

Some Experimental Studies on the Earthquake Proof Design
of the Foundation of Bridge Pier in Soft Ground

by Yasuji TAHARA¹, Takanobu TAKATA² and Masami FUKUOKA³

I. Synopsis

The authors have made a series of experiments on the well foundation of various rigidities and lengths which exist on various ground conditions, in order to solve, from the experimental standpoint, the behavior and vibration characteristics of bridge foundation such as wells due to the horizontal force caused by earthquake.

First, the authors have set up a K-value measuring device to determine the modulus of lateral reaction of foundation ground. Then, by means of this device, they have determined, at the job site of bridge erection, the moduli of static lateral reaction with the repeated loadings for the various depths of foundation ground.

Second, after a well has been sunk, they have determined the dynamic displacement and bending deformation of a well subjected to the horizontal vibration, and the values and distribution of apparent moduli of lateral reaction of foundation ground along the well depth. For these determinations, a simple horizontal vibration was given to the well by means of a vibrator and earth pressure cells, pore pressure meters, acceleration meters and velocity meters were set and recorded within the well.

A comparison of the K-values at the various depths between the above two determinations has shown a similar relationship, generally the former values being a little smaller than the latter ones.

If the modulus of static lateral reaction is determined in the bore holes, within a reasonable accuracy, by means of the simple device the authors have proposed before the well is designed, and if the correlations between the static K-values thus measured and the dynamic K-values are justified with a number of back data through extensive experiment works, this research work of which subject the authors have reported herein at an intermediate stage of research progress would be useful for the more rational earthquake-proof design of well foundation.

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1. JAPAN HIGHWAY PUBLIC CORPORATION
2. THE PUBLIC WORKS RESEARCH INSTITUTE, MINISTRY OF CONSTRUCTION
3. THE PUBLIC WORKS RESEARCH INSTITUTE, MINISTRY OF CONSTRUCTION

II. On Measurement of the Lateral Counterpressure Coefficient K of a Foundation Ground by Utilizing a Bore Hole

I. Forewords

It is necessary to determine the lateral K-value of the foundation ground in order to investigate stability of a foundation structure acted upon by a horizontal force.

The authors invented a device to measure the lateral K-value which is operated in conjunction with a bore hole. Given repeated improvements at every opportunity available, the device has been able to achieve recommendable results. The measurement procedures and examples of application will be outlined in the forthcoming chapters.

2. Measuring instruments and procedures

The device, shown diagrammatically in Fig. 1, consists mainly of the following parts.

- (1) A part mainly consisting of a rubber tube which is to be inserted into a Bore hole. (See Fig.2)
- (2) Tank A which supplies the pressure water, equipped with the glass-made stand pipe M to read the head inside the tank and with a Bourdon gauge to measure the pressure.
- (3) Tank B which stores the compressed air.
- (4) Compressor, The rubber tube, 110 mm I.D., 0.8 mm thick and 2.0 m long, consists of 2 folds of relatively soft elastic rubber. The upper and lower ends of the rubber tube is tightened with a rubber packing and metal fittings in order to ensure water-tightness. The bottom is cone-shaped so that it may penetrate slightly into the bottom of the bore hole. Along the center of the rubber tube a boring rod with a perforated wall is inserted, from which the pressure water enters the rubber tube.

The upper end of the perforated rod is connected directly to the ordinary 40 mm boring rod, which is then connected to Tank A by a high-pressure rubber hose. A pipe installed at the upper end of Tank A is directly connected to Air Tank B, which is then connected to the Compressor. Since these equipments are exposed to a pressure ranging between 3 - 5 kg/cm², every joint must be completely sealed with a rubber packing or enamel during assembling.

In using the device for a test, a bore hole is sunk by a 110 mm core-tube and further treated with a bailer in order to rid unevenness from around the hole. A rod connected to the tube is lowered into the hole slowly enough not to cause damages to the rubber tube. After the tube is settled at a required position, water is supplied through the rod while measuring the quantity until the rod is filled up to the top. The rod and Tank A are then connected by the high-pressure rubber pipe taking care not to leave air bubbles inside. After reading the head of Tank A, the valve V₁ is opened and the pressure applied. The pressure

is increased step-wise by 0.25 kg/cm², and at each stage of pressure the time-head variation inside Tank A is recorded by reading Stand Pipe M. Since the pressure decreases slightly according to the variation of the head, the valve V₂ must be adjusted to maintain a constant pressure. When the head variation decreases to less than 1 mm per 2 minutes, the next stage of pressure is applied. After the pressure has reached 1.5 - 2.0 kg/cm², it is lowered slowly until the rubber tube barely touches the hole sides. The same operation is repeated all over again.

