EFFECTS OF VIBRO-COMPOSER AND PRELOADING ON THE ELASTIC MODULUS OF SOIL

by Dr. SHIZUO YAMAMOTO* Member of A.I.J.

TAKASHI SEKI**

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NOBUAKI SUZUKI**

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Introduction

It is necessary to know the elastic modulus of the soil for the dynamical design of structures considering the effects of the soil. If the elastic modulus of the soil can be made clear in relation to the physical property of the soil or the some coefficient representing the soil condition such as N -value, void ratio, strength of the soil etc., the dynamical design of structure considering soil will become more easily. In this report the elastic modulus of the soil is evaluated from the forced vibration test of foundations and the measurement of the shear wave velocity, and is studied in relation with the physical property of the soil and the some coefficient representing the soil condition. Also, the effects of the vibro-composer and preloading for elastic modulus of soil are studied.

1. Soil Condition

The locality and the soil conditions of test ground are shown in Fig. 1 and Fig. 2. The soil stabilization method was performed to strengthen the F and S stratum and to prevent differential settlement, i.e., sand-drain method was performed

Fig. 1 Locality of Kawasaki Factory Site Nihon Buthyl Co., Ltd.

Fig. 2 Boring Log

* Chiyoda Chemical Engineering & Construction Co., Ltd. Research Department Staff Engineer

** Research Engineer

Fig. 3 Settlement and Pore Pressure

before preloading and after preloading are compared in Fig. 2. As a result of soil stabilization method, N value was strengthened to 7 from 3 at F stratum and 20 from 7 at S stratum. The laboratory test results of soil are shown in Table 1.

2. Experimental Method

The vibration tests of model foundations and the measurement of S wave velocity were performed at the ground before and after preloading in order to know the contributions of sand-drains and preloading to increasing the elastic modulus of the soil. The foundations for forced vibration test are shown in Fig. 4-(a) and (b). The ground levels for each test are shown in Fig. 2.

The forced vibration test of foundation was performed, oscillating the foundations by the vibrator installed on it, varying the exciting frequency step by step, and the amplitudes at the several points of the foundation were measured by vibrometer. Then, resonance curves and vibrating modes of the foundation at resonance were obtained. The center of exciting force is located at 38 cm high from the surface of test foundation. The standard measuring points for two kinds of oscillating direction, i.e. vertical and horizontal direction to obtain the resonant curves are shown in Fig. 4-(a) and (b). In the case of horizontal excitation, the modes of sliding and rocking were coupled, since the center of

Table 1 Results of Soil Test

			F-Stratum Boring No. 1 5.00~5.80	S-Stratum Boring No. 4 10.00~10.60	CI-Stratum Boring No. 2 17.00~17.80
Before Preloading	w	%	45.9	39.0	57.5
	G		2.72	2.70	2.71
	r_t	g/cm ³	1.93	1.80	1.24
	r_d	g/cm ³	1.18	1.29	1.04
	e		1.29	0.90	1.60
	q_v	kg/cm ²	0.405~0.514	0.285~0.710	0.890~1.265
	c	kg/cm ²	0.00	0.45	0.00
	ϕ		23°00'	27°10'	24°30'
After Preloading			B-6' 6.00~6.30	B-6' 11.00~13.30	B-6' 14.00~15.30
	w	%	33.8	29.3	27.2
	G		2.72	2.69	2.70
	r_t	g/cm ³	1.89	1.82	2.03
	r_d	g/cm ³	1.41	1.33	1.60
	e		0.93	1.08	0.69
	q_u	kg/cm ²			
	c	kg/cm ²			

for F and S stratum with preloading. The time-load curve for preloading and time-settlement curves are shown in Fig. 3. The boring logs

Fig. 4-(a) Test Foundations and Arrangement of Measuring Points before Preloading

Fig. 4-(b) Test Foundations and Arrangement of Measuring Points after Preloading

exciting force did not coincide with the center of gravity of test foundations. "VERTICAL" and "HORIZONTAL" in the following figures and tables show the direction of excitation, respectively. This experiment was repeated several times, according to several magnitude of eccentric moment of the vibrator. The dynamical k -values and the critical damping ratio from these data were obtained.

