FIELD TEST ON THE LATERAL RESISTANCE OF LARGE DIAMETER STEEL PIPE PILES
AND ITS APPLICATION TO THE ASEISMIC DESIGN OF PILE BENT-TYPE PIER

BY

YASUMARU   ISHII *
KEIICHI   FUJITA **

Abstract

In constructing Biwako-Ohashi Bridge, steel pipe piles, 1,200mm and 1,500mm in diameter, were used to allow lateral load of 30-43.5 ton/pile during earthquake.

Static lateral and forced vibration tests were performed at the site using single-top-free piles, and the data analysis disclosed that natural vibration period of pier was remarkably longer than predominant seismic vibration period.

Because of no danger of resonance expected, design was carried out on seismic coefficients, and lateral resistance was calculated by modified Chang's method. Design was checked with equations, $p=k'Bxy^{0.5}$ and $p=k''By^{0.5}$ conforming to static test results, and was confirmed to be sufficiently safe.

Nomenclature

P: Lateral load
p: Subgrade reaction on pile
y: Deflection of the embedded portion of pile
x: Vertical coordinate taking upwards positive
k: Subgrade reaction modulus for $p=kBy$ (kg/cm^3)
k':   Subgrade reaction modulus for $p=k'Bxy^{0.5}$ $(kg/cm^{3.5})$
k":   Subgrade reaction modulus for $p=k''By^{0.5}$ $(kg/cm^{2.5})$
B: Diameter or width of pile

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1. Design Problems:

To clarify certain engineering problems, particularly lateral resistance, of the pier structure of Biwako-Ohashi Bridge which is to be constructed with large diameter steel-pile piles, job-site experiments were carried out and carefully analyzied.

The Bridge, 1,350m long and 7m wide, was designed to allow for navigation underneath on the lake by providing 3-span continuous box-girder sections 95m, 140m and 95m long respectively. The water depth at this point was 7 to 8m with the bottom covered with 10m deep soft clay and the pier was designed to rise about 18m above water surface. Remaining spans are of simple-composite girders supported by piers at every 42.42m. (Figs. 1 & 2)

To construct these bridge piers, a method was selected to drive steel-pipe piles of 1,500mm to 1,200mm diameter, which are to be filled later with reinforced concrete, and then to begin pier-body concreting upward from 1 meter level above the water surface. As 16 piles of 1,500mm diameter were in place to construct the main pier with fixed bearing between main span, the design horizontal load was 43.5 ton/pile in earthquake, whereas the vertical load was 192 ton/pile under normal condition, and 328 ton/pile in earthquake. (Table-1.)

Main problems in the above design were the lateral resistance of the bridge piers against lateral load by earthquake and the calculation of displacement of the pier heads corresponding to the properties of bearing shoes. It was also necessary to find out how to prevent seismic resonance of the pier structure.

2. Description of Experiment:

Various experiments including vertical loading tests were made directly on the job-site piles as follows:

Alternative static
   lateral loading test $\emptyset$ 1,500mm x t 13mm x L 33,000mm
Ditto $\emptyset$ 1,200mm x t 12mm x L 34,000mm
Forced vibration test $\emptyset$ 1,200mm x t 12mm x L 34,000mm

All the above tests were performed with wire strain gauge in place, but without any reinforced concrete fill in the pipes. The tests were made with single, head-free piles, and the loading position and ground surface are as outlined in Fig. 3.

With a view to interfering least with construction schedule, vibration tests were not performed of the pier alone. The tests are, however, to be made only after the work is complete with the superstructure.

3. Test Results:

Figs. 4 & 5 show load — deflection curve and observed strain distribution derived from the alternative static loading tests conducted on $\emptyset$ 1,500mm and $\emptyset$ 1,200mm piles. In addition, the strain distribution calculated with various k values is given for comparison. Figs. 7 & 8 show resonance curve based on the forced vibration tests.

4. Studies on Static Loading Test Results:

Various analyses were made to interpret the data obtained from the static lateral loading tests. The results are as summarized below:

i) Analysis by Chang's method: Variables in Chang's equation p=kBy are substituted with the test figures and the k value thus obtained decreases with the load increase. Similar calculation on the maximum bending moment indicates that when the load is sufficiently small, k value approximates to that of top deflection, whereas k becomes extremely small as the load increases. For example, when $\emptyset$=1,200mm and P=3 ton, the k value corresponding to top deflection is 0.8 kg/cm^3 , whereas for max. bending moment, k=0.2 kg/cm^3 , and in case of P=12 ton, k=0.2 kg/cm^3 for top deflection and k=0.015 kg/cm^3 for maximum moment. In other words, Chang's equation does not allow calculating both the top deflection and the max. bending moment with the same k value. If calculated, however, by assuming the resisting level at a certain depth from the ground surface, the Figs. 4 & 5 indicate that the observed value approximates to the computed value by Chang's equation. This is considered to be caused by the fact that with increasing load, the resisting level near the surface is lowered thru failure. If apparent free length is properly selected for every load, the same value of k is applicable to calculate both top deflection and maximum bending moment. The results indicate that depth of failure zone increases with load, but the k value stays between 1.2 and 2.0 kg/cm^3 .