The pressure which is transmitted through the high-pressure rubber pipe and the boring rod to expand the rubber tube is read from the Bourdon gauge attached to Tank A.

The outer wall of the rubber tube oppresses the inner side of the boring hole, producing a radius increment Δr . This increment can be computed from the head variation ΔH of Tank A. This condition is illustrated in Fig.3. Denoting the radius of Tank A by R, the radius of the rubber tube under zero pressure by r, and the length of the rubber tube by l , the equation (I) results.

$$\Delta r = -r + \sqrt{r^2 + \frac{R^2}{l} \Delta H} \quad \dots\dots\dots (1)$$

Substituting into the equation (I) the magnitude of the measuring instrument used by the Public Works Research Institute of the Ministry of Construction, i.e. r=2.0 cm, R=16.5 cm and $l=200$ cm, the equation (2) is obtained.

$$\Delta r = -2 + \sqrt{4 + 1.36 \Delta H} \quad \dots\dots\dots (2)$$

3. Measurement of Lateral K-value

19 separate measurements of the lateral K-values were made at three places, namely the Shin-ogata bridge, the Nishiarai bridge, and the Great Yoshida bridge. Let us first review the representative results of measurement.

Fig. 4-1 shows the relationship between the radius increment and the inner pressure of the tank measured inside the representative sand layer lying at the depth of 22 - 24 m. For the inner pressure of the tank equal to 0.25 kg/cm², little increment of radius occurred because the inner pressure of the rubber tube was lower than the outer pressure (bentonite hydraulic pressure inside the boring hole). When the inner pressure reached 0.5 kg/cm², the radius increased abruptly by 3.3 cm, presumably as the result that the rubber tube has felt the side wall of the boring hole. The radius of the boring hole was, at this stage, 5.3 cm, i.e. the sum of the rod radius r=2.0 cm and the radius increment. The K-value was computed from both the increments in the inner pressure and radius at the instants when the rubber tube first felt the side wall of the boring hole and when it showed a maximum value.

It is evident also from this test that in a sand layer recovery of the once compacted soil is generally less than that of a clay layer.

Fig. 4-2 shows a relationship between the radius increment and the inner pressure of the tank measured in a silt layer which is located at the depth 5.5 - 7.5 m. While the inner pressure of the tank was still 0 kg/cm², the radius increment has already bounded to 2.9 cm. This increment has been produced solely by a static pressure, since the inner pressure (static) was greater than the outer pressure (bentonite pressure) of the rubber tube due to shallow position of measurement. The rate of radius increment for 0 - 0.25 kg/cm² is great probably because of influences of slime inside the boring hole. At this time the radius of the boring hole was 5.6 cm. As the rubber tube was torn at the inner pressure 2.25 kg/cm², further measurements were abandoned. The K-value was obtained through the same procedure as used for a sand layer.

The results of the measurements made at the construction site of the Great Yoshida bridge are also illustrated in Fig. 5. Other values of measurement are given in Table 1 together with the results of the soil tests.

III. Measurement of the Ground Counterpressure Coefficient K through a Vibration Test for a well Foundation

I. Vibration test for a well foundation

This chapter will deal with distribution of the counterpressure and the counterpressure coefficient K of a ground when a bridge body is subjected to a dynamical lateral load, with reference to the results of a vibration test made for a well foundation during erection of the Great Yoshida bridge. The test procedures are outlined in Fig. 11-1. A sinusoidal wave of a forced horizontal load is applied by a vibrator mounted on a slab lying on the top of a well in which earth pressure gauges, pore pressure meters and acceleration meters are installed, and distribution of the earth pressure and acceleration induced in various segments of the well were determined. At the same time a velocity meter was installed on the upper slab and also an acceleration meter on the top of the well. Fig. 11-2 shows the static earth pressure and hydrostatic pressure which was found to act on the well immediately before the test. The test was conducted according to 3 separate runs as follows.

- Run 1. Measurement of displacement distribution in the well.
- Run 2. Measurement of earth pressure distribution acting of the well.
- Run 3. Simultaneous measurement of earth pressure and displacement at the same position.

