Lateral Resistance of Pipe-Pile Well Foundation

Résistance Latérale des Fondations en Tubage

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SYNOPSIS There now exists an established method of designing a well foundation composed of steel pipe piles. However, this type of foundation has a wide range of dimensions and structures and the existing method is not good enough to cover the wide range of foundations. In this paper, the mechanical behavior of the well foundation under static lateral load is studied by three-dimensional network analysis, and characteristics which have not been clarified are discussed. A comparison of the experimental results with those of the theoretical analysis is performed. In addition, a double wall type well foundation composed of interlocked pipe piles was used as the foundation for a blast furnace. Since three-dimensional network analysis was used to design this well foundation, this paper also compares earthquake observation results with those of the theoretical approaches.

INTRODUCTION

The interlocked pipe piling method is a new foundation construction method by which steel pipe piles with interlocking junctions are consecutively driven into the grounds, forming a round, rectangular or oval shape to offer rigidity comparable to that of caisson foundations. This method permits the construction of foundations as fast as pile foundations. Because of these features, the method is ideal for constructing foundations for bridge piers, underground structures and offshore structures.

Recently, with structures becoming larger, foundations of this kind have increased in size, some of them with diameters exceeding the penetration length of the piles. Thus, studies of the structural characteristics of this foundation have just been initiated with a view to establishing better design standards. Small-scale and full-scale experiments (M. Ishiwata et al, 1973), (H. Nagaoka et al, 1976) have been carried out to clarify the mechanical behavior of the well foundation, but problems still remain unsolved.

In this report, the lateral resistance of interlocked pipe piles is analyzed by three-dimensional network analysis. In order to investigate the lateral resistance of interlocked pipe piles, model experiments were carried out using three models.

Moreover, a double wall type well foundation composed of interlocked pipe piles was introduced and used as the foundation for the blast furnace. Three-dimensional network analysis was used to design this well foundation. The results of earthquake observations are discussed in comparison with the results of the theoretical approaches.

ANALYTICAL MODEL AND METHOD OF ANALYSIS

The analytical model of the well foundation, as shown in Fig. 1, is a three-dimensional network structure. This model is given an

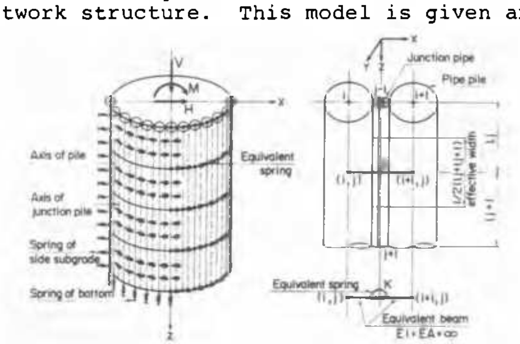

Fig. 1 Mathematical model of the Interlocked Pipe Piling

x-axis symmetric structure because the soil springs inside and outside the well foundation differ. The steel pipe piles are modelled as equivalent beam elements. The soil inside and outside the foundation are modelled as three-dimensional springs.

Equilibrium equation between pile and soil

Each of the steel pipe piles is assumed to be a beam on an elastic medium. The beam is divided into about ten elements according to depth and is supported by a uniformly distributed spring in the horizontal (x,y), vertical (z) and torsional directions.
The assumptions are:
(1) The load acting on the beam elements varies linearly in depth.
(2) Pipe deformations are evaluated as those of the junction spring.
The equations of equilibrium for the generic element are obtained as:

$$EI_{y(x)} \frac{d^4 \delta_{x(y)}}{dZ^4} + K_{x(y)} \delta_{x(y)} = q_{x(y)} + \frac{dm_{y(x)}}{dZ}$$

(x-z and y-z planes) (1)

$$-EA \frac{d^2 \delta_z}{dZ^2} + K_z \delta_z = q_z \quad (2)$$

$$-GJ \frac{d^2 \theta_z}{dZ^2} + K_\psi \theta_z = t_r \quad (3)$$

in which δ = displacement (m), θ = torsional angle (rad), q = lateral load function (t/m), t = torsion function (t·m/m), K = coefficient of subgrade reaction (t/m^2) and m = distributed moment function (t·m/m)