The measurement of S -wave velocity was performed as follows, i.e., a wooden plate, 1 m long, 40 cm wide and 3 cm thick, was placed on the ground, contacting smoothly the plate and the ground surface, by cement powder scattered on the surface of ground, and then SH-waves were generated by hitting either end of the wooden plate horizontally which weighted by several persons using a wooden sledge hammer. The seismometers were arranged on the ground surface as shown in Fig. 4-(a) and (b). Then, the shear wave velocity was obtained as phase velocity from the time-lag-distance curve.

3. Experimental Results and Consideration

3-1 Resonance Curve and Subgrade Reaction

The resonance curves at the top of foundation obtained from forced vibration test of foundations are shown in Fig. 5 thru Fig. 10. The test results for the dynamical k -value and the critical damping ratio are shown in Table 2 and 3. From the resonance curves, it is observed that the resonant frequency becomes low according to increasing the amplitude. Fig. 11 shows the relationship between resonant frequency and resonant amplitude. Fig. 12 shows the relationship between dynamical k -value evaluated from coupled vibration of horizontal and rotation and resonant amplitude. The under linear characteristic of stress-strain is observed more clearly from these figures. In Fig. 11 and 12 the test

results before preloading and after preloading are plotted on the same figures. It is observed from these figures that after preloading the natural frequency and dynamical k -

Fig. 5 Resonance Curves

Fig. 6 Resonance Curves

Fig. 7 Resonance Curves

Fig. 8 Resonance Curves

Fig. 9 Resonance Curves

Fig. 10 Resonance Curves

Table 2 Test Results of Dynamic Coefficient of Subgrade Reaction and Critical Damping Ratio (Before Preloading)

Type of Concrete Block	Test No.	Excited Direction	Eccentric Moment (kg·cm)	Resonant Frequency		Dynamic Coefficient of Subgrade Reaction		Critical Damping Ratio h
				f (cps)	ν^2 (rad/sec) ²	k_h (kg/cm ³)	k_v (kg/cm ³)	
L Type	1	Horizontal	14	6.17	1,500	1.01	3.58	0.0605
	2		28	5.83	1,345	0.97	3.11	0.06
	3		56	5.42	1,160	0.68	2.82	0.070
	4		83	4.75	892	0.55	2.08	0.073
	5	Vertical	28	11.33	5,090		1.68	
	6		52	10.42	4,290		1.42	
	7		75					
M Type	8	Horizontal	7	8.75	3,030	1.23	4.93	0.069
	9		14	8.33	2,740	1.04	4.95	0.0697
	10		24	7.83	2,425	1.13	3.45	0.0705
	11		34	7.18	2,030	0.91	3.00	0.071
	12		48	6.67	1,755	0.34	2.44	0.081
	13	Vertical	24	11.00	4,780		1.23	
	14		48	10.83	4,650		1.19	
	15		88	10.42	4,290		1.10	
	16		122	9.58	3,630		0.93	
	17		170	9.58	3,630		0.93	
S Type	18	Horizontal	7	9.67	3,700	0.98	3.20	0.098
	19		12	9.50	3,570	0.89	3.43	0.0925
	20		18	8.75	3,030	0.80	2.66	0.087
	21		26	8.33	2,740	0.83	2.04	0.089
	22		34	7.83	2,425	0.64	2.13	0.0855
	23		40	7.50	2,230	0.56	2.14	0.088
	24		45	6.67	1,760	0.48	1.48	0.104
	26	Vertical	24	11.17	4,930			
	27		40	10.83	4,750			
	29		80	10.33	4,220			
	30		120	10.33	4,220			

Table 3 Test Results of Dynamic Coefficient of Subgrade Reaction and Critical Damping Ratio. (After Preloading)

Type of Concrete Block	Excited Direction	Test No.	Eccentric Moment (kg·cm)	Resonant Frequency		Dynamic Coefficient of Subgrade Reaction		Critical Damping Ratio h
				f (cps)	ν (rad/sec)	k_h (kg/cm ³)	k_v (kg/cm ³)	
L-Type	Horizontal	1	16	9.6	60.3	2.34	9.05	0.091
		2	43	8.75	55.0	2.16	6.93	0.092
		3	62	8.0	50.3	1.50	6.70	0.103
		4	103	7.1	44.6	1.16	5.36	0.109
		5	155	6.3	39.6	0.795	4.98	0.127
	Vertical	6	90	14.4	90.5		2.70	0.070
		7	145	14.0	88.0		2.55	0.070
M-Type	Vertical	8	70	14.3	90.0		2.06	0.097
		9	114	14.1	88.5		2.00	0.069
		10	145	14.1	88.5		2.00	0.060
	Horizontal	11	21	9.4	59.0	1.24	7.05	0.106
		12	44	8.7	54.6	1.15	5.15	0.081
		13	81	7.5	47.0	0.780	4.56	0.123
		14	118	6.6	41.4	0.55	4.50	0.151
S-Type	Horizontal	16	24	12.2	76.6	1.47	5.50	0.127
		17	40	11.0	69.0	1.21	4.44	0.116
		18	81	9.2	57.8	0.826	3.32	0.133
	Vertical	20	63	14.9	93.5		1.60	0.064
		21	92	14.6	91.6		1.54	0.067
		22	120	14.15	89.0		1.45	0.062