Each run was repeated several times under the identical conditions by gradually increasing the revolving rate of a vibrating motor. However, the earth pressure meter failed to furnish sufficient data due to various damages inflicted during installation. The displacement was obtained by transforming once and twice from the records of the velocity meter and the acceleration meter. It was so arranged that the forced load P of the vibrator was proportional to the square of the revolution of an eccentric mass. As shown in Fig. 10 the displacement y and earth pressure p at each

position are also roughly parabolic against forced frequencies f . According to Fig. 7 which plots the horizontal displacement y against the depthward direction of the well, it is clear that the vibration in our case is a perfect rocking vibration. (The acceleration meter A5 is in the reversed phase to A0 and A4. See Oscillograph Record Fig. 8)

Fig. 10-1 plots the relationship $kd = p/y$ against the forced frequencies f from the measurements of displacement and earth pressure obtained at the same position. The kd values obtained from the average displacement and earth pressure from Runs 1 and 2 also plotted in Fig. 11-2.

2. Determination of K-value

In the above procedures the K-value could be determined directly for the positions where the earth pressure p and the displacement y were obtained simultaneously. However, since this value characterizes the nature of any particular position, it should not be extended to the overall vibration pattern of the well. The K-value for the entire well vibration is determined as follows.

According to the actual measurements the elastic deformation of the well vibration caused by a forced vibration $P \sin \omega t$ by a vibrator is negligibly small that it can be regarded as a perfect rocking motion. Therefore, in this case, it is sufficient to consider the horizontal and rotating motions in regard to the center of gravity G of the entire well. As a result the following equations are obtained. (See Fig. 9)

$$\frac{W}{g} \frac{d^2 y_G}{dt^2} + \int_0^l K(x) y \cdot dx = P \sin \omega t \quad \dots\dots\dots (3)$$

and
$$I_G \frac{d^2 \theta}{dt^2} + \int_0^l K(x) y (l_G - x) dx = P (l_G - l_0) \sin \omega t \quad \dots\dots\dots (4)$$

where y_G and θ denote horizontal displacement of the center of gravity G and the rotating angle around G , respectively. Therefore, they can be rewritten as,

$$y = y_G + \theta (l_G - x) \quad \dots\dots\dots (a)$$

The distribution of the ground counterpressure K is often expressed generally as,

$$K(x) = K_0 \left(\frac{x}{l}\right)^n \quad \dots\dots\dots (b)$$

where n is an index value dependent on the soil nature, and K_0 and n are unknown. Assuming that such treatment as above is allowable, let us determine K_0 and n , i.e. distribution of the K-value from the equations (3) and (4).

Putting

$$\frac{x}{l} = \xi, \quad \frac{l_0}{l} = \xi_0, \quad \frac{l_G}{l} = \xi_G \quad \dots\dots\dots (c)$$

the equations (3) and (4) reduce to the following equation

$$\left\{ \frac{W}{g} \frac{d^2 y_G}{dt^2} + K_0 l \int_0^1 \xi^n [y_G + \theta l (\xi_G - \xi)] d\xi = P \sin \omega t \dots\dots\dots (3) \right.$$

$$\left. \left\{ I_G \frac{d^2 \theta}{dt^2} + K_0 l^2 \int_0^1 \xi^n [y_G + \theta l (\xi_G - \xi)] (\xi_G - \xi) d\xi = P l (\xi_G - \xi_0) \sin \omega t \dots\dots\dots (4) \right. \right.$$

also putting

$$y_G = y_0 \sin \omega t, \quad \theta = \theta_0 \sin \omega t \dots\dots\dots (d)$$

We obtain the equations

$$\left\{ -\frac{W}{g} \omega^2 y_0 + K_0 l F_1(n) = P \dots\dots\dots (5) \right.$$

$$\left. \left\{ -I_G \omega^2 \theta_0 - K_0 l^2 F_2(n) = P l (\xi_G - \xi_0) \dots\dots\dots (6) \right. \right.$$

where

$$F_1(n) = \frac{y_0 + l_G \theta_0}{n+1} - \frac{l \theta_0}{n+2} \dots\dots\dots (e)$$

$$F_2(n) = \frac{(y_0 + l_G \theta_0) \xi_G}{n+1} - \frac{y_0 + 2l_G \theta_0}{n+2} + \frac{l \cdot \theta_0}{n+3}$$