Idealization of the junctions

The junctions connecting the neighboring piles are modelled as equivalent beam elements connecting the neighboring panel points, as shown in Fig. 1. We assumed that:
(1) The elastic deformation of the equivalent beam elements is evaluated as that of the junction springs.
(2) Subgrade reaction acting on these elements is not taken into consideration.
(3) No external forces act on these elements.
From the experimental results, the relationship between the section force and the relative displacement of the neighboring piles is generally as shown in Fig. 2. It is assumed that the junction spring constant is expressed by Eq. 4.

$$Q_k = K_k (\delta_{i+1} - \delta_i) + C_k \quad (4)$$

in which Q_k = section force (t/m), K_k = equivalent spring constant (t/m^2), δ = relative displacement (m), C_k = constant (t/m)

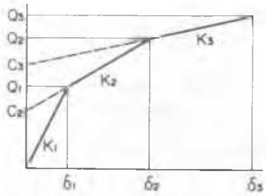

Fig. 2 The Relationship between the Section Force and the Relative Displacement of the Neighboring Piles

The equivalent spring constant at point j as shown in Fig. 1 varies linearly, given the assumption that the displacement over and effective width of 1/2 ($l_j + l_{j+1}$), including point j, shows a straight distribution that is governed by the displacement at point j.

Footing model

The heads of pipe piles are rigidly connected to a concrete footing of several meters in thickness and so the footing is modelled as a rigid body.

The ground model

The ground model consists of inelastic springs with the side subgrade having four elements (x, y, z - displacements and rotation about the z axis) and the bottom subgrade having six elements (x, y, z axes and the rotations of these axes). The distribution of subgrade reaction coefficients is uniform in each element. The coefficients of subgrade reaction inside the foundation are evaluated by multiplying those outside the foundation by a reduction constant.

Method of analysis

The lateral resistance of interlocked pipe piles is analyzed by three-dimensional network analysis. The displacement method is applied in this analysis.

EXPERIMENTAL INVESTIGATIONS

Test device and procedure

Experimental studies on three model structures as shown in Fig. 3 were carried out to find out whether the abovementioned theoretical analysis is valid. Dimensions of these models take into consideration their similitude with regard to actual structures. The cement paste is poured into the inside of the junction pipes. The rigidity and strength of these pipes have been previously analyzed by another model test. For the surface layer, pit sand having a maximum dry density of 1.7 g/cm^3 and a water content of 1.0% is used. For the basement, soil-cement having a cement-sand ratio of 8% and a water content of 1.0% is used. The rigidity of the model ground was analyzed by plate bearing tests after completing the experimental test. Experiments on the lateral behavior of model structures under vertical loading were carried out in a sand box of 10.4 x 4.0 x 3.0 m in size.

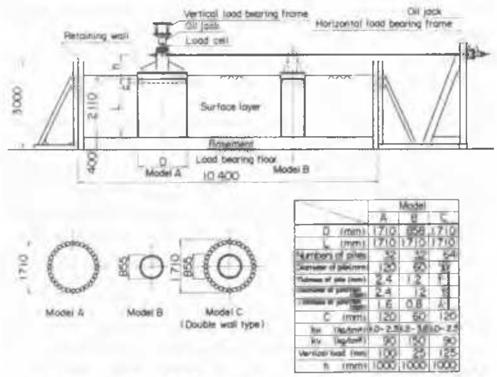

Fig. 3 Profile of Model Experiments

Test Results

The typical test results are shown in Figs. 4 to 6. Analytical results for Model A are shown in bold lines. Fig. 4 shows the relationship between the lateral load and the top deflection of the model structure measured at the centrally located pile. Analytical results for deformation in the elastic domain agree well with the corresponding experimental results, confirming the appropriateness of this analytical method. The nonlinearity of the curves in this figure is attributed to the soil's deformation properties. Fig. 5 shows the soil reaction distribution under a vertical load of V=100 tons and a horizontal load of H=24 tons. These calculated values conform fairly well with the measured values. The soil spring constants inside the foundation adopted in this calculation are 0.2 times as large as those outside.