value increase considerably due to the effect of soil stabilization method.

3-2 Critical Damping Ratio

Fig. 13-(a) and (b) show the relationship between the critical damping ratio obtained from the

resonance curves and the resonant amplitude. It is observed from Fig. 13-(a) that the critical damping ratio h is function of the dimension of foundation, i.e., h is larger for thin foundation than for thick foundation while the base area is constant. Also, it is observed that according to increasing amplitude the critical damping ratio h increase.

There are two kinds of damping, the loss of energy through propagation of waves away from the immediate vicinity of the footing, and the internal energy loss within the soil due to hysteretic and viscous effect. The radiational damping seems to be superior to the internal damping, then the test results are plotted in theoretical curves¹⁾ for radiation damping as shown in Fig. 14. The experimental values are located at the middle of two theoretical curves, because these values are the data for the coupled vibration of horizontal and rotational vibration.

The critical damping ratio h increases when the amplitude increases.

The reason for this seems to be due to the hysteretic damping. In horizontal excitation, the critical damping ratio h increase clearly after preloading. However, in vertical excitation the opposite tendency is

Fig. 11 Relationship between Natural Frequency and Resonant Amplitude

Fig. 12 Relationship between k_v' , k_h' and Resonant Amplitude

Fig. 13 Relationship between Critical Damping Ratio and Resonant Amplitude

observed (see Fig. 13-(b)). The reason for this phenomenon is not clear.

3-3 Relationship Between Shear Wave Velocity and N -Value

The time-lag-distance curves getting from the hitting the wooden plate method are shown in Fig. 15-(a) and (b). The phase velocity increase to 120 m/sec from 85 m/sec due to the effect of preloading. Fig. 16 shows a relationship between shear wave velocity V_s and N -value proposed by Imai et al²⁾. The experimental data are plotted in the figure and show a good coincident with proposed relations-

Fig. 14 Relationship between Critical Damping Ratio h and Mass Ratio b

Fig. 15-(a) Time Lag-Distance Curves for Evaluating Shear Wave Velocity by Method of Hitting a Wooden Plate Horizontally

Fig. 15-(b) Time Lag-Distance Curve for Evaluating Shear wave velocity by method of hitting a Wooden Plate Horizontally

hip. Fig. 17 show the relationship between V_S and N -value proposed by Kanai³⁾ and our test results are plotted in it. It is observed from this figure that the tendency coincident appropriately, but our test values of V_S are a little larger than proposed values.

3-4 Relationship between Shear Wave Velocity and Void Ratio

Hardin and Richart¹⁾ presented the relationship between shear wave velocity V_S and void ratio e , for varying values of the confining pressure as shown in Fig. 18 by laboratory test. Our field test results are plotted in this figure too. From this result, the confining pressure is observed about 0.08~0.15 kg/cm², which is equivalent confining pressure at 1 m depth under the surface of ground, for the measured velocity on the surface of ground.

3-5 Evaluation of Elastic Modulus by Shear Wave Velocity V_S and Dynamical k -Value

The elastic modulus of the test ground shall be evaluated from test results for dynamical k -value and shear wave velocity V_S . The shear modulus G is evaluated from the test results of the dynamical k -value using the several formulae, which are the static k -value on elastic half media, shown in Table 4 and from the shear wave velocity using the following formula :

$$V_S = \sqrt{G/\rho}$$

where ρ is density of soil.