Therefore, according to the equation (5), (6)

$$K_0 l = \frac{P + \frac{W}{g} \omega^2 y_0}{F_1(n)} = \frac{P (\xi_G - \xi_0) + \frac{I_G}{l} \omega^2 \theta_0}{F_2(n)} \dots\dots\dots (7)$$

The values of K_0 and n are determined from the eq. (7)
To the well

$$\frac{W}{g} = 1.220,4 \times 10^3 \text{ kg} \cdot \text{cm}^{-1} \cdot \text{s}$$

$$I_G = 5.673,7 \times 10^8 \text{ kg} \cdot \text{cm} \cdot \text{s}^2$$

$$l_0 = 150 \text{ cm}$$

$$l_G = 1220 \text{ cm}$$

$$l = 2300 \text{ cm}$$

And taking the case of frequency in 10 cycles per second

$$\omega^2 = 3948 \text{ s}^{-2}$$

$$P = 7338 \text{ Kg}$$

and also we get following values from Fig.7.

$$\left\{ \begin{array}{l} y_0 = 5.00 \times 10^{-3} \text{ cm} \\ l_G \theta_0 = 4.25 \times 10^{-3} \text{ cm} \\ \theta_0 = 3.478 \times 10^{-6} \text{ rad.} \end{array} \right.$$

Substituting above values to the eq. (7), we get

$$\frac{F_1(n)}{F_2(n)} = 1.558$$

Therefore we get following by graphically

$$n \approx 0.15$$

And then from the eq. (7)

$$K_{o,l} = 14.398 \times 10^6 \text{ kg/cm}$$

$$K_o = 6.260 \times 10^3 \text{ kg/cm}^2$$

$$k_o = K_o/650 = 9.63 \text{ kg/cm}^3$$

Therefore we get following values as Table 2 to the distribution of K-value and reaction of the ground.

IV. Conclusive Remarks

The relationship of the ground counterpressure caused by a lateral load to the well of a real bridge has been obtained through the two procedures: a static method and a dynamical method based on a vibration test. The results are illustrated in Fig 11-2 which shows that the static and dynamic K-values follow a considerably similar trend. However, it must be noted that there exists a substantial difference in the motion of the ground between the static and dynamic tests. In the dynamic test the measurements are made within the ranges of extremely small displacement and pressure, while the static test, where considerably large displacement and pressure are applied, is subject to creep plus consolidation, in the latter, influences of consolidation due to repeated loading are also involved. However, as the nature of dynamic loading during earthquake shocks belongs to the type of repeated loading, the results obtained through a repeated-loading test should preferably be made the basis in order to determine the K-value through a static test. Further, when the measurements are made swiftly and under conditions of small displacement and pressure, the results seem to approach gradually those obtained through a dynamic test. This is a problem to be dealt with by a future research. Also, there exists a considerable discrepancy between the apparent average K-value obtained through a vibration test and an individual K-value. This is a problem related to virtual mass of the adjoining ground due to vibration of the well as well as pressure distribution around the well. This obscurity must also be cleared in order to produce an accurate conclusion.

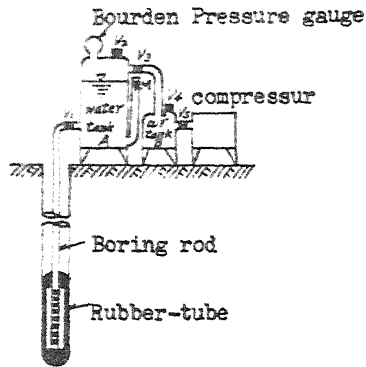


Fig. 1 Apparatus for determining the coefficient of soil lateral reaction (K-Value)