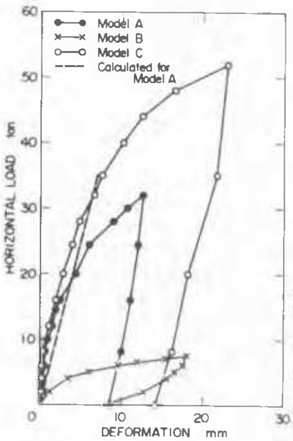

Fig. 4 The Relationship between the Lateral Load and the Top Deflection

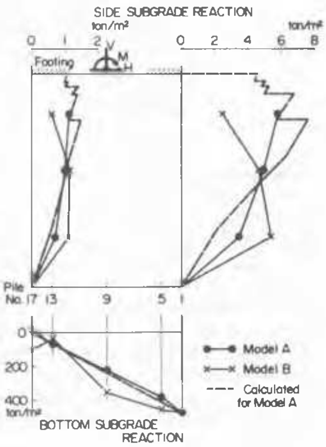

Fig. 5 The Soil Reaction Distribution

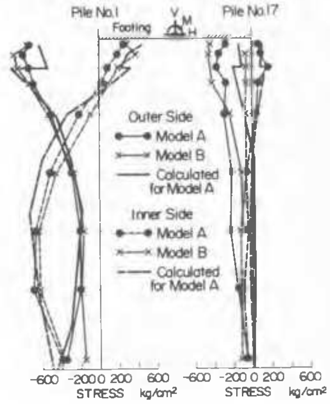

Fig. 6 The Bending Stress Distribution of Piles

Fig. 6 represents the bending stress distribution at No. 1 and No. 17 piles, as shown in Fig. 5, where maximum bending stress occurs. Though these calculated values are influenced by the confining moment of the footing, that moment is loosened by the cement paste inside the junction pipes. In the low lateral load domain, bending moments in the ground are comparatively large, so it can not necessarily be true that the confining bending moments are always prominent. It also has become clear that use of the existing method results in a foundation design that is exceedingly safe.

APLICATION TO LARGE-STRUCTURE FOUNDATIONS

Comparative design

On the occasion of constructing a large-scale foundation for a blast furnace, a design comparison of several types of foundations constructed of large-diameter steel pipe piles was carried out to obtain an aseismic design for the ground at the construction site which is very soft and frequently subjected to earthquakes. The comparative results were as follows.

(1) In a detailed comparison of the pile group foundation and the steel pipe pile well foundation from the aspect of aseismicity, the former is slightly superior with regard to the superstructure response while the latter is much superior as to the substructure response on soft ground. Therefore, the steel pipe pile well foundation was adopted for this foundation.

(2) Investigations concerning aseismicity were carried out for both the single and double wall type well foundations and no differences were found. The double wall type was introduced, since this well foundation is rigidly constructed by placing concrete between the inside and outside walls.

Execution design

Fig. 7 shows the cross section of the foundation adopted.

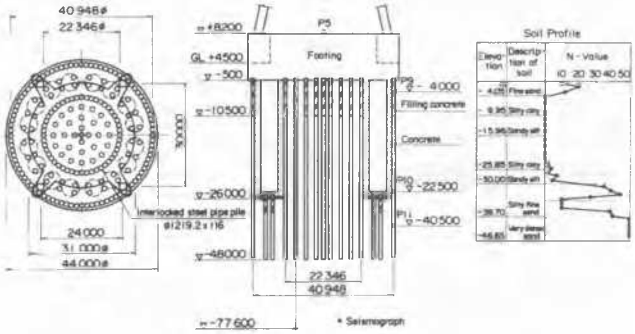

Fig. 7 Cross Section of the Double Wall Typed Well Foundation

(1) Static design

Table I shows the loading conditions and the allowable stress for structural steels. The abovementioned analysis was used for the structural calculation. Maximum stress in steel piles was 652 kg/cm^2 under normal conditions and 1526-1964 kg/cm^2 under earthquake conditions; vertical reaction on piles was 395 tons under ordinary conditions and 553-563 tons under earthquake conditions.

TABLE I The Loading Conditions and the Allowable Stress

	Vertical Load (ton)	Horizontal Load (ton)	Bending Moment(t·m)	Allowable Stress(kg/cm²) STK41	STK50
Ordinary	30500	400	32000	1400	2100
Temporary	30500	7200	227000	2100	2700

(2) Dynamic design

A lumped mass model of a superstructure, piles and surrounding soil as shown in Fig. 8 was adopted to simulate observed earthquake records. A bending and shearing type model was used for the entire structure (consisting of a superstructure and substructure), and a shearing type model for the ground. The first improved ground was

