 50 | 24.9 | 24.9 | 24.9 | 24.9 |
| 100 | Nonlinear | 50 | 21.5 | 21.0 | 21.5 | 21.5 |
| 116 | Nonlinear | 50 | 24.9 | 24.9 | 24.9 | 24.9 |




(a) TYPE I




(b) TYPE II

Fig.7 Prototype foundations

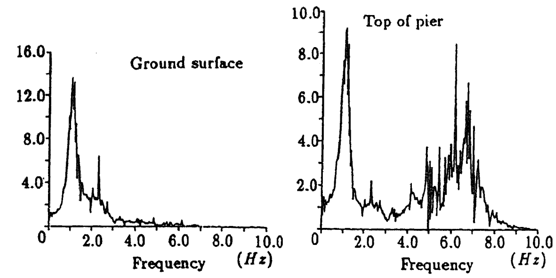



Fig.8 Frequency response curves of TYPE I

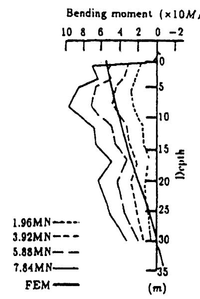



Fig.11 Bending moment

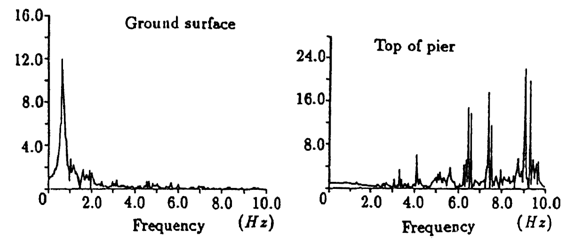
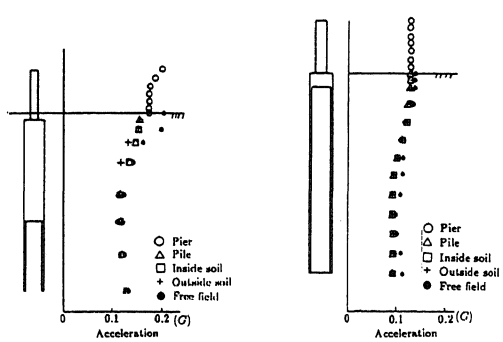



Fig.9 Frequency response curves of TYPE II




(a) TYPE I (b) TYPE II

Fig.10 Distribution of response acceleration

are shown in Fig.8(a),(b). These are calculated from the fourier spectrum at each point divided by the input wave to neglect the influence of the frequency characteristics of the input. These figures indicate that the top of the pier vibrated at the first natural frequency of the ground. Then, the natural frequency of pier will be estimated as $7Hz$ from $(a)/(b)$. Fig.9 is a frequency response function of TYPE II having a similar tendency to TYPE I. These results indicate taht the earthquake response must be analysed by the soil-foundation system or the ground response displacement should be calculated when it acts on the foundation, i.e. the lumped-mass-spring model can not simulate a real response.

Fig.10(a), (b) shows the response acceleration of a steel pipe, a soil in the area of a pile and a soil which was out of a foundation. This figure shows that these accelerations are almost of the same level, so foundation and soil vibrated uniformly. This tendency is confirmed for both foundations. The comparison with the free field acceleration, the acceleration decrease at the shallow part. In Fig.10(b), accelerations under the ground are almost the same. It is because it could be assumed as a mono profile ground.

From the earthquake response analysis, the maximum displacement at the top of the foundation was 63.2mm and the maximum bending moment was $313.6MNm$. Therefore the horizontal displacement at the top of the foundation observed by a horizontal static loading test is 10.95mm with the load level at $7,84MN$. This value is 0.17 times that of the response analysis. Fig.11 shows a comparison of the response bending moment times 0.17 with the results of the static test. It indicates a similar trend between these values.

## 6. CONCLUSIONS

This study clarified the basic dynamics and the characteristics of earthquake response of the steel pipe piled well foundation. The foundation is vibrated at the first natural frequency of the ground. Therfore, the earthquake response analys is must use a foundation-soil interaction model, and a soil pressure acting inside the foundation is almost $30 \sim 50\%$ of total inside soil inertial force.

This study is being jointly conducted with Association of the Japan Steel Pipe Pile.

## REFERENCES

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