ii) Analysis by means of p=k'Bxy$^{0.5}$(1)(2):   When analized with Shinohara-Kubo's method utilizing the conversion table derived from the standard curve of similitude law with reference pile, both the load — deflection and load — max. bending moment may be calculated for all loads at the same k' value of 0.003 kg/cm$^{3.5}$ and without changing the original free length. The results thus calculated conform very well to those of experiment. (See Figs. 7 & 8) Graph reflecting the correlation of N in standard penetration and k' clearly indicates that this k' value corresponds well to the ground properties.   .

iii) Analysis by means of p=k"By$^{0.5}$(3):   Conversion table by p=k"By$^{0.5}$, when utilized with the same reasoning as in the (ii) above, gave almost the same results, provided that k"=0.33 kg/cm$^{2.5}$ is assumed herein. (See Figs. 7 & 8)

5. Studies on the Vibration Systems of the Piles:

By means of various analytical methods as stated below, relations between k value and proper frequency are calculated and compared with the results of the forced vibration test. The models for vibration system used in the analysis are as shown in Fig. 9.

i) Analysis by Elastic Cantilever System: Natural frequency is first calculated by presuming a cantilever with fixed end at the first deflection zero point and having load concentration at its free end or at pile head. By making following assumptions that amplitude and deflection angle are zero at fixed end, that bending moment is zero at free end, and that the shear equals the inertia force of mass, natural frequency is given by such data as surcharge, cantilever length, weight and rigidity of piles in place. Correlations between subgrade reaction modulus and natural frequency by this method are as shown in Fig. 10.

ii) Analysis by the Rigid Inverted Oscillator: As in the model shown in Fig. 9 (a), by assuming spring and damper at pile head, spring constants are calculated from the pile head deflection and the first deflection zero point, and natural frequency is obtained therefrom. Correlation between subgrade reaction modulus and natural frequency by this method is given in the curve in Fig. 10.

iii) Analysis by Hayashi-Miyajima's Method [3][4]:   This is the modified Klotter's method assuming non-linear spring constant system. In the load — deflection curve of the alternative static lateral loading test, maximum load for each stage lasted for 2 hours. After eliminating by parallel shift the hysterisis, a frame work was made up to determine spring constants and damping factors. The resonance curve prepared closely approximates the experimental results. (See Fig. 11) This method makes it possible to calculate values of amplitude and frequency similtaneously.

iv) Analysis by Lumped Mass System: Any actual structure has its mass distributed continuously over its entire extent. In general, one equilibrium equation is written for each element of mass. Consequently, as infinite number of equations are required to define the equilibrium of the complete structure. In order to reduce the analysis to practical proportions, it is customary to replace the distributed mass of the actual structure with a relatively limited number of concentrated point mass. In this case, we assumed such a model as in Fig. 9 (b) and obtained the natural frequency of 0.8 cycle/sec as a result of the computation. This result is a little far from the experimental result. But the former can approach the latter if we select the appropriate values of spring constants and the positions of the lumped masses.

v)   Analysis by Simplified Methods: Analysis is made in the simplest form of model as shown in Fig. 9 (c). Surcharge was adopted to determine the size of particles, and the spring constants were calculated from the results of alternate loading tests, where the mass of piles or of spring was neglected. In calculating the amplitude, damping coefficients taken from the vibration test were put to use, and the resonance curve pattern is found to approximate well to the test results. (See Fig. 12)

Comparison of Various Methods:

It is found out that in case natural frequency is the only concern, either the simplified method or the elastic cantilever system method can be used for the calculation, but if it is necessary to calculate various design elements for each components of the bridge pier, the lumped mass system will be convenient. Whatever methods used in this case are intended to give approximate figures, and proper methods meeting each purpose should be selected in accordance with conditions whether static or dynamic test results are available or not.

The conceptions of virtual mass of surrounding water has been examined in many ways, and consideration of such factors in the said calculation has given practically the same results in this case and is therefore negligible in this experiment.

6.   Proposal:

Regarding natural vibration period calculation, it is made clear that the forced vibration test results of single piles can be computed by means of modulus of ground reaction similar to the static test results. As described above, natural vibration period can be calculated by elastic cantilever method and others. The amplitude can be calculated by selecting proper damping coefficient.

In designing actual structures, various models as mentioned in the Paragraph 5 and the vibration to be encountered in earthquake were both employed to obtain dynamic response of the structure, from which the design elements are obtainable. However, since hardly any seismic records or relevant data are available to date, the application of such methods in actual design is considered to be premature.

After examination of such experimental and analytical data, it is confirmed that when pile heads are loaded with heavy concrete body and superstructure, and are subject to dynamic water pressure due to earthquake, the vibration period becomes much longer than 1 second representing the natural vibration period based on the single pile experiments.

Since prevailing vibration period of any destructive earthquake in the vicinity of the job-site is considered to be about 0.2 to 0.4 second, it may be safely considered that hardly any danger of resonance is expected in this instance. Therefore, static calculation methods are to be used with the following assumptions:

Lateral force of earthquake (based on the conventional
                              seismic coefficients)   0.2 g
Thickness of failure zone   5 m
Free length for calculation   Original free length + 5 m
Pile head   Fixed
k - value   0.8 kg/cm^3
Equation   I.L. Chang's method

By assuming lateral force at pile heads and by converting the total overturning moment into a vertical force for the pile, the vertical load capacities of the piles are examined. Since the maximum bending moment at the pile butt is made smaller due to the rotation caused by vertical displacement, the said moment is neglected in the design. As for the underground portion of piles, however, the moment due to the said rotation are to be taken into account because of its considerable influence.