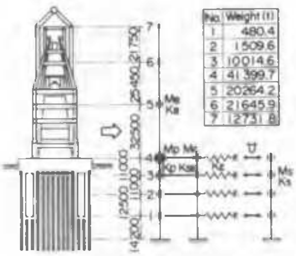

Fig. 8 Analytical Model of Soil-Structure Coupled System

considered to be natural ground and the second improved ground as an equivalent ground (T. Yamasaki et al, 1980). The damping ratio used was 10% for both the structure and the ground. Three observed earthquakes were evaluated by coupled vibration analysis and these maximum accelerations at the basement were 130 gal. The model structure had the fundamental period of 0.847 sec. Fig. 9 shows the storey shear coefficients. These coefficients indicate the highest value in Chiba (2.8, 1975) and also the lowest value in Chiba (11.16, 1974).

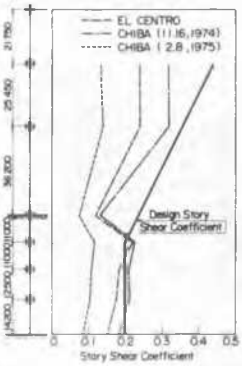

Fig. 9 The Storey Shear Coefficients

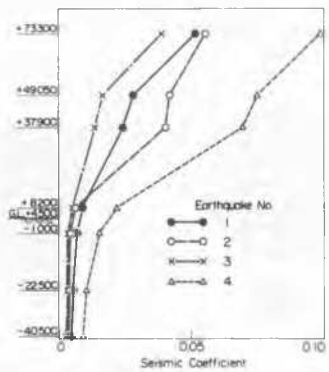

Fig. 10 The Distribution of Seismic Coefficients

This seems to suggest that the coupled vibration response depends largely on the predominant wave period of the earthquake. Namely, the period of the response spectrum was 0.6-0.7 sec for the El Centro earthquake, about 0.7 sec for Chiba (11.16, 1974) and 0.2 sec for Chiba (2.8, 1974). The displacement was 4.5-12.7 sec on the superstructure (mass No. 7) and 1.0-2.1 cm on the footing (mass No. 4). As regards foundation displacement, it was very small. This is attributable to the reinforcement of the foundation to a great depth and to ground improvement works. The storey shear coefficient calculated from static design seismicity, as shown in Fig. 9, was a little larger than the one calculated from the coupled vibration response; the seismic coefficient 0.2 used in the foundation design was particularly appropriate.

(3) Results of earthquake observations

Earthquake measurements have been taken since 1977 in order to investigate the dynamic behavior of the blast furnace. Eight servo-type accelerometers were set on the superstructure, on the well foundation and in the ground. The dynamic characteristics of comparatively large-scale earthquakes in 1978 as listed in Table II,

TABLE II List of Observed Earthquakes

Earthquake No.	1	2	3	4
Date	1.14,1978	2.20,1978	3.07,1978	6.12,1978
Time	12.24	13:36	11:48	17.14
Hypocenter Location	In the Sea Izuooshima	Off Miyagi Pref.	In the Sea Tokaido	Off Miyagi Pref.
Latitude	34°46′	38°45′	32°09′	38°09′
Longitude	139°15′	142°12′	137°45′	142°10′
Depth (km)	0	50	440	40
Magnitude	7.0	6.7	7.6	7.4
Epicentral Distance (km)	130	410	455	345
Seismic intensity	III	II	IV	IV

are presented in this paper. Fig. 10 shows the distribution of the seismic coefficients for the earthquakes. From this figure, it is confirmed that the seismic coefficients of the substructure are much smaller than those of the supersturcture. Judging from the storey shear coefficients, namely, 0.132 -0.318 at the superstructure and 0.074-0.132 at the substructure, as shown in Fig. 9, the constructed foundation has a rigidity above the design value, and the effect of ground improvement is substantially great.

CONCLUSIONS

(1) The mechanical behavior of a well foundation was exactly analyzed by three-dimensional network model analysis.

(2) The existing method is not good enough to cover a wide variety of well foundations, but the use of this method generally ensures a safe design.

(3) On the occasion of constructing a large scale foundation for the blast furnace on soft ground, design comparison for several foundation types was made from the view point of aseismic design, and the double wall type well foundation was introduced. From the coupled vibration response, the seismic coefficient of 0.2 which was used in the foundation design is the appropriate value.

(4) From earthquake observations, it can be concluded that this foundation has a rigidity above the design value and that the effect of ground improvement is significantly great.

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