International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

1991 - Second International Conference on Recent Advances in Geotechnical Earthquake Engineering & Soil Dynamics

13 Mar 1991, 1:30 pm - 3:30 pm

Dynamic Characteristics of Soils in Calculation of Vibrations of Foundations for Machines with Periodical, Pulse and Random Loads

Vadim M. Platesky
Leningradsky Promatroyproekt, Leningrad, USSR

Recommended Citation

Platesky, Vadim M., "Dynamic Characteristics of Soils in Calculation of Vibrations of Foundations for Machines with Periodical, Pulse and Random Loads" (1991). *International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*. 9.
https://scholarsmine.mst.edu/icrageesd/02icrageesd/session11/9

Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, March 11-15, 1991 St. Louis, Missouri, Paper No. 11.27

Dynamic Characteristics of Soils in Calculation of Vibrations of Foundations for Machines with Periodical, Pulse and Random Loads

Vadim M. Platesky
Cand.Techn.Scs., Head of Department for Dynamics, Leningradsky Promstroyproekt, Leningrad, USSR

SYNOPSIS: The paper deals with the results of the experimental studies performed by the author on the foundations 0.5 , 2.4 and 25 sq.m in area under periodical, pulse and random dynamic loads. Some formulas are given to determine dynamic characteristics of soils. Methods and devices for field dynamic tests of soils are described. The results of the proposed relationship check are given.

At present there exists a number of soil base models used for dynamic calculation of foundations. Convevtionally, they can be divided into the following groups: models based on the hypothesis of local elastic soil base deformation, models based on the hypothesis of general elastic deformations and a model with two other elastic characteristics. There is no need to describe advantages and disadvantages of the familiar models but I would like to point out that in a number of countries designers use a model drawn on the analogy between the system of a spring and a damper connected in parallel and elastic inertia half-space. It is known that the properly chosen parameters of the model result in better presentation of vibrations of a foundation resting on elastic inertia soil base.

If neglecting the soil base stratification, the analog model with the influence of periodical loads can be presented with the added mass of soil or without it. To choose the analog model, we compare the results of the calculation by the model with two parameters (rigidity, damping) and three parameters (rigidity, damping, inertia mass) with the results of the experimental data on foundations 0.5, 2.0, 4.0 and 25 sq.m in area. The first foundadion, 2 x 2 m, was of cast in-situ concrete having a form of a solid slab 400 mm thick. A frame of surfaced lumber was placed in the foundation prior to concreting, which permitted to cut off the parts of the foundation so as to get a smaller foundation, i.e. 1.5 x 1.5 m in area. The second foundation, 5 x 5 m, was concreted at the same level as the first one. The general view of the foundation is presented in Fig. 1.

Prior to concreting the foundation 5 x 5 m , the third foundation 0.5 x 0.5 sq.m was tested at the same place. The technical characteristics of the experimental foundations, base soil and vibration machine placed on the foundations are presented in Tables 1, 2 and 3. Vibration parameters were measured by standard instruments which were also used for measuring vertical amplitudes of vibrations and the phase shift between displacement of the foundation and the forces applied to it.

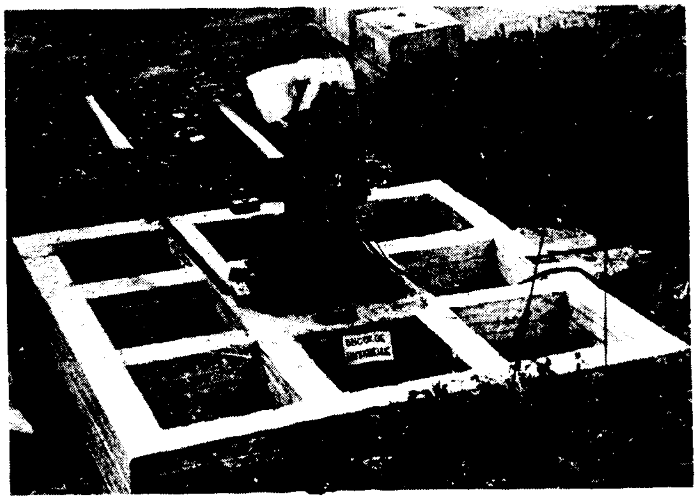

Fig. 1 General view of experimental foundation 25 sq.m in area.