7. Check of Design

The design was made by Chang's method as modified for a failure zone. As previously explained, it is advisable that the relation between top deflection and max. moment should be calculated by equations $p=k'Bxy^{0.5}$ and $p=k''By^{0.5}$. Regarding the equation $p=k'Bxy^{0.5}$, standatd curves are prepared for the top-free and top-fixed conditions, and by making use of the curve, the design check was carried out and the results are as outlined in Table-II.

In either case, the design figures are found to be larger than those checked, and it is confirmed that sufficient safety is considered in making the design. For example, the pile-displacement at ground level was found to be 1.3 to 1.6 times, and the maximum bending moments about 1.4 times respectively as much as the checked figures.

8. Acknowledgements

We also take this space to extend our words of hearty thanks to
Prof. A. Hirai of Tokyo University, and to Messrs. S. Hayashi,
K. Kubo, N. Miyajima and I. Yamashita of Port & Harbour Technical
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of Yawata Iron & Steel Co., Ltd. and Messrs. H. Tanaka and Y. Yamaguchi
of Hazama-Gumi, Ltd. for their active helps and suggestions in this
preparation.

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Pile, Report No.2 and No.3", Monthly Report of T.T.R.I.,
Report No.2, Dec. 1961 and Report No.3, March 1962. (In Japanese)

(3)   Port & Harbour Technical Research Institute, Ministry of
Transportation, Japanese Government, and Yawata Iron & Steel
Co., Ltd., "Studies on the Lateral Resistance of H-piles",
March 1963. (Mostly in Japanese with conclusion in English)

(4)   Hayashi S., Miyazawa, N. and Yamashita, I., "Lateral
Resistance of Steel Piles against Statical and Dynamical Loads,"
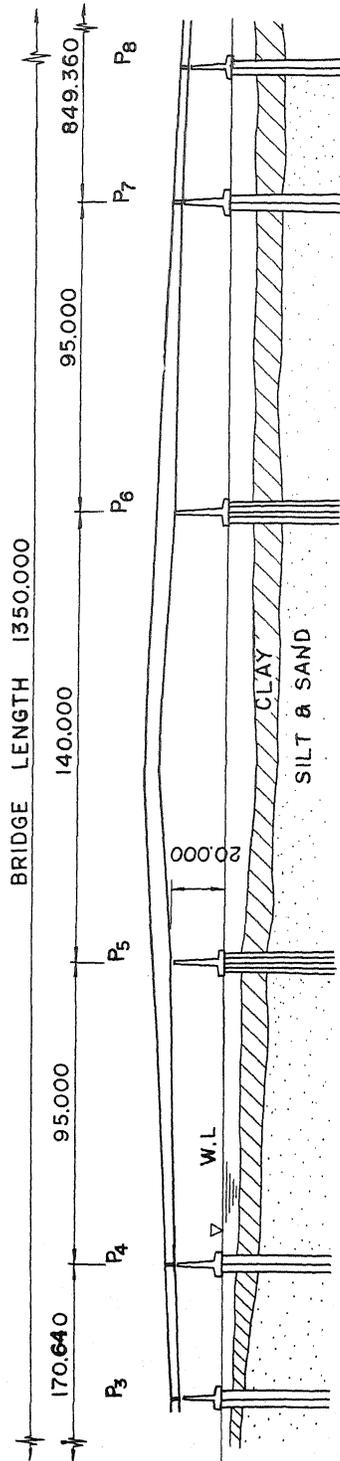Proc. Third World Conference on Earthquake Engr. 1965.