Fig. 19-(a) shows the theoretical relationship between the dynamical k_v' -value (K_0/Ia) for coupled vibration of sliding and rocking vibration and the shear modulus G calculated from the formulae shown in Table 4. Similarly the same relationship for vertical vibration (k_v) are shown in Fig. 19-(b). Substituting the test results for k_v' and k_v into these figure, the shear modulus G can be assumed. The dynamical k -value is approximately in inverse proportion to the acceleration of center of gravity of the foundation as shown in Fig. 20-(a) and (b). Then, the values at which the acceleration of center of gravity is equal to zero in Fig. 20 are used to evaluate the shear modulus from Fig. 19-(a) and (b).

The shear modulus G evaluated from the test results of the dynamical k -value and the shear wave velocity V_S are shown in Table 5. It is observed from Table 5 that the maximum values of shear modulus G evaluated from dynamical k -value in the case of assuming triangular pressure distribution in rotational mode coincide pretty well with the values evaluated from the shear wave velocity V_S . It is confirmed that after preloading the shear modulus of the ground increase to be twice larger than it at proto ground.

3-6 Study on Applicability of Elastic Theory

The simulation of experimental values by means of elastic theory is made to study on the applica-

Fig. 16 Relationship between Shear Wave Velocity and N -Value

Fig. 17 Relationship between Shear Wave Velocity and N -Value

Fig. 18 Shear Wave Velocity Versus Void Ratio and Confining Pressure, for Quartz Sand

MOTION	PRESSURE DIST.	SPRING CONSTNT
CIRCULAR BASE		
VERTICAL	UNIFORM	$K_v = \frac{\pi a \mu}{1-\nu} \text{ (kg/cm)}$
	RIGID BASE	$K_v = \frac{4 a \mu}{1-\nu} \text{ (kg/cm)}$
HORIZONTAL	UNIFORM	$K_h = \frac{2 \pi a \mu}{2-\nu} \text{ (kg/cm)}$
	RIGID BASE	$K_h = \frac{8 a \mu}{2-\nu} \text{ (kg/cm)}$
ROCKING	TRIANGULAR	$K_\theta = \frac{\pi \mu a^3}{2(1-\nu)} \text{ (kg-cm/rad)}$
	RIGID BASE	$K_\theta = \frac{8 \mu a^3}{3(1-\nu)} \text{ (kg-cm/rad)}$

Where a : radius of base contact surface of foundation
 μ : Lamé's constant
 ν : Poisson's ratio

Table 4 Spring Constants for Circular Base Resting on Elastic Half-Space

Fig. 19-(a) Relationship between k_v' and Shear Modulus G

Fig. 19-(b) Relationship between k_v and Shear Modulus G

Fig. 20-(a) Relationship between h_v' and Acceleration of Center of Gravity of Foundation

Fig. 20-(b) Relationship between k_v and Acceleration of Center of Gravity of Foundation

Fig. 21 Coefficients β_x , β_y and β_z for rectangular footings (after Whitman and Richart, 1967)

TYPE OF PRESSURE DISTRIBUTION	SHEAR MODULUS G (kg/cm²)			
	BEFORE PRELOADING BY k_v' or k_v	AFTER PRELOADING BY V_k	BEFORE PRELOADING BY k_v' or k_v	AFTER PRELOADING BY V_k
VERTICAL	UNIFORM	60~98	79~137	278
	RIGID BASE	46~76	62~108	
ROTATION	TRIANGULAR	86~147	151~242	
	RIGID BASE	50~86	88~142	

Table 5 Comparison of Shear Modulus G

Motion	Mass Ratio	Spring Constant	Critical Damping Ratio
Vertical	$B_z = \frac{1-\nu}{4} \cdot \frac{m}{\rho_1 t^3}$	$K_z = \frac{G}{1-\nu} \cdot \beta_z \sqrt{4cd}$	$h_z = \frac{0.425}{\sqrt{B_z}}$
Horizontal	$B_x = \frac{7-8\nu}{32(1-\nu)} \cdot \frac{m}{\rho_1 t^3}$	$K_x = 4(1+\nu)G\beta_x \sqrt{cd}$	$h_x = \frac{0.288}{\sqrt{B_x}}$
Rocking	$B_\theta = \frac{3(1-\nu)}{8} \cdot \frac{I_x}{\rho_1 t^3}$	$K_\theta = \frac{G}{1-\nu} \cdot \beta_\theta 8cd^2$	$h_\theta = \frac{0.15}{(1+B_\theta)\sqrt{B_\theta}}$

Note: values for β_x , β_y , and β_z are given in Fig. 21 for various values of d/c

Table 6 Mass Ratio, Spring Constant and Critical Damping Ratio for Rigid Rectangular Footing Resting on Elastic Half Space.