Table 1.

Experimental foundation	Foundation dimensions			Weight of foundation and vibromachine, N	Static pressure kPa
	in plan m	area sq.m	height m		
F-0.5	0.7x0.7	0.5	0.4	7.5	15.0
F-2.25	1.5x1.5	2.25	0.4	25.0	11.0
F-4	2.0x2.0	4.0	0.4	42.5	10.5
F-25	5.0x5.0	25.0	0.8	415.0	16.5

Table 2.

Soil type	Layer thickness m	Soil density t/m^3	Porosity factor	Poisson's ratio	Deformation modulus kPa	Modulus of elasticity * kPa
Hard loam	5.3	1.95	0.65	0.37	13000	80000

* From vibrostabilometer tests

Table 3.

Name on vibromachine	Electric motor power kW	Range of frequency of shaft rotation with disbalancing members, Hz	Moment of vibrator disbalancing members Nm	Vibrator weight kN
ВШ	0.5	0-60.0	0.73-1.34	2.5
ВНД-1	32.0	0-25.0	200.0	40.0

Fig.2 shows resonance curves of the experimental foundation vertical vibrations from which it follows that the calculated curves drawn with regard to the added mass of soil (model with three parameters) are close to the experimental ones while the calculated curves without the added mass of soil greatly differ from the experimental curves.

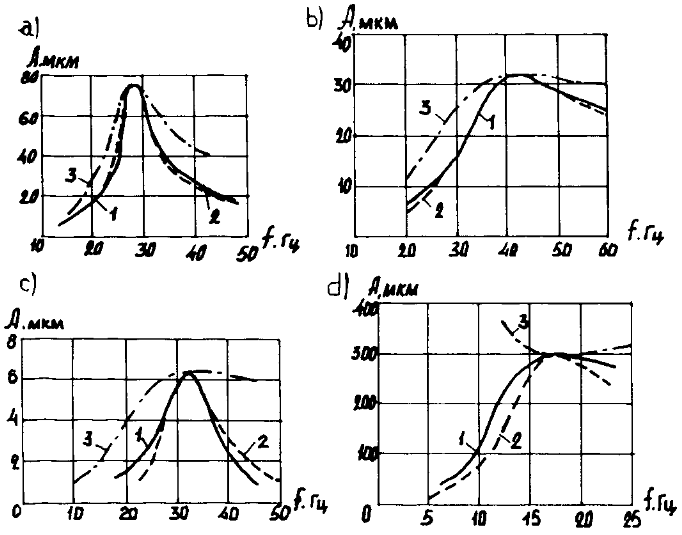

Fig.2. Resonance curves for experimental foundations of different areas:

a) F=0.5 sq.m
b) F=2.0 sq.m
c) F=4.0 sq.m
d) F=25.0 sq.m
1 - experimental curve
2 - calculated curve with added mass of soil
3 - calculated curve without added mass of soil

The model with the added mass of soil was studied by many specialists. The most popular, described in the literature, is the method for determination of the added mass of soil based on the processing of the experimental data results. There are different ways to use this method. To my opinion the most advantageous result was obtained by M.N.Golubtsova [1] who developed a method for determination of the added mass of soil by the width of the resonance curve. The processing of a great number of the experimental resonance curves performed by M.N. Golubtsova showed that a minimum deviation of the calculated resonance curve from the experimental one could be achieved with β taken within 1.5 - 1.9 (the greater value corresponds to clay, the less value corresponds to sand).I would like to point out for the sake of comparison that by calculation O.J. Shehter [2] obtained β within 1.06 - 1.64 and in the studies performed by V.A. Iljichev and V.G.Taranova [3] β changes within 1.59 - 1.98, while M. Novak [4] β determined as equal to 1.62.