Fig. 1 Profile showing main span scheme of Biwako-Ohashi Bridge which included a 95 – 140 – 95m central system, and geological details of the foundation
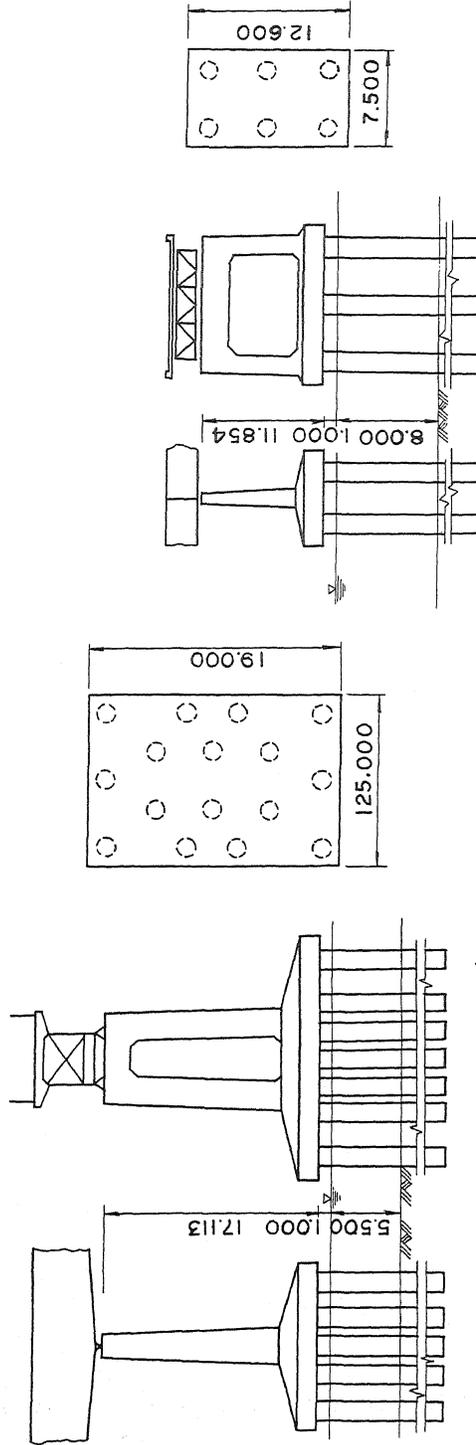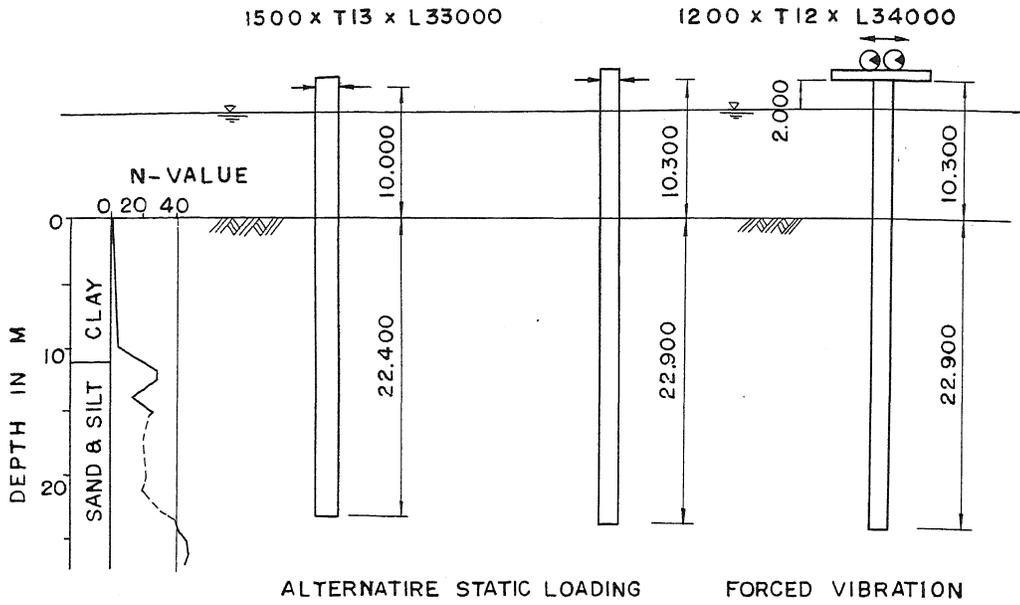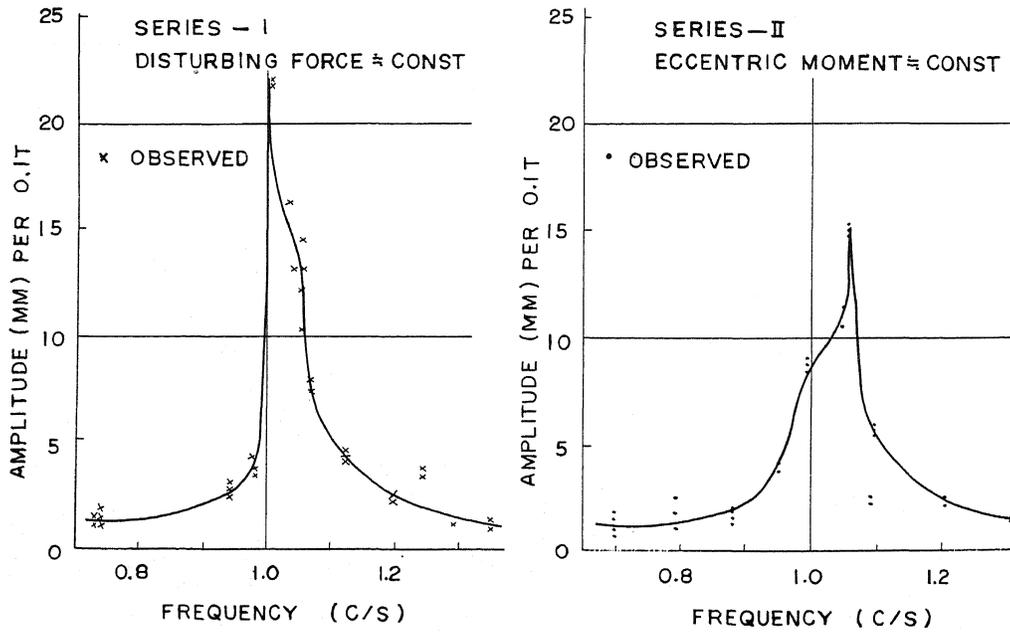