Fig. 22 Comparison of Experimental and Theoretical (Elastic) Results

Fig. 23 Comparison of Experimental and theoretical (Elastic)⁽⁴⁾⁽⁵⁾ Results

Fig. 25 Comparison of Experimental and Theoretical (Elastic)⁽⁴⁾⁽⁵⁾ Results

Fig. 24 Comparison of Experimental and theoretical (Elastic)⁽⁴⁾⁽⁵⁾ Results

Fig. 26 Comparison of Experimental and Theoretical (Elastic)⁽⁴⁾⁽⁵⁾ Results

bility of elastic theory assuming the ground as the homogeneous, isotropic, elastic semi infinite body and calculating the response of a footing resting on it. The convenient method by Lysmer^{(4),(5)} et al. is used, and which outline is as follows. The equivalent radius of a rectangular footing is determined by the following :

$$\text{For translation : } r_0 = \sqrt{\frac{4cd}{\pi}}$$

$$\text{For rocking : } r_0 = \sqrt[4]{\frac{16cd^3}{3\pi}}$$

$$\text{For torsion : } r_0 = \sqrt[4]{\frac{16cd(c^2+d^2)}{6\pi}}$$

S-TYPE, (VERTICAL)

Fig. 27 Comparison of Experimental and Theoretical (Elastic)⁴⁾⁵⁾ Results

inwhich

$2c$ = width of the footing (along axis of rotation for the case of rocking),

$2d$ = length of the footing (in the plane of rotation for rocking).

The spring constants and the critical damping ratio of ground are calculated from the formulae listed in Table 6.

The experimental values for minimum eccentric moment of two kinds of test, i.e., horizontal and vertical excitations, for each footing are compared with the theoretical values in Fig. 22 thru Fig. 27, where the shear moduli used in the calculation of elastic theory are selected as coinciding the theoretical resonant frequencies with the corresponding measured resonant frequencies.

It is observed clearly from these figures that the results of elastic theory give too large damping in comparison with experimental values. This reason may be due to the fact that the ground is considered to be elastic half-space in elastic theory, while the real ground is composed of layered medium.

Then, the damping may be decreased by the effects of reflected wave from the under solid stratum. Consequently, in the case of using elastic theory, it is important to examine the layered condition of ground carefully and determine the reasonable value of damping coefficient. Authors experiences show that it is unreasonable to take the value over 0.2 as the critical damping ratio. The experimental and corresponding theoretical values for the critical damping ratio are shown in Table 7 for reference. Then, the equivalent values of shear modulus G from shear wave velocity are shown in Table 8. It is observed that the theoretical values are smaller than the values derived from the shear wave velocity considerably. It may be the reason that the strain of the soil in the forced vibration test is larger than it in the hitting the plate method.

Also, it is observed that the larger values of shear modulus were obtained in the case of thick foundation than thin foundation. This reason may be explained by the difference of confining pressure, i.e. the confining pressure of thick foundation is larger than thin foundation.

Conclusion

It is desirable that the elastic modulus of soil shall be assumed from a laboratory test results in a future. For this purpose, further investigation based on the many laboratory and field test results should be necessary.

References

FOOTING TYPE		EXPERIMENT		ELASTIC THEORY		
		δ_s	$\delta_{c,21}$	δ_s	δ_c	δ_p
L	BEFORE	0.183	0.080 ~0.070	0.418	0.239	0.0406
	AFTER	0.070	0.082 ~0.127			
M	BEFORE	0.122 ~0.130	0.067 ~0.081	0.474	0.271	0.0756
	AFTER	0.080 ~0.078	0.081 ~0.181			
S	BEFORE	0.154 ~0.159	0.087 ~0.104	0.561	0.321	0.0425
	AFTER	0.082 ~0.087	0.118 ~0.133			

Table 7 Comparison of Critical Damping Ratio

Table 8 Equivalent Value of Shear Modulus G

FOUND	THEORETICAL VALUES								G Kg/cm ² BY V _s	
	HORIZONTAL				VERTICAL					
	BEFORE		AFTER		BEFORE		AFTER		BEFORE	AFTER
TYPE	E.M. kg-cm	G Kg/cm ²	E.M. Kg-cm	G Kg/cm ²	E.M. Kg-cm	G Kg/cm ²	E.M. Kg-cm	G Kg/cm ²		
L	14	50	16	115	28	37.5	90	69.7		
M	7	60	21	70	24	24.7	70	45.9	140	280
S	7	40	24	65	24	11.6	63	25.1		