As in the three-parametric model all parameters are in close relation and those properties of soil which are ignored in determination of one parameter can be presented in calculation of the other,I propose to take the added mass of soil o as equal to 1.7 irrespective of soil type. The rigidity factor in this case can be determined from Equation:

$$K_z = \beta \quad C_z \quad F \qquad (1)$$

where C_z - elastic uniform compression factor,
F - foundation footing area.

The model of the elastic half-space shows that the elastic uniform compression is in linear relation with the modulus of soil elasticity. However,in practice the static modulus of soil deformation is usually determined when studying soils in the field or laboratory. As deretmined fromthe experiments described in paper [5], there is a direct relationship between the modulus of elasticity (E_y) and modulus of deformation (E) which takes the following form:

$$E_y \approx 8E \qquad (2)$$

The soil in the elastic half-space features both the modulus of elasticity and Poisson's ratio which changes depending on the soil type. Let us follow the influence of the soil type on the magnitude of the elastic uniform compression in the analog model when changing the modulus of elasticity for the modulus of deformation. We denote relationship of C_z to the type of soil as factor and determine the factor by means of a loading plate which is described below. The experiments were performed on ten sites.The weight of the loading plate was 20 kN and the value of eccentricity of dynamic loading was 0.54 - 0.68 Nm for all the experiments. The experiments described in the paper were conducte d by B.K.Alexandrov, V.G. Taranov and V.M. Piatetsky. As we can see from Table 4, the factor is constant for sand, equal to 1, and very close to 1.5 for hard loam (irrespective of experiment 8). The same result was obtained from the analysis of the experiments performed by other scientists [6].

Table 4.

Srl. No	Name of area	Soil type	Modulus of deformation $E \cdot 10^3$ kPa	$C_z \cdot 10^4$ kN/cu.m	b_0
1.	Kremenchug Dneprovsky GOK	fine sand	28.0	17.1	1.1
2.	Katskany MSSR Festing ground	fine sand	24.0	12.4	0.95
3.	Kishinev Building for kolhoz Soviet	fine sand water-saturated	15.0	8.63	1.05
4.	Donetsk Experimental base of DPSNIIP	hard loam	13.0	11.6	1.63
5.	Beltsy, MSSR Sanytary Epidemic Station	loam	12.0	9.7	1.48
6.	Kolpino Izhovsky plant	hard plastic soil	17.0	12.1	1.3
7.	Soroky , MSSR Electropribor plant	semihard loam	13.0	11.2	1.57
8.	Kishinjev Dwelling house	semihard loam	20.0	8.00	0.74
9.	Kutuzov, MSSR NPO building	sandyloam	15.0	8.80	1.07
10.	Tchasov Yar	hard loam	17.0	14.5	1.5

Now, let us consider the relationship of the elastis uniform compression factor to the foundation footing area.By analogy with the model of elastic half-space, the rigidity of the equivalent model should change in proportion to the square root of the footing area .

Beginning from the forties D.D.Barkan and some other scientists noted no agreement between the elastic uniform compression factor and the mentioned relationship.

During the years which followed there were many proposals made for determination of factor C .Fig. 3 presents the results of a great number of experiments on the foundations of different sizes processed according to the same method .It also shows four curves. Three of them are drawn by the familiar methods: models of elastic half-space (A), Vinkler-Foiht model (B) and Filonenko-Borodich method (O.A. Savinov model) (C). The curve D is formed according to the experimental data proceeding from the condition that the rootmean-square deviation of the approximating function from the experimental values comes to a minimum.This approximating function can be written as