Fig. 2 General features of piers P_5 & P_9

1500 x T13 x L33000   1200 x T12 x L34000

N-VALUE

ALTERNATIRE STATIC LOADING   FORCED VIBRATION

Fig. 3 Test conditions

SERIES — I
DISTURBING FORCE ≒ CONST

× OBSERVED

AMPLITUDE (MM) PER 0.1T

FREQUENCY (C/S)

SERIES—Ⅱ
ECCENTRIC MOMENT ≒ CONST

• OBSERVED

AMPLITUDE (MM) PER 0.1T

FREQUENCY (C/S)

Fig. 6 Resonance curves of forced vibration test
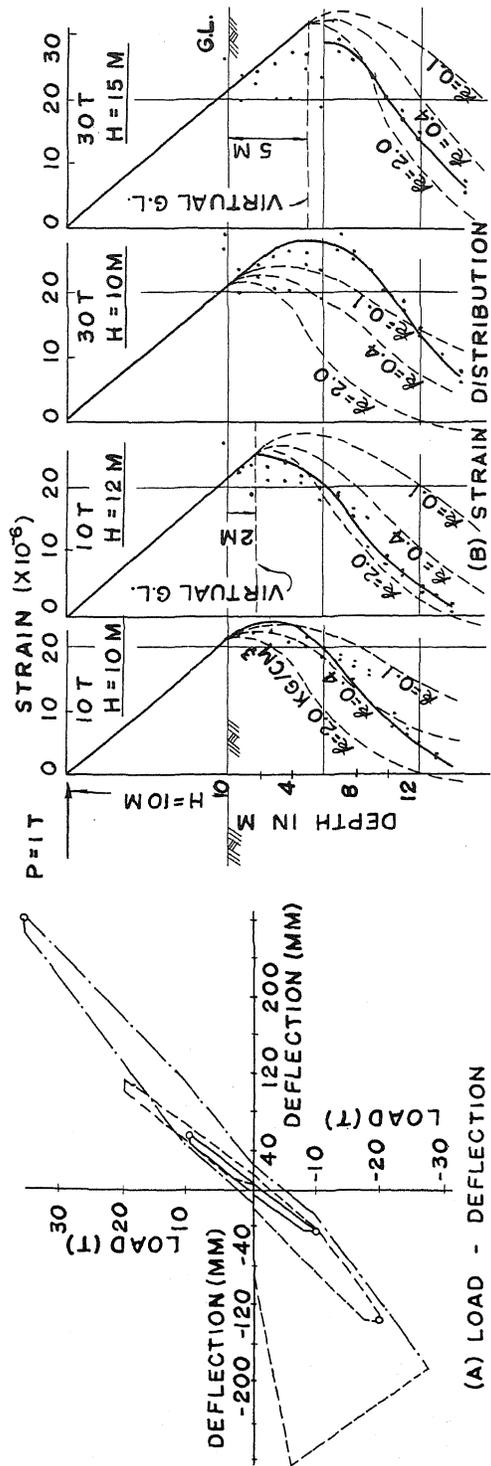
Fig. 4 Alternative loading test results (ø 1500mm x t 13mm)

(A) LOAD - DEFLECTION (B) STRAIN DISTRIBUTION
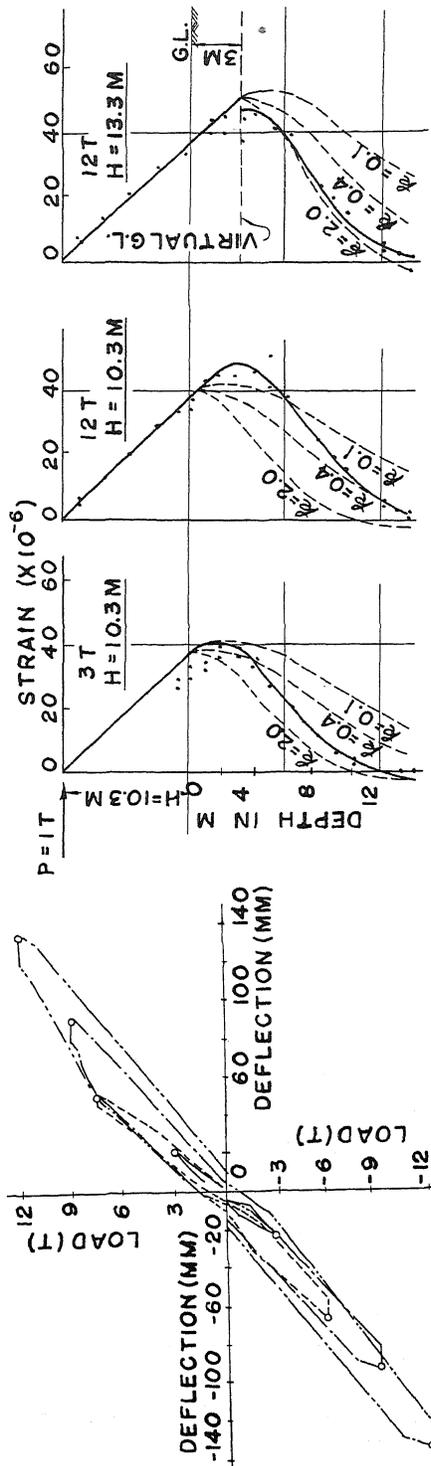
Fig. 5 Alternative static lateral loading test results (ø 1200mm x t 12mm)