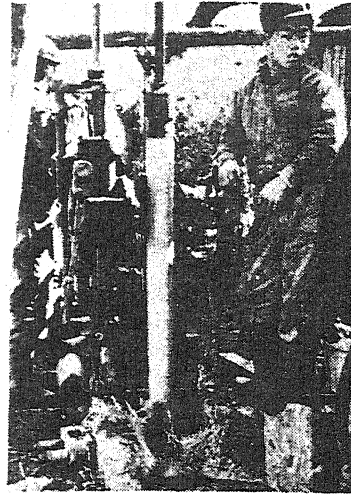


Fig.2-1 Photograph of Rubber-tube

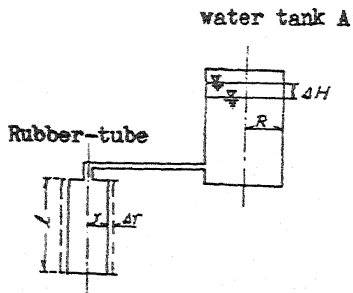


Fig. 3 Relationship between water-tank and Rubber-tube

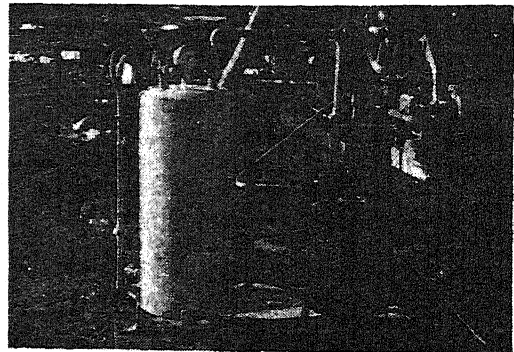


Fig.2-2 Photograph of tank A and B

Earthquake Proof Design of the Foundation of Bridge Pier

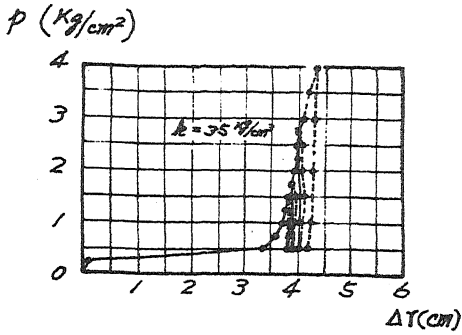


Fig. 4-1

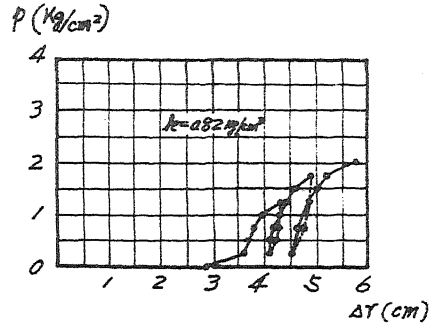


Fig. 4-2

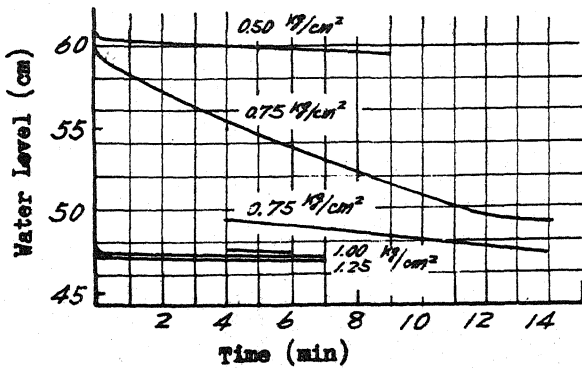
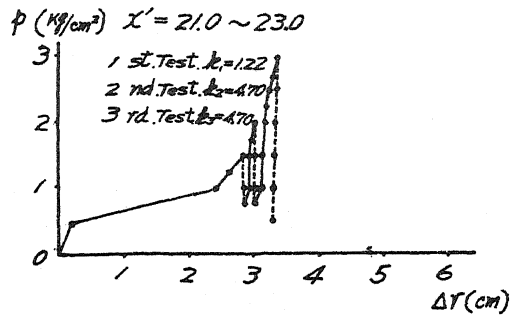
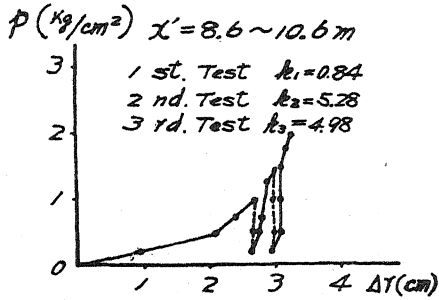
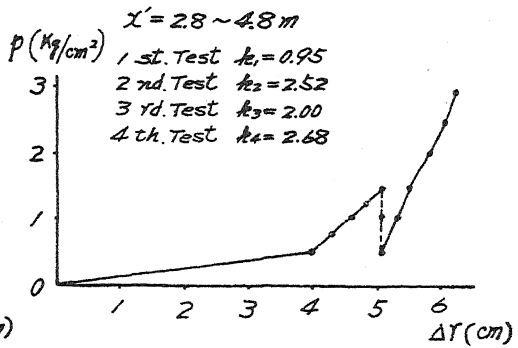
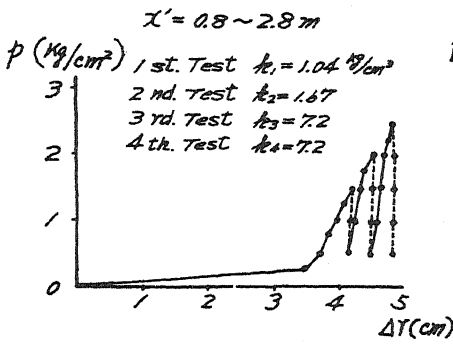