$$C_z / E = 0.5 + \sqrt{F_{15} / F} \qquad (3)$$

where F - foundation footing area, m
F_{15} - 15 sq.m

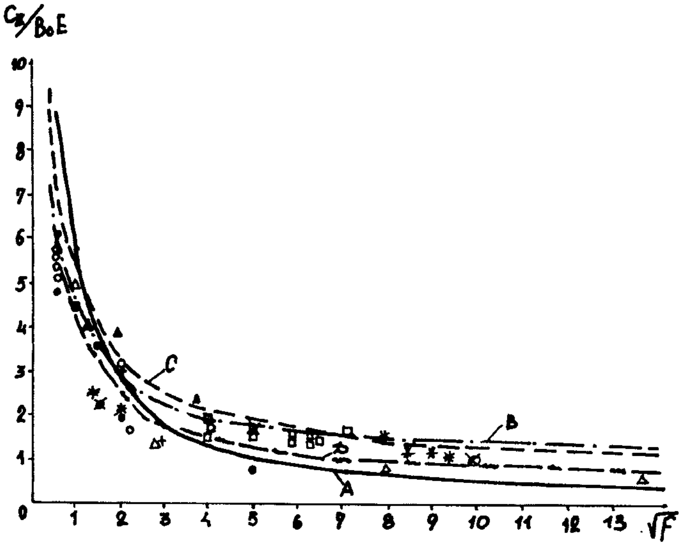

Fig.3. Relationship $C_z / b_0 E$ to foundation footing by different methods:

A - half-space
B - Vinkler-Foiht's method
C - O.A.Savinov's method
D - proposed curve

In order to check the proposed relationship, we tested two foundations of much different sizes.A foundation 0.5 sq.m in area was tested on the site for construction of a foundation for press. The characteristics of the foundation and the vibromachine are given above. Then at the same place there was built and tested a foundation 384 sq.m in area. The characteristic of soil under the foundation footing and the vibromachine for testing the foundaption are given in Table 5. The results of the experimental determination of C_z for foundationds 0.5 m^2 and 384 m^2 in area and calculation by formula (3) as well as the comparison with the mentioned methods are given in Table 6.

Table 5.

Type of soil under footing	Soil density t/m^3	Deformation modulus $E.10^5$ kPa	Foundation dimensions		Foundation weight kN	Pressure under footing kPa	Moment of vibration disbalansing members
			in plan m	height m			
Hard plastic loam	2.0	17.0	16x24	4.8	45000	117.0	147.0

Table 6.

Name of foundation	Experimental value $C. 10^4$ kN/cu.m	Calculated vlues of $C_z . 10^4$, kN/cu.m			
		Winkler Foiht's model	O.A. Savinov's method	Elastikc half-space	By proposed formula (4)
F-0.5	12.10	9.12	20.9	25.03	15.2
F-384	0.95	2.50	5.82	0.90	1.7

Cosequently, the equation for determination of the elastic uniform compression factor may have the following form

$$C_z = \beta_0 \, E \, (0.5 + \sqrt{F_{15} / F}) \qquad (4)$$

where β_0 - factor M equal to 1 for sand and 1.5 for loam and clay.
E - modulus of soil deformation under the foundation footing, kPa
F, F_{15} - see above.

Prior to considering the third parameter of the model, I would like to point out that some of the scientists paid attention to the relationship of elastic uniform compression factor to to the value of static pressure. To check this, four foundations were tested, two of an area of 0.5 sq.m each at different sites and two 4 sq.m and 5sq.m in area at the same site.

The static load was applied via spring vibroisolators in a manner that the vibrating foundation mass for all the experiments on one foundation remained constant. The pressures under the foundation footing were 20.0, 30.0, 40.0, 60.0 and 70.0 kN/sq.m. The results of the studies showed that the value of the elastic uniform compression factor changed close to the relationship proposed by O.A. Savinov [7], i.e. $C_z = \sqrt{P_0 / P}$ where P_0 is a pressure of 20 kN/sq.m; P - actual pressure under the footing not exceeding 40.0 kN/sq.m. The further experiments showed no difference in the values of C_z with P=40, 60 and 70 kN/sq.m. These findings coincide with O.Y. Shehter's [2] theoretical approach for the loading plate placed on the elastic half-space where C_z is asymptotic to the constant value. This approach comes about more for larger foundations. S.K. Lapin [8] came to the same conclusion, having studied a great number of foundations from 0.5 to 37.00 sq.m in area. Therefore, the static pressure influence should be only considered in making experimental studies. For the real foundations with a pressure greater than 40.0 kPa, the influence of this factor on the value of C_z can be neglected.