(A) LOAD - DEFLECTION (B) STRAIN DISTRIBUTION

---- COMPUTED BY $P=K'BXY^{0.5}$
$K'=0.003$ KG/CM$^{3.5}$

—— COMPUTED BY $P=K''BY^{0.5}$
$K''=0.33$ KG/CM$^{2.5}$

---- COMPUTED BY $P=K'BXY^{0.5}$
$K'=0.003$ KG/CM$^{3.5}$

—— COMPUTED BY $P=K''BY^{0.5}$
$K''=0.33$ KG/CM$^{2.5}$

Φ 1200×12
× OBSERVED

Φ 1200×12
× OBSERVED

LOAD (T)

LOAD (T)

5 10   50 100 200
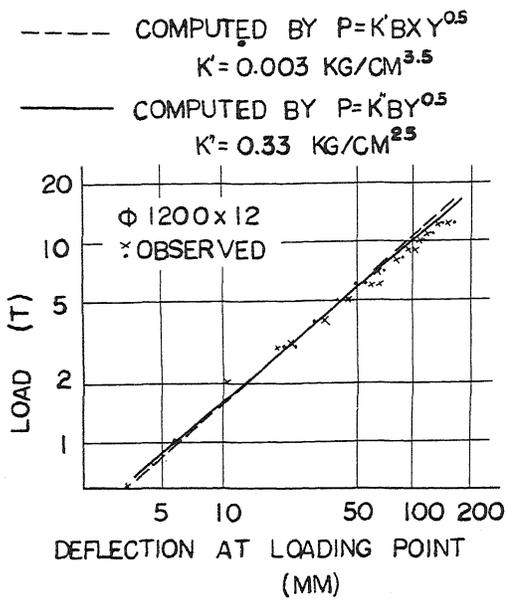DEFLECTION AT LOADING POINT
(MM)

10   20   50 100 200
MAX BENDING MOMENT (T-M)

Fig. 7 Load—deflection curve
observed & computed
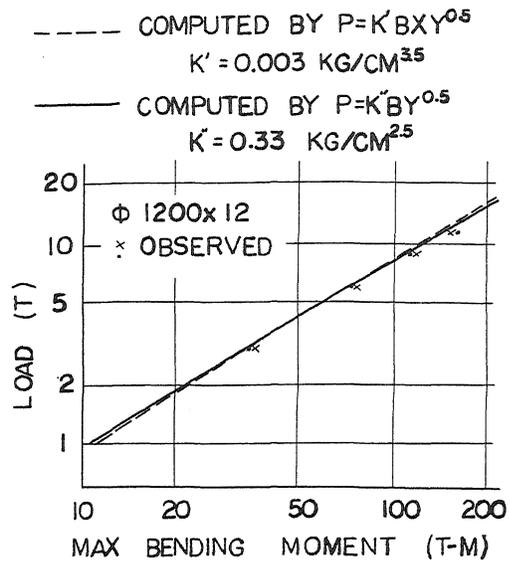
Fig. 8 Load—bending moment curve
observed & computed