Fig. 5

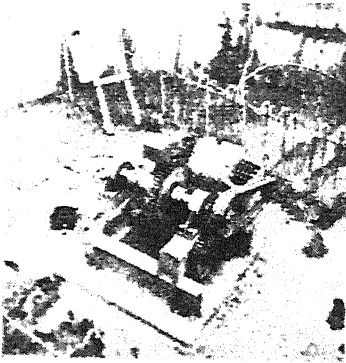


Fig. 6

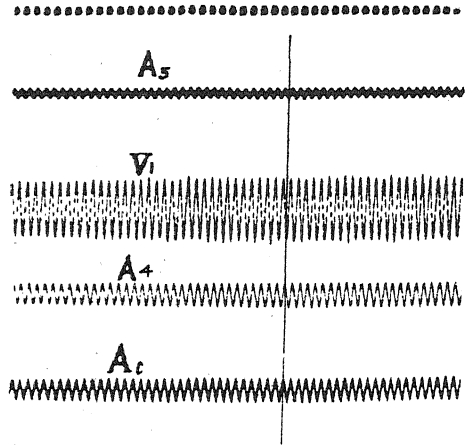


Fig. 8

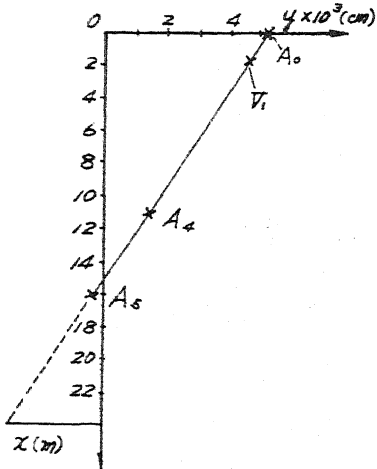


Fig. 7 Displacement Distribution of the Well in 10 cycles

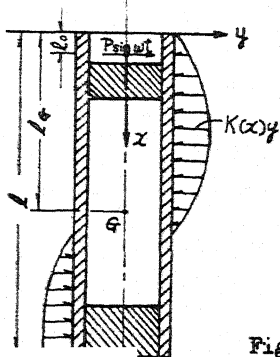


Fig. 9

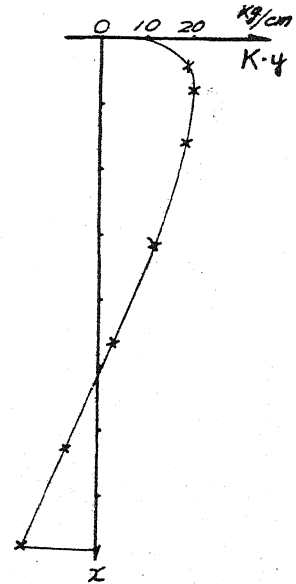