To determine the factor of relative damping according to O.Y. Shehter [7] using the Poisson's factor ν=0.33 we have

$$\xi_z = \frac{1}{\sqrt{\beta}} \quad , \quad \beta = \frac{m}{\rho \, r_0^{\,3}} \qquad (5)$$

where m, r_0 - foundation footing mass and radius, respectively. ρ - soil density under the foundation footing.

$\xi_z - \beta$ relatiship in one or another form can be found in the works of some American scientists and Chinese researchers (eg. D. Shi-Vei [9]). To check the relatiship of ξ_z to β, some experiments were taken in which the foundation natural frequences were a bit different and were in the range of 15 - 20 Hz. The choice of foundations close in frequency was due to the fact that in literature we can find some statements on the relationship of ξ_z to the vibration frequency (see [10]). Comparison of the relationship (5) with the results of the experiments shown in Fig. 4 permits us to reduce damping at the cost of introduction of a correction factor of 0.35. Thus, the third parameter of t the model can be determined from

$$\xi_z = \frac{0.35}{\sqrt{\beta}} \qquad (6)$$

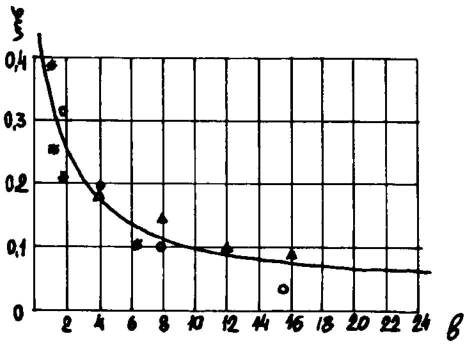

Fig.4. Relationship of relative damping factor to factor

Up to now, we have been considering the foundation behaviour under harmonic loading. Let us take some other dynamic loading. Particularly, the weak point in research is behaviour of foundation under random loading produced by ore grinding machines, barking drums, drying and mixing drums and the like. Not dwelling on the methods for load determination which are described in papers [11, 12], it should be noted that foundation vibrations produced by the above-mentioned equipment is a stationary random process and the inlet spectral density can be assumed as "white noise". Having studied the vibrations of the loading plate on the elastic inertia half-space under the influence of a random load of the "white noise type", we found it possible to model vibrations as a system with one degree of freedom and three parameters mentioned above.

In order to determine the parameters of the analog mechanical system and compare them with the determined ones with harmonic vibrations, fairly elaborate experiments were made.

An ore grinding drum mill with the drum 900mm in diameter, 2.5 kN in weight, grinding rods 2.3 kN in weight and rotation speed 42 r.p.m was installed on rubber elements on the floor resting on foundations 4 and 25 sq.m in area (Fig.5). It should be noted that the mill bearing frame was installed in such a way that only the vertical actions were trasmitted to the bearing structures. Besides, the rigidity of the rubber was designed so that the natural frequency of the plant in the shop coincided with the natural frequency of the foundations laid on the ground.

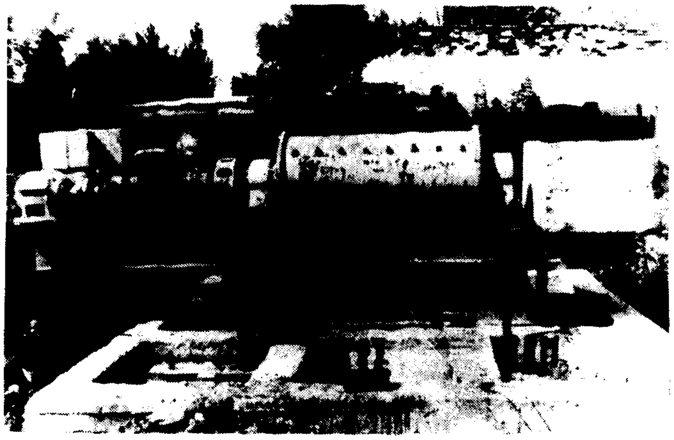

Fig.5. General view of the experimental foundation with a drum mill.