M K_0

(A) RIGID INVERTED
OSCILLATOR SYSTEM

$K=1.0×10^3$ KG/CM
M
$\Lambda=0.023$

(C) SIMPLIFIED METHOD

550 CM
1,100 CM
1,150 CM
600 CM

MA = 22.45

MB = 16.64

MC = 21.74

KC = 5.28 × 10^5

MD = 23.67

KD = 2.88 × 10^7

(M : KG - SEC^2/CM
  K : KG/CM
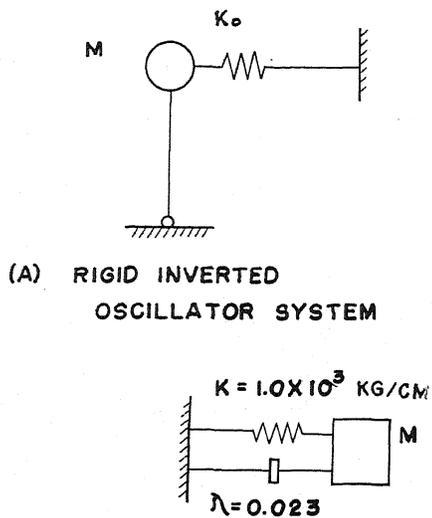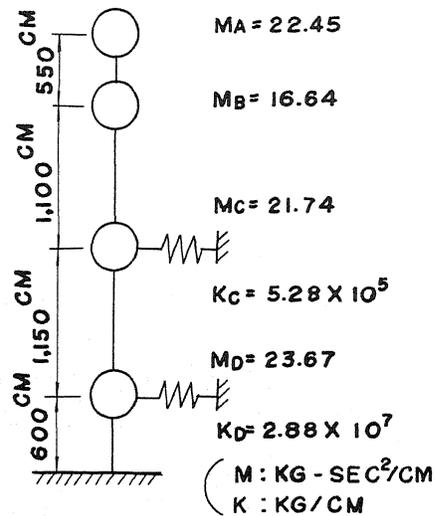
(B) LUMPED MASS SYSTEM
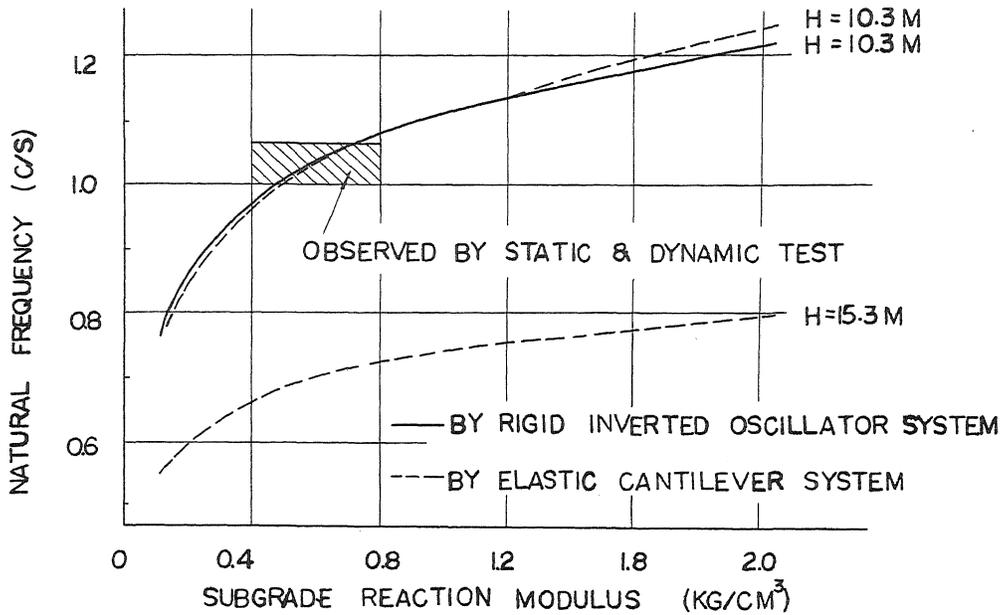
Fig. 9 Model for analysis of dynamic response

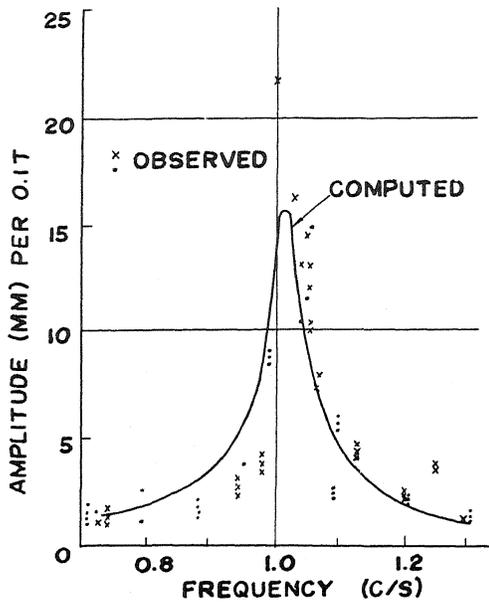Fig. 10 Relation between subgrade reaction modulus
to natural frequency

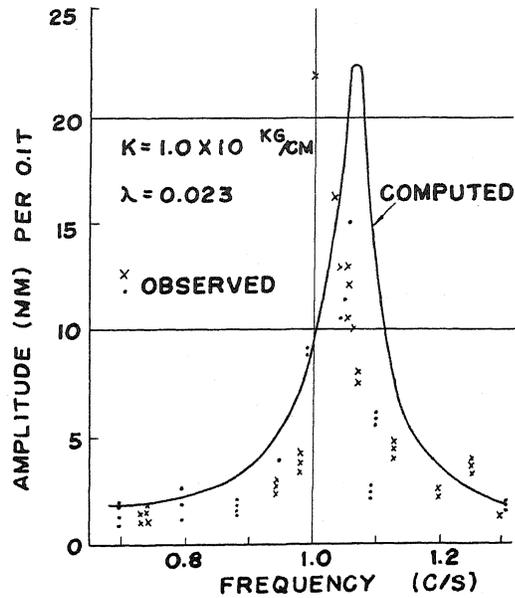Fig. 11 Resonance curve by
Hayashi-Miyajima's method

Fig. 12 Resonance curve by
Simplified method

TABLE - 1

Design Data of Piers & Piles

Pier	Size Pile (m)		No. of Pile	Load per Pile (t)			Free Length of Pile (m)	Height of Concrete Pier Body (m)
	Dia.	Length		Vertical (max.)		Lateral		
				Normal	Earthquake	Earthquake		
P_1	1.2	22.5	4	140.5	167.8	19.9	3.0	9.773
P_5	1.5	29.5	16	192.4	327.9	43.5	6.5	17.113
P_6	1.5	32.5	16	192.4	223.9	25.2	8.5	17.113
P_7	1.5	38.5	8	178.3	250.7	27.3	8.7	16.065
P_9	1.5	33.0	6	202.6	266.6	30.4	9.0	11.854
P_{11}	1.2	34.0	8	129.8	200.6	19.2	8.7	7.612
P_{17}	1.2	35.5	5	144.8	143.6	20.7	7.7	3.747
P_{26}	1.5	33.0	2	255.6	167.5	30.1	8.9	1.800

Note: Number of Piers: 26

Number of Piles: ϕ 1,200mm 75 ea.
ϕ 1,500mm 79 ea.