Table 8 Equivalent Value of Shear Modulus G

- 1) R.V. Whitman and F.E. Richart, Jr. (1967)
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地盤安定工法 (Vibro Composer および Preloading) が地盤の弾性定数に与える影響

正会員 山 本 鎮 男* 正会員 関 孝**
正会員 鈴 木 延 明**

ま え が き

地盤の影響を考慮した構造物の動的設計を行なうためには、地盤の弾性定数を知る必要がある。もし地盤の弾性定数と、土の物理定数あるいは N 値、間隙比、圧縮強度のような地盤条件をあらわすある指数との関係が、明らかにすることができるならば、地盤を考慮した構造物の動的設計はより簡単になる。この論文では、地盤の弾性定数を基礎の強制振動実験と地盤のせん断波速度の測定結果から推定し、地盤条件を示す諸定数と関連づけて検討した。また、パイプロンコンポーザーとプレローディングが地盤の弾性定数におよぼす影響について検討した。

1. 地 盤 条 件

実験場所と試験地盤の土質条件を Fig. 1, 2 に示した。表層 (F 層) とそれにつづく砂層 (S 層) を強化し、不等沈下を防ぐために、プレローディングとサンドドレインによる土質安定工法が施工された。Fig. 3 に時間-載荷曲線と時間-沈下曲線を示した。プレローディング前後の土質柱状図は Fig. 2 に示すとおりである。

土質安定工法の結果 N 値は F 層で 3 から 7 に、S 層で 7 から 20 に増加した。

土の室内試験結果を Table. 1 に示した。

2. 実 験 方 法

サンドドレインとプレローディングが地盤の弾性定数に与える影響を知るために、プレローディング前と後にそれぞれ模型基礎の強制振動実験と S 波速度の測定を行なった。強制振動実験用の模型基礎は Fig. 4-(a),(b) に、各実験の地盤レベルは Fig. 2 に示すとおりである。模型基礎の強制振動実験では、基礎上に据付けられた起振機によって、振動数を段階的に変化させながら加振し、基礎の各点の振幅を振動計で測定し共振曲線と共振時の振動モードを得た。なお、加振位置は、基礎上面から 38 cm の高さであり、共振曲線を得るための観測点は Fig. 4-(a),(b) 中に・印で示した。加振は、上下、水平の二方向について行なった。

水平加振は、加振位置が基礎の重心と一致していないため、水平動と回転動の連成した振動となっている。以下の図および表中の VERTICAL, HORIZONTAL の表示はそれぞれ加振方向を示している。この実験は、起振機の偏心モーメントを変え、数回くり返した。これらの data から、地盤の動的 k 値と減衰常数を求めた。地盤のせん断波速度は次のようにして求められた。長さ 1 m, 幅 40 cm, 厚さ 3 cm の木製の板を、平坦にならし、接触をよくするためにセメント粉をまいた地表面におき、その上に摩擦力を増すための重しとして数人がのった。この板の端部を木製のかけやで水平にたたくことによってせん断波を発生させた。

こうした発生したせん断波を Fig. 4-(a),(b) に示したように地表面に配置された地震計により、同時記録をとり、走時曲線から位相速度を求めた。

3. 実験結果および考察

3-1 共振曲線および地盤の復元力

基礎上で測定した強制振動実験による共振曲線を Fig. 5~10 に示した。実験から求めた動的 k 値と減衰常数の一覧を Table 2 と 3 に示した。共振曲線からは振幅が増大するにつれて、共振振動数が低下するのが観察される。Fig. 11 に共振振動数と共振振幅の関係を Fig. 12 に水平ロッキング振動から求めた動的 k 値と共振振幅の関係を示した。これらの図から、地盤の復元力のアンダーリニヤー特性が明瞭にみられる。Fig. 11 と 12 にはプレローディング前後の結果が同一図上にプロットしてあるが、この図からプレローディング後においては、土質安定工法の効果によって、共振振動数と動的 k 値が増大しているのが観察される。