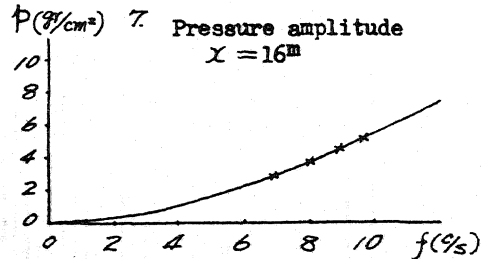
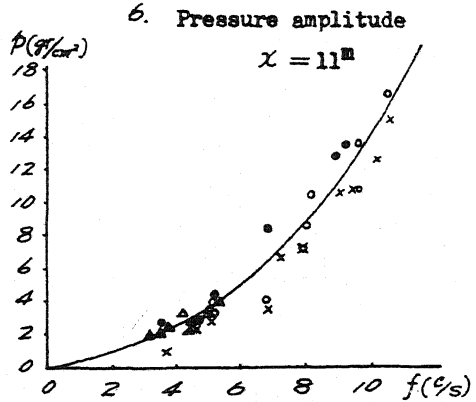
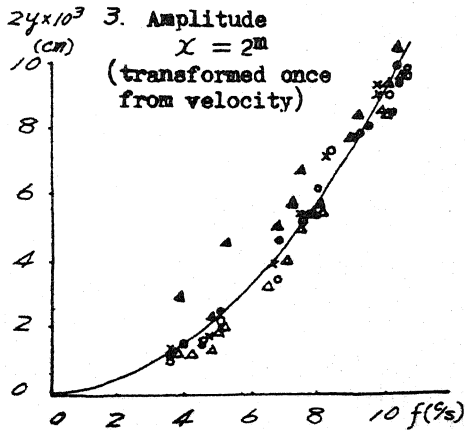
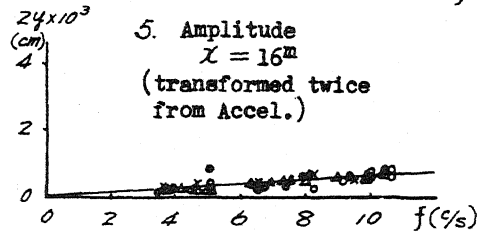
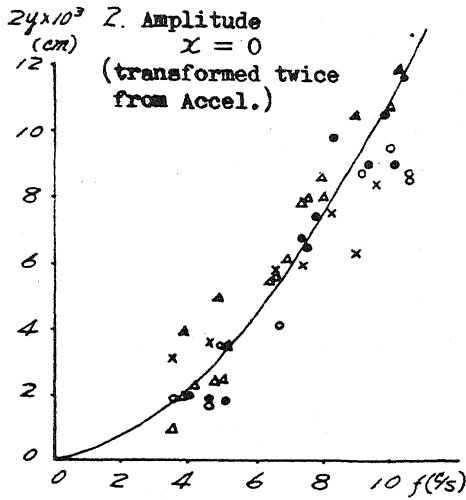
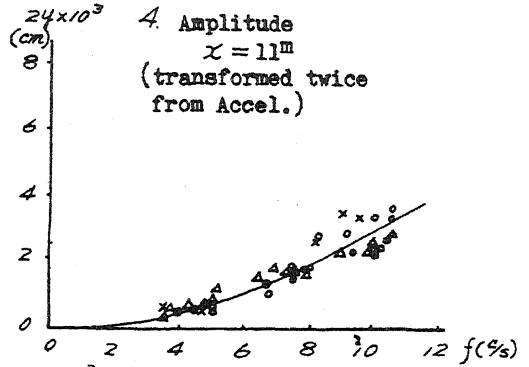
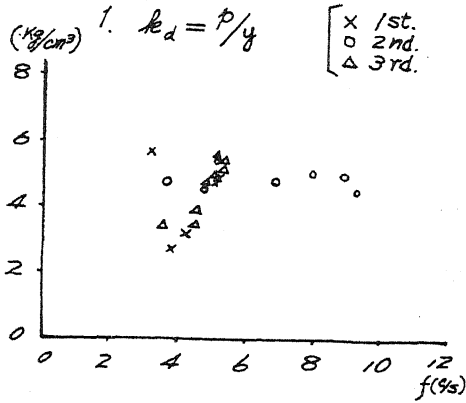