Total Steel Weight of Pile: 1,836 t

Thickness of Pipe Pile: 9-13mm

EI for Design: ϕ 1,200 2,810 x 10^{12} kg/cm²
ϕ 1,500 6,203 x 10^{12} kg/cm²

TABLE II

Deflection & Bending Moment of Piles Designed & Checked

Pier No.	Lateral Load (t)	Deflection of Pile (cm)				Max. Bending Moment (t-m)		Depth of First Zero Point of Bending Moment (m)	
		Ground Line		Top					
		Design*	Check	Design	Check	Design	Check	Design**	Check
P₅	43.51	1.27	0.99	3.91	1.79	395.3	281.0	17.32	15.5
P₆	25.15	0.95	0.58	2.99	1.35	254.6	188.0	17.76	14.4
P₇	27.31	1.02	0.68	3.40	1.58	280.6	203.0	17.80	14.8
P₉	30.37	1.18	0.80	3.92	1.88	314.6	234.3	17.88	14.7
P₁₂	18.03	1.10	0.79	4.22	1.93	176.7	133.0	16.44	12.9
P₁₇	20.69	1.17	0.83	4.16	1.95	191.9	138.0	16.26	13.0

Design : .... by Y.L. Chang's method
Check   : .... by $P = k'Bxy^{0.5}$ method

* : Virtual ground line i.e. 5m below ground line
** : Depth below ground line

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AND ITS APPLICATION TO THE ASEISMIC DESIGN OF PILE BENT-TYPE PIER

BY Y. ISHII AND K. FUJITA

QUESTION BY: J.B. WILSON - NEW ZEALAND

Why were some of the piles not inclined to the vertical i.e. raked?

AUTHORS' REPLY: With large diameter piles actual driving and incorporation of the tops in a slab become most difficult, therefore a battered pile foundation is never favourable. Moreover, the vibration of liquefied soils produces vertical loads in the inclined piles.

QUESTION BY: J.F. BORGES - PORTUGAL

I think it may be of interest to compare the results presented with those recently obtained by testing the piers of Tagus suspension bridge in Lisbon (fig.1). The amount of damping obtained there is considerably higher than that indicated for the large diameter piers.
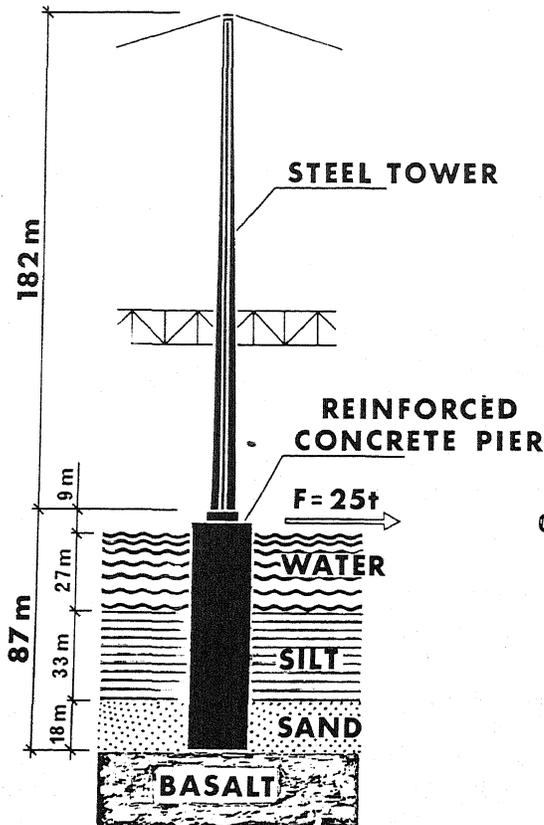
DYNAMIC TEST

NATURAL FREQUENCY 1.4 Hz

FRACTION OF CRITICAL DAMPING .... 5 %

1 SECOND
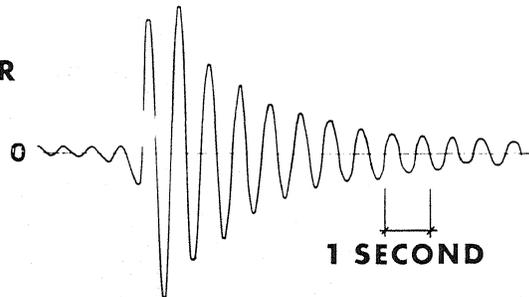
COMPUTED NATURAL FREQUENCY .... 1.1 Hz

ASSUMED FRACTION OF CRITICAL DAMPING .... 10 %

AUTHORS' REPLY: The writers may not discuss this question without detailed information of your tests. In the writer's test, the vibrating force is very small compared to that of an actual earthquake and the amount of damping, therefore might be small.

QUESTION BY: Z. BAZANT - CZECHOSLOVAKIA

We shall appreciate data concerning the estimated settlement during the earthquake, of piles which are founded at a depth where no compaction can be expected. But the approaches have a depth of foundations of smaller depth. There is no detail showing that there may be during an earthquake, trouble at spans which connect both parts.

AUTHORS' REPLY: It is true. Some differential settlement between abutments and end piers must be expected in an earthquake.