3-2 減 衰 常 数

Fig. 13-(a),(b) に共振曲線から求めた減衰常数と共振振幅の関係を示した。

Fig. 13-(a) によれば、減衰常数 k は基礎形状の関数となっている。すなわち、 k は基礎底面積が一定の場合、厚い基礎より、薄い基礎の方が大きい、また、この図から、 k は振幅の増大につれて増加しているのが観察される。一般に、減衰には二つの種類がある。基礎底面から地盤への波動伝播によるエネルギー損失としての地

* 千代田化工建設株式会社 技術総合研究所スタッフ研究員
工博, ** 同研究員
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下逸散減衰と、土の履歴特性と、粘性による内部減衰である。

地下逸散減衰の方が優勢であると考えられるので、実験結果を、地下逸散減衰の理論曲線¹⁾ に対しプロットしてみた (Fig. 14)。

実験値が horizontal と rotation の二つの理論曲線の間にあるのは、これらの data が水平 rocking 振動から求めたものだからである。減衰常数 k が振幅の増加につれて、増大しているのは、履歴減衰によるものである。

水平加振においては、 k はプレローディング後明らかに増加しているが、上下加振においては逆の傾向になっている。この理由は明らかではない。

3—3 セン断波速度と N 値の関係

板たたき法から求めた走時曲線を Fig. 15-(a),(b) に示す。プレローディングの効果により、位相速度は 85 m/sec から 120 m/sec に増加している。

Fig. 16 は今井²⁾ が提案したせん断波速度 V_s と N 値の関係であり、これに実験値をプロットしてみたが、よく一致している。

Fig. 17 は金井が提案したせん断波速度と N 値の関係に、実験値をプロットしたものである。傾向的にはほぼ一致しているが、今回の実験値の方が提案値よりも若干、高めにでているようである。

3—4 セン断波速度と間隙比の関係

Hardin と Richart¹⁾ は室内実験の結果から、種々の confining pressure に対してせん断波速度 V_s と間隙比 e の関係を Fig. 18 のように示した。

実験値をこれにプロットしたが、これによれば confining pressure は約 0.08~0.15 kg/cm² に相当し、これは地表面から、約 1 m の深さにおける confining pressure に相当する。

3—5 動的 k 値とせん断波速度による地盤の弾性定数の推定

実験地盤におけるせん断弾性係数 G は、Table 4 に示した半無限弾性体上における静的 k 値の理論式を用いることによって、動的 k 値の実験値から推定することにした。また、次式を用いることによって、せん断波速度の実験値から推定することができる。

$$V_s = \sqrt{G/\rho} \quad \rho: \text{土の密度}$$

Fig. 19-(a) は、Table 4 の各式から求めた、水平と回転の連成振動の場合の垂直方向動的 k 値 $k_v'(K_\theta/Ia)$ とせん断弾性係数 G の関係を示したものである。上下振動に対する同様の関係を Fig. 19-(b) に示した。これらの図に k_v' と k_v の実験値を代入すれば、せん断弾性係数 G が推定できる。動的 k 値は Fig. 20-(a),(b) に示したように、ほぼ基礎の重心加速度に反比例する。したがって、Fig. 19-(a),(b) からせん断弾性係数 G

を推定するにあたっては、便宜上 Fig. 20 において基礎の重心加速度が 0 の場合の値を用いることとする。

動的 k 値とせん断波速度の実験値から求めた、せん断弾性係数 G を Table 5 に示した。

Table 5 によれば、回転モードと、応力分布を三角形と仮定して、動的 k 値から求めたせん断弾性係数 G の最大値とせん断波速度から求めた G の値が非常によく一致していることがわかる。また、プレローディング後は、地盤のせん断弾性係数が 2 倍に増加したことが確認される。

3—6 弾性理論の適用性の検討

地盤を均質な等方性の半無限弾性体と仮定し、その地盤上の基礎の応答を解析する弾性理論によって、実験値のシミュレーションを行ない、弾性理論の適用性について検討した。弾性理論としては、Lysmer らによる簡便法を用いた。すなわち、

矩形基礎の等価半径は

$$\text{上下動, 水平動} \quad r_0 = \sqrt{\frac{4cd}{\pi}}$$

$$\text{ロッキング} \quad r_0 = \sqrt[4]{\frac{16cd^3}{3\pi}}$$

$$\text{ねじり} \quad r_0 = \sqrt[4]{\frac{16cd(c^2+d^2)}{6\pi}}$$

ここで

$2c$ = 基礎幅 (ロッキングの場合は、回転軸に平行)