Thus, dynamic loading (spectral density) was determined from the laboratory-scale experiment and the relative damping factor was determined from the experiments made on foundations. The value ξ_z of determined for foundations 4 sq.m and 25 sq.m in area is given in Table 7. The values of the relative damping factor obtained from the experiments with the periodical loading as well as calculated from formula (6) are also given in this table.

Table 7.

Bearing structure	Main frequency on the curve of spectral density Hz	Mean-square-root deviation mm	Experimental factor of relative damping		Inlet spectral density, 10^{-4} kN.C	Weight of unit kN	Calculatid relative damping factor (6)
			periodical load	random load			
Rubber bearers in the shop	9.5	0.078	0.110	-	1.28	11.7	-
Foudat—on 4 sq.m in area	9.6	0.038	0.100	0.12	1.28	17.3	0.14
Foundation 25sq.m in area	10.8	0.012	0.320	0.31	1.28	51.7	0.32

The results of the experiment show that the relative damping factor at random loading which features the narrow strip random "white noise" process, is very close to the values obtained from the experiments with foundations under periodical loading. Therefore, a foundation for machine with random dynamic loading can be calculated by a three-parameter model where rigidity and damping factors can be determined the same as with periodical vibrations.

The problem of non-stationary vibrations of the loading plate on the inertia half-space and its relation to the adequate mechanical model was studied in detail by V.A. Iljichev [13]. On the basis of the solution which can be found in the mentioned work, V.A. Iljichev came to the conclusion that the loading plate on the half-space as to pulse response can be substituted by a system of 0.5 degrees of freedom consisting of a spring and a damper connected in parallel. The difference of this model from that described above is at first in the absence of the added mass of soil. The rigidity factor of the new system should be determined from Equation (1) with β=1 and for determination of damping we refer to the results of the experiments. Table 8 presents the results of vibrations of four hammer foundations and also the results obtained in calculations of the relative damping value by formula (5)

Table 8.

Name of enterprise	Pulse value kN.c	Foundation weight kN	Footing area sq.m	Natural frequency Hz	Pressure under footing kPa	Changed amplitude of vibrations mm	Factor ξ_z	
							Experimental	Calculated value (5)
Plant in Kaliningrad	28.3	4400	49.5	12.3	89.0	0.36	0.78	0.56
Plant in Kirovograd	8.0	1350	19.3	18.8	70.0	0.32	0.34	0.45
Mechanical repair plant	1.1	225	5.35	21.5	42.0	0.20	0.49	0.45
Turbine making plant in Kcharkov	28.3	5570	43.2	14.0	129.0	0.30	0.56	0.44

This comparison gives good convergence of the results for practical purposes. The experimental verification of the proposed relationship was performed with foundations 4.0 sq.m and 25 sq.m in area subjected to a falling load of 3.0 kN. To prevent rebound giving the second impact, a sand layer 5 cm thick was provided on the foundation. The heights of falling on the experimental foundations 4 sq.m and 25 sq.m in area were 0.1 m, 0.2 m, 0.3 m and 1.85 m, 1.95 m, 3.3 m, respectively. Damping characteristics were determined from the first amplitude of free vibrations. It should be noted that the linear character between the load and displacement was preserved in the experiments. The results of comparison of the values calculated by formula (5) and the experimental ones are given in Table 9. As we can see from Table 9 the use of formula (8) gives the results fully suitable for practical use irrespective of some difference between theoretical and experinental values.

Table 9.

Foundation area sq.m	Height of load falling m	Pressure under footing kPa	Frequency of natural vibrations Hz	Factor ξ_z	
				Experimental	Calculated value by formula (6)
4.0	0.3	15.0	21.0	0.51	0.68
25.0	3.7	20.5	12.5	1.13	0.92

Even though the convergence of the design and experimental values is good, the significant information can only be obtained from the results of field experiments. Different designs of loading plates can be used for making experiments with soils. However, the most applicable and widely used in the SU is a vibrating loading plate of our design [14]. The main parts of the l loading plate shown in Fig.6 are a bearing plate, a vibrator and ballasting plates.

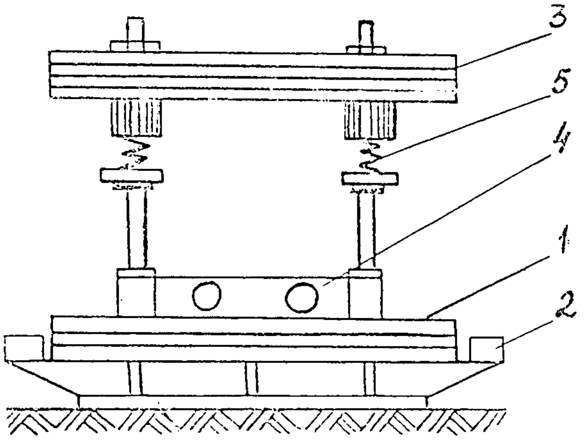

Fig.6. Vibrating loading plate for determination of dynamic characteristics of soils.
1 -loading plate
2 -measuring instruments
3 -ballasting plates
4 -vibromachine
5 -springs

By manipulation of the ballasting plates and the appropriate use of the disbalancig members of the vibrator, independence in changing of static and dynamic load parameters is achieved which are dynamic load amplitudes, medium static pressure in the base of the loading plate and its vibrating mass for different types of soil base deformation. The area of the base, 0.71 x 0.71, and the knock-down design permitted to work with the device on the site without using lifting mechanisms. The vibrator was rigidly fixed on the rectangular plate. Springs with a number of loading plates were placed on the vibrator. The plates can be placed under the springs and the vibrator. The complete standard loading plate device can provide for a change of vibration frequency from 7 to 80 Hz, disbalancers moment from 0.12 to 0.67 Nm and medium static pressure in the base from 5.0 to 40. 0 kPa.

Therefore, the loading plate device permits to change static pressure in the loading plate base at a permanent natural frequency of the system or to change the natural frequency at a permanent static pressure.

The work with the loading plate device resulted in gaining considerable experience which was described in the Manual [15] to the valid in the SU Specifications for designig foundations for machines with dynamic loads. Now I am going to dwell on forcasting of vibrations for large foundatons based on the results of the loading plate tests.

Prior to construction of foundations 25 sq.m and 384 sq.m in area, some loading plate tests were performed. The curves of frequency amplitude relatinship were drawn from the test results and the formula proposed by M.N. Golubtsova [1] was used to determine the soil mass involved in vibration along with the loading plate.

$$M_\omega = \frac{m_o \varepsilon}{a_i} \sqrt{\frac{1 - \left(\frac{a_j}{a_p}\right)^2 m^2}{1 - m^2}} \qquad (7)$$

$$m = 1 / 2 \left(1 + \sqrt{1 + \frac{2 t^2}{\omega_p^2}} \right.$$

where
$m_o\varepsilon$ - vibrator eccentric disbalancers moment
a_i - amplitude of foundation vibration with frequency
a_p - the greatest amplitude on the resonance curve
ω - frequency corresponding to the greatest amplitude
t - width of the curve with amplitude
ω_{zi} ω_{ii} - frequencies on the curve with amplitude a

Three or four values of M_ω with different heights of the curve and their average value was taken to obtain more accurate results.

The uniform compression factor C_z and relative damping ξ_z for the steady-state vibrations was found from the following Equation

$$C_{z\omega} = \frac{M_\omega \lambda_{z\omega}^2}{A_\omega} \qquad (8)$$

$$\xi_{z\omega} = \frac{m_o \varepsilon}{2 M_\omega a_{z\omega}}$$

where $\lambda_{z\omega}, a_{z\omega}$ - frequency and amplitude of loading plate resonance vibrations, respectively
M_ω, A_ω - loading plate mass with added mass of soil and loading plate base area, respectively

The change-over from the dynamic properties of the loading plate to the real foundations was performed according to the above-mentioned formulars, where $\beta_{z\omega} = \beta_z$ and C_z and ξ_z for the foundations were determined from the formulas

$$C_z = C_{z\omega} \frac{0.5 + \sqrt{F_{15} / F}}{0.5 + \sqrt{F_{15} / F_\omega}} \qquad (9)$$

$$\xi_z = \xi_{z\omega} \sqrt{\frac{b_\omega}{b}}$$

where F, F_ω - area of the foundation and the loading plate
b, b_ω - factors found from formula (5) for the foundation and the loading plate.

Table 10 presents the results of prediction on the basis of the formulas, the results obtained after testing of the built foundations as well as the results of calculations using Equations (4) and (6).

Table 10.

Area of foundations under design	Loading plate tests		Tests on built foundations		Calculated values by formulas (4) and (6)		Predicted values by formula .(9)	
	$C_z \cdot 10^4$ kN/m^3	ξ_z	C_z 10^4 kN/m^3	ξ_z	$C_z \cdot 10^4$ kN/m^3	ξ_z	C_z 10^4 kN/m^3	ξ_z
25	9.2	0.095	1.5	0.38	2.8	0.35	1.9	0.38
384	12.1	0.082	0.095	0.19	1.7	0.27	1.4	0.25

As one can see from the Table the results obtained from the loading plate tests are in good line with the experimental values and closer than the results obttained from the proposed formulas though the latter also give the results well suited for practice.

CONCLUSION: The studies resulted in obtaining relationships for the factors of elastic uniform compression, relative damping and a value of added mass of soil by means of which one can achieve good description of vibrations of foundations under periodical, pulse and random loads.

The proposed device for dynamic tests of soils and the formulas to change-over from the plate loading tests to the real foundations permit to achieve sufficiently true predictions of any foundation vibrations.

REFERENCES.

1. Глубцова М.Н. Об учете присоединенной массы грунта при расчете вертикальных колебаний массивных фундаментов. Основания, фундаменты и механика грунтов, 1986 , N 1 .

2. Шехтер О.Я. Об учете инерционных свойств грунта при расчете вертикальных вынужденных колебаний массивных фундаментов. тр. НИИОСП Вибрации сооружений и фундаментов. М., Стройиздат 1948 , вып. 12.

3. Ильичев В.А., Таранов В.Г. Экспериментальное изучение взаимодействия вертикально колеблющегося фундамента и его основания. Основания, фундаменты и механика грунтов, 1976 , N 2.

4. Novak M. Uber die Nichtlinearität der Vertikalschwingungen von starren Körpern auf dem Baugrunde - Praga, Acta Technica, 1957, Nr 5.

5. Баркан Д.Д.,Трофименков Ю.Г.,Голубцова М.Н. О зависимости между упругими и прочностными характеристиками грунтов. Основания, фундаменты и механика грунтов, 1974 , N 1.

6. Александров Б.К., Пятецкий В.М., Степанов Г.Н., Тарянов В.Г. Экспериментальное исследование вынужденных колебаний большеразмерного фундамента. Тр. 5 Всесоюзной конференции "Динамика оснований,фундаментов и подземельных сооружений" - М.,1981 .

7. Савинов О.А. Современные конструкции фундаментов под машины и их расчет. - Л., Стройиздат , 1979.

8. Лапин С.К. Об определении динамических характеристик жесткости естественных оснований. Основания, фундаменты и механика грунтов, 1977 , N 3.

9. Shiwei D. The Effect of foundation Shape on Dynamic Parameter of bases . Second International Conference on Case Histores in Geotechnical Engineerin. Rolla, Missouri, 1988.

10. Александров Б.К. Фундаменты под оборудование. Тр.Ленинградского Промстройпроекта Л. , 1978.

11. Пятецкий В.М., Файнберг И.И. Виброизолированные фундаменты под барабанные окомкователи и мельницы для измельчения руды. Тр. Международного симпозиума " Виброзащита в строительстве", Л., 1984.

12. Пятецкий В.М. Колебания фундаментов под машины со случайными динамическими нагрузками. Тр. Международного симпозиума " Фундаменты под машины с динамическими нагрузками ". Л., 1989 .