Fig. 12 Reaction Distribution for Well Foundation



- Fig.10 -

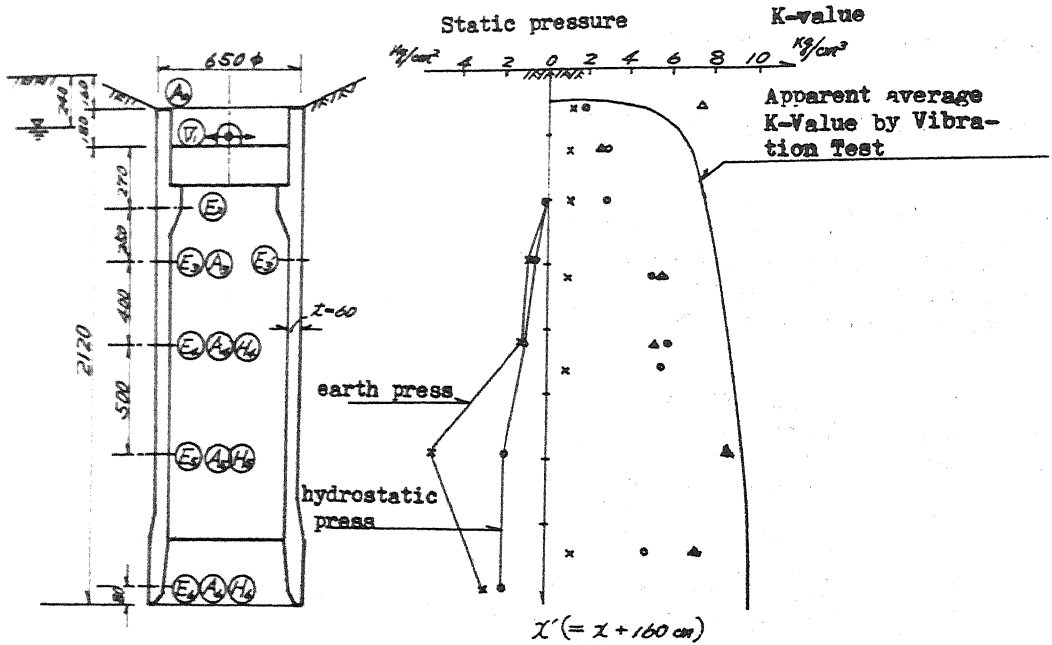


Fig. 11-1 Type and dimension of Tested Well and Setpoints of installed gages. (unit: cm)

Fig. 11-2 Static earth press, and Hydrostatic press. acted on the Well before the Vibration Test and K-Value Distribution.

- (E) earth press gage
- (H) pore press meter
- (A) Acceleration meter
- (V) Velocity meter

- $k_2 = p/y$ by simultaneously measuring
- ▲ $k_d = \bar{p} / \bar{y}$ mean in 10c/s
- x, o, Δ, K-Value by rubber tube method

Table 2

0	0.05	0.1	0.2	0.4	0.6	0.8	1.0
0	6.14	6.80	7.55	8.36	8.90	9.28	9.60
0	3994	4431	4917	5454	5798	6054	6260
5.25	4.84	4.44	3.64	2.04	0.44	-1.16	-2.76
0	19.3	19.7	17.9	11.1	2.5	-7.0	-17.3

Table 1. Summary of Soil Test Result.

Places	Shinogata Bridge			Nishiarai Bridge			Yoshida Bridge											
depth (m)	2-4	5.5 11 13	18	22	3.5	9	12	15.6	24	26	0.8	2.8	5.0	8.6	12.7	21	21	
soil	silt	clay loam	sandy loam	sandy loam	clay loam	clay loam	clay loam	clay loam	sandy loam	sandy loam	sandy loam	coarse sand	loam sand	loam sand	gravelly sand	sandy gravel	sandy gravel	sandy clay
the coefficient of soil lateral reaction (K-value)(kg/cm ²)	0.4	0.82	0.73	1.2	1.33	3.5	0.43	0.62	0.70	2.0	1.67	3.5	1.67	1.11	3.75	1.30	1.54	6.88
borehole radius r (cm)	4.5	5.6	4.4	5.9	4.3	5.3	5.5	4.0	6.3	6.0	3.2	4.6	5.5	6.0	4.8	4.1	5.8	4.6
standard penetration test N (blows/ft)	1.7	2.2	3.5	11.0	5.3	12.0	1.5	1.0	0.5	3.0	7.5	3.7	3	4	4	33	-	60
vane shear test c (kg/cm ²)	0.2	0.5	0.5	0.5	0.5	-	0.4	0.45	0.9	1.0	-	-	-	-	-	-	-	-
Swedish sounding test (round/m)	100	100	100	100	100	25	100	50	100	100	100	100	100	100	100	-	-	-
compression index C _c	0	40	100	145	145	200	0	0	0	20	70	100	-	-	-	-	-	-
unconfined compression strength q _u (kg/cm ²)	0.59	0.714	0.58	0.632	0.549	-	0.43	0.63	0.45	0.41	0.41	-	-	-	-	-	-	-
	0.57	0.95	0.70	0.76	-	1.26	0.80	0.50	0.86	1.00	-	-	-	-	-	-	-	-