$2d$ = 基礎長 (ロッキングの場合は、回転軸に直角方向)

バネ常数と減衰常数は Table 6 を用いる。

各試験基礎について、水平ロッキング振動、上下振動の実験からそれぞれ、偏心モーメント最小のものをえらび、弾性理論による共振振動数を実験値に等置して解いた共振曲線を実験値と対比して、Fig. 22~27 に示す。一見して明らかなように、弾性理論による解は、過大な減衰を与えていることがわかる。これは、弾性理論においては、地盤を半無限弾性体としているが、実際の地盤は成層をなしているため、下層の固い層からの反射波の影響により、理論値よりも低い値となるものと思われる。したがって、設計において、弾性理論を用いる際には、地盤の成層状態を考慮に入れて、適切な減衰常数を仮定することが重要である。筆者らの経験によれば、減衰常数としては 0.2 以上の値をとることは適切ではないと思われる。Table 7 に参考のために、減衰常数の実験値と理論値を対比して示した。

次に、この計算値から求めたせん断弾性係数の値を Table 8 に示した。この計算値は、板たたき実験によるせん断波速度から求めた値に比べてかなり小さい値となっている。これは、板たたきに比べて、歪が大きい

めと思われる。また、基礎高が大きいものほど G が大きな値をとっているのは、confining pressure の増加による影響であろう。

む す び

今後、地盤の弾性定数が、土の室内試験結果から推定できるようになることが望まれる。このためには、室内試験と野外実験の結果を種々の角度から比較、検討することが必要であろう。

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THEORETICAL STUDY ON OSCILLATORY TENDENCIES OF CYLINDRICAL STRUCTURES IN NATURAL WINDS (3)

--Analysis of Some Practical Phenomena--

by Dr. MORIHISA FUJIMOTO, Prof., Tokyo Inst. of Tech., TAKESHI
OHKUMA, Graduate Student, Tokyo Inst. of Tech., Members of A.I.J.

This paper checks the validity of the theory shown in the former paper.

Two examples are calculated. One is the aerodynamic behavior of the model of 1-mass 2-freedom system stood vertically in the open*. The other is the case of the steel stack of about 90 m high. The former provides effective data on the beat phenomenon and the latter is an example of aeroelastic behaviors which have never yet sufficiently evaluated by analytical approaches.

The calculated results agree with the measured. This shows the validity of the approach.

* The cylinder is 0.37 m in diameter and 1.88 m in length.

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EXPERIMENTAL STUDY OF BRACING BEAM-COLUMNS

by Dr. TOSHIRO SUZUKI, Assistant Prof. of Tokyo Institute of Tech., and
TETSURO ONO, Assistant of Tokyo Institute of Tech., Members of A.I.J.

The purpose of this paper is to investigate problems of the buckling strength and bracing spaces for beam-column by experimental analysis. This experiments include two experimental series A and B.

The object of A-series is to investigate the lateral buckling strength of beam-column. In this experiment, new spherical block is used. Experimental values of buckling strength are fairly good approximation to theoretical values and the behavior of deformation is good.

From experimental results, we find that the increase of post buckling strength is a little. Then, if we take notice of the strength of the members, we may take the post buckling strength into consideration at design the members. However, we can not expect the deformation capacity of the nonbraced beam-column.

B-series is made experiment to explain the bracing space of beam-column from point of view of the plastic steel design. The theoretical ground of bracing space is the theory of M.G.Lay. In generally, there are two fundamental optimum condition to consider the bracing space of beam-column.

- 1) The attainment of the maximum in-plane moment capacity.
- 2) The attainment of rotation at which local buckling occurs in member.

From these experimental results, we can not expect the rotation capacity after maximum in-plane moment capacity. In particular, this tendency is remarkable in the beam-column subjected two end moments. Therefore, deciding the bracing space of beam-column, it is appropriate to adopt the first optimum condition.

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EFFECTS OF VIBRO-COMPOSER AND PRELOADING ON THE ELASTIC MODULUS OF SOIL

(See Page 41)

by Dr. SHIZUO YAMAMOTO, TAKASHI SEKI, NOBUAKI SUZUKI and SHOICHIKI
MATSUNAGA, Chiodo Construction Ltd. Co. Members of A.I.J.