SPECIALTY SESSION 8

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SOIL DYNAMICS AND SEISMIC EFFECTS ON FOUNDATIONS

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Gentlemen,

The topics for discussion in this session are:

1) Dynamic characteristics of soils (especially at high pressures)
2) Vibrations of foundations
3) Interaction of seismic waves and foundation bases and
4) Liquefaction of soils.

An examination of the literature published on these topics since the VII International Conference,where a specialty session on "Soil Dynamics" had also been held,is too voluminous to be discussed here. Besides the normal channels of dissemination of knowledge like technical journals,mention must be made of two- conferences which have since been held where papers on these topics have also been presented.

One of these is the conference on "Behaviour of Earth and Earth Structures Subjected to Earthquakes and Other Dynamic Loads" held at Roorkee in March 1973 and the other is the V-World Conference on Earthquake Engineering held in Rome in June 1973. The classification of papers in these conferences on the themes of to-day's discussion were as follows:

Theme	Conference at Roorkee (March 1973)	Conference at Rome (June 1973)
i	8	1
ii	5	2
iii	2	26+
iv	1	2

+General theme of soil structure interactio

Although fairly large amount of literature is being published, coherent pictures have not emerged in many cases. I propose to discuss a few important considerations on these topics and solicit the views of the house.

1. Dynamic characteristics of soils (especially at high pressures) are non-linear from the very beginning of the loading cycle.In several analysis of static problems, this fact is being accounted for. For analysis of problems of foundations and substructures under dynamic loads,the following soil properties are needed depending upon the method of analysis to be adopted:

(i) Elastic constants and damping properties
(ii) Modified shear parameters
(iii) Non-linear stress-strain

For shear modulus,effect of (i) effective mean principal stress (ii) degree of saturation (iii) void ratio and (iv) number of cycles of loading have been studied by Hardin and Drnvich (1972a). The authors data shows that single values of modulus and damping cannot be used for the complete analysis because of their dependence on strain amplitude, state of stress and environmental conditions. Cohesive soils are affected differently than clean sands. The authors have subsequently presented equations and graphs for the determination of shear modulus and damping of soils,for use in design problems involving repeated loading or vibrations of soils on both clean sands and clays. Similar results on soft saturated clays were obtained by other investigators also.

Field methods have also come into use of late. A block of standard dimensions is excited to resonance and shear modulus is inferred either by analysis proposed by Barkan or Richart. The values of shear modulus and elastic spring constants depend particularly on the 'dynamic force level' and the resulting 'dynamic strain .

In a stiff soil, if resonance of block is not reached a method has been proposed to extrapolate the resonance frequency only.

The important question is the identification of the "loading" and "strain" conditions and their simulation in laboratory tests. Typical values for different soils for use in "preliminary design" are not available. And finally "effort" needs be made for evolving a "standard" or at least a "recommended" procedure to determine the "shear modulus" and "damping" characteristics for a given problem.

For analysis, based on the assumption that soils are rigid plastic materials, modified "c" and "ϕ" values for dynamic loading are used. Typical analysis for dynamic bearing capacity are available for both 'cohesive' and 'c- ϕ' soils. The authors do not discuss how the relevant 'c' and 'ϕ' needs be determined.

Casagrande and his associates initiated work on danamic stress-strain characteristics of soils. Seed and Chan presented procedure for obtaining "total stress" (static plus dynamic) versus total strain characteristics (under static plus dynamic stress) of typical soils. Their approach was excellant for understanding the physical behaviour of soils under the influence of a combination of static and dymaic stress. Any further information on this topic would be a very welcome contribution to this session.

Non-linear stress-strain is automatically obtained in the above test set-up and can be used to solve stability problems.

Under high pressures, very little work has been done.

2. Vibrations of Foundations:

Under this topic, the problem of machine foundations shall only be discussed. Most of the literature concerns the behaviour of 'block' type foundations. For isolated blocks, vertical vibrations, simultaneous rocking and sliding and torsional vibrations of foundations have been considered. Methods of determination of relevant in-situ dynamic soil constants have also been proposed.

The methods could be classified into two groups:

(i) Based upon spring theory
(ii) Based upon elastic half space theory

Considerable amount of literature concerns the latter types.

The problem is to determine (i) natural frequency of the machine-foundation-soil system and (ii) dynamic amplitudes of motion at operating frequency. We have come much ahead of the days when a hammer of certain weight was dropped and natural frequency was assigned to a particular site. Natural frequencies of "typical soils" are listed even to-day (1973) in some texts. We appreciate that these concepts have no place in understanding the machine foundation behaviour.

Moore reports comparisons between observed and calculated (Sungs solution) maximum vertical displacement amplitudes which suggest that for small footings on sandy silty soils, the pressure distribution changes from parabolic at low mass ratios to the rigid base type at higher mass ratios, and for small footings on clay soils, theoretical displacement amplitudes for a rigid base stress distribution are in approximate agreement with observations.

Weissman'(1971) modified the analytical solution of forced torsional oscillations of an elastic homogeneous isotropic halfspace to account for the internal friction of soils and slipping of the foundations. Good agreement between the experimental data on silty clays and sand and the theoretically predicted values has been reported.

Based on the concept of elastic half-space and possible difference in the distribution of soil pressure at the base and some other variations from the solution of Sung (1953), "Ratay (1971) and Veletsos and Wei (1971) reported studies on sliding rocking vibrations of rigid bodies and plates respectively. A large list can be added to investigations of this category.

Most of these investigations do not consider the effect of depth of embedment on the response of footings. Gupta (1972) reported results of eight model footings of maximum size 5.75" x 4"(14.4 cm x 10 cm) four of which had the same shape, equal weights and different areas, whereas the other four had equal areas and different shapes. The resonant frequency was found to increase with increase in depth of embedment and this increase is independent of the shape of the foundation. The resonant amplitude decreased with increase in embedment, irrespective of the area and shape of the foundation.

More data from large sized footings or actual full scale foundations is very much needed in this direction.

Agarwal et al (1971), report prototype vibration studies on four compressor foundations under actual working condition. It was generally observed that vibration characteristics assessed by different theories do not differ significantly but Barkan's method seems to yield results which are in most cases quite close to observed values.

Barkan's method has the advantage of simplicity. However, it needs be modified to take into account the effect of (i) damping of the system (ii) depth of embedment and (iii) non-homogeneity of soils.

Several studies on development of simplified expressions for natural frequency, based upon model tests have since been reported (Prakash and Gupta 1968, Prakash and Puri 1972). Such studies have, at best, local applicability.

3. Interaction of Seismic Waves and Foundation Bases:

It is well known that the characteristics of ground surface motions during earthquakes and the corresponding forms of the response spectra vary with soil conditions (Seed and Idriss 1969). The study of damage to multistoreyed buildings during the 1967 Caracas earthquake indicates that the locations of zones of heavy damage may be attributed to unfavourable combinations of soil conditions and building characteristics which resulted in particularly strong response of the damaged structures (Seed et al 1972). Further, studies on the bridge foundation behaviour in Alaska Earthquake showed that bridges founded on bed rock sustained little or no foundation displacement. The greatest concentrations of severe damage occured in thick deposits of saturated cohesionless soils. Bridge foundations in saturated sand and silts sustained severe displacements even where the average SPT of the upper 9m (30 ft) of the soil was as high as 25. A method to calculate th response-spectrum of earthquakes for linear structures on an elastic half space has been developed.

The above mentioned and similar other studies point out to the importance of soil-structure interaction under dynamic loads.

Incidentally, the response on the subject for this session is rather poor. Hence, I would like to proceed with the next topic.

4. Liquefaction of Soils:

There is far greater awareness of the problem of liquefaction to-day (1973), than what it was a decade ago, liquefaction of soils is responsible for major land slides, lateral movements of bridge abutments, settling and tilting of buildings, failure of water front retaining structures, cracking in embankments and 'float up' of septic tanks.

Factors affecting liquefaction are very well known e.g. (i) soil type (ii) relative density or void ratio (iii) initial effective confining pressure (iv) intensity and duration of ground shaking.

The concept of "critical void ratio" advanced by Casagrande cannot be applied to liquefaction phenomenon since CVR is not a unique property of sands. However, according to the concept of "critical acceleration" (Maslov 1957), every sand can be liquefied at and above critical acceleration. Such a concept shall have obvious limitations since (i) it does not take account of the effect of frequency and (ii) direction of vibrations and (iii) realistic intensity of ground motion during actual earthquakes.

The first question that an engineer would like the answer for, is as follows:-

GIVEN:- (i) A sand with known grain size distribution, grain shape and initial relative density (ii) the intensity and duration of ground shaking (iii) initial state of stress.

REQUIRED:- (i) To predict if the sand is likely to undergo liquefaction, partial or complete (ii) if liquefaction is anticipated, how to predict the amount of "tilt of buildings", "cracking in earth dam" and in general the extent of damage to structures (iii) alternatively, the remedial measures to check liquefaction.

Obviously, 'critical void ratio' and 'critical acceleration', concept do not hold any promise.

Two other procedures have come into vogue for studies on liquefaction:

A. Laboratory Test: Oscillatory triaxial and pure shear tests have been extensively developed at University of California, Berkeley. The essence of this procedure is as follows: (Seed and Idriss 1971).

a) For given soil conditions and design earthquake, determine time history of shear stresses with depth, (b) Convert the stress history into an equivalent number of uniform stress cycles, (c) From available test data on specially conducted field tests, determine the cyclic shear stresses which cause liquefaction in the same number of cycles as in (b). (d) A comparison of shear stresses induced by the earthquake with those required to cause liquefaction, the zone of possible liquefaction is determined.

Seed and Peacock (1971) further report that both analytical and experimental evidence as well as available field data suggest that most laboratory triaxial compression test data should be reduced by a factor of 0.55-0.70 depending upon the density of soil and the duration of the earthquake in order to determine the factor $\frac{\text{H-shear stress}}{\text{confining pressure}}\left(\frac{T_{dv}}{\sigma_o}\right)$ causing liquefaction under field condition.

However, based upon 37 cases where liquefaction occured and had not occured, the authors suggest that for a maximum ground acceleration, it is possible to designate 3-ranges of relative density:

(i) A range in which liquefaction is very likely to occur
(ii) A range in which liquefaction is very unlikely to occur and
(iii) A range in which liquefaction may or may not occur depending upon characteristics of sand and other factors.

B. Vibration Table Tests: In these studies, the sand is deposited in a large tank mounted on vibration table and vibrated at predetermined frequency and amplitude (Florin and Ivanov 1961, Maslov 1957, Yasnimi 1967, Prakash and Gupta 1970). The response of soil deposits is carried out in some what a similar manner as outlined above, A correlation of field and laboratory data from such studies has also been attempted (Prakash and Gupta 1970b).

About the Scientific Reports:

It may well be in order briefly to summarise the scientific reports which will come up for discussion in this session.

Of the 19 reports finally selected, their classification into different themes is as under:

Theme No.	(i)	(ii)	(iii)	(iv)
Number of Reports	3	4	5+	7

+ including vibration transmission

Of the three reports on stress-strain behaviour of soils under dynamic loads, two evaluate deformations of soils. In addition, in one of these reports, the effect of pulsating load on 'Φ' has been found to be negligible. In the third report, determination of seismic properties of soils by steady state vibrator has been described.

Of the four reports on machine foundations, two deal with response of machines from the field and in the other report, a SDF mass-spring dash-pot with Coulomb damping has been proposed for the study of dynamic response of an embeded footing. There is one report on vibration insulation.

Of the seven scientific reports on liquefaction, one reports pore pressures in field due to pile driving, two report triaxial tests to study liquefaction in sands and three advance analytical tools for study of liquefaction based upon some sort of laboratory test data. And finally, one report deals with liquefaction of sandy silt.

Studies on liquefaction and fine grained soils have not received adequate attention so far. A recent work (Gupta and Gangopadhyaya 1973) suffer from the disadvantage that uniform samples could not be prepared.

Of the other five scientific reports, three reports describe vibration transmission from ramming vibrations and demolition of cooling towers and their measurement. Of the other two reports, one deals with decreasing the building vibrations by increasing its period and the other describes pile response with the introduction of intermediate cushion.

I had no access to very useful Russian literature on the themes of this session. I, therefore, believe, that we would benefit very much from the contributions of our hosts of the conference.

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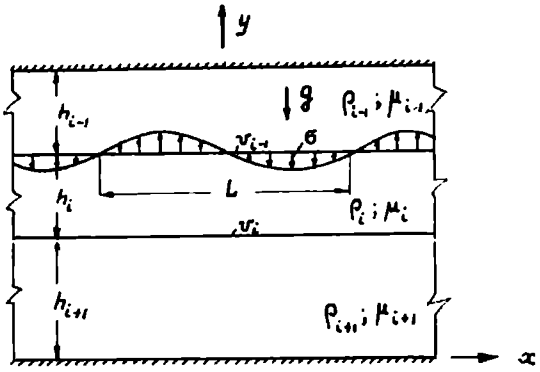
Vice-Chairman Prof. Sinitsyn A.P. (USSR)

The soil dynamic acquired an important meaning for elaboration of earthquake resistant buildings and structures,in seismic region of the USSR. The theoretical investigations fulfilled by means of the computers and the experiments due in situ allowed to design such a prominant structures as the Nurek dam with the height of 300 mt, and the multistory buildings in the townes situated in areas with seismicity of 8-9 degrees. Those investigations are shown that the reaction of the structure on seismic boards is dependent from the engineering-geological conditions for the building area on which the structure is placed. The theoretical solution of the problem is considered by mean of the design model of the multidegrees of freedom system and by using the finite elements method for calculation of vibrations of the buildings and structures influenced by seismic waves propagated in soil foundation. The calculations were made for a travelling seismic wave taking into account the displacements and the rotations of the structure. The general stability of the structure is evaluated by means of the soil reactions diagrams. The nonlinear motion problem of the system "structure-foundation" is solved and the instability areas of motion are determined which depends on the correlation between the physical parameters of the structure and foundation. For example the upper sedimentary stratum of variable thickness highly influenced on the transmission of the seismic energy to the structure. The inclination of the rock foundation which spreads the sedimentary stratum and the influence of the variable thickness was investigated by the finite elements method for the dynamical problem. It is shown that the amplitudes of accelerations on the surface of the sedimentary stratum may be twice more as the accelerations of the rock foundation. For the stratum with a triangular crossection the correction coefficients are determined and the conditions are evaluated for arising of elasto-plastic waves in connection with the geometric dimensions of the stratum. The problems of seismic waves propagation in a layered foundation and the interaction with the structures foundation require to fulfilling further investigations. It is necessary to discuss some questions. In my opinion there is made insufficient attention to the vertical component of travelling seismic wave. This component creates a vertical force under the foot of the structure which may lift the structure and must be added to the others vertical loads. The vertical and horisontal vibrations are connected and those frequencies must be determinated jointly. To determinate this suplementary lifting force it is necessary to investigate the vertical vibrations. The amplitudes of the vertical component of the seismic wave propagating in the sedimentary layer due by strong earthquake depend on the reflection and the seismograms recorded in situ should be considered as the realisations of a random process. The evaluation of auto and crosscorrelation functions by means of records of the soil displacements registered on the surface of the sedimentary layer arised from a dynamic source permit to determine the energy flux transmitted through the sedimentary layer during earthquake. The graph of alteration of the potential energy as function of the relative maximum displacement of the foundation may be drawn. The energy accumulated in the building during elastic range of deformation more as proportional to the square of the value of the displacement,but if the displacements increase more in the buildings structure should occur the plastic regions and the rate of growth of the potential energy decrease. At least after any rupture in structure take place potential energy diminished. For each level of energy transmitted during earthquake to the building corresponds a certain displacement of the structure foundation and the definite value of potential energy accumulated in the building. The comparison of these two energies permit to evaluate the seismic resistance of the building. These statements shown that the dynamic of soil must be developed further by means of experiments made in situ and theoretical investigations with using of computers. It is interesting to know the considerations of members of our session about these questions.

On slides it is shown the determination of instability areas by propagation of the vertical component of a simple seismic wave in the layered foundation.

Slide N 1. There is shown the scheme of a



layered foundation. The parameter is characterised by the rates of the physical properties of the layers, and permit to determine the instability areas.

On the slide N 2, are shown the equations

$$a_{11}a_{22} - a_{12}a_{21} = 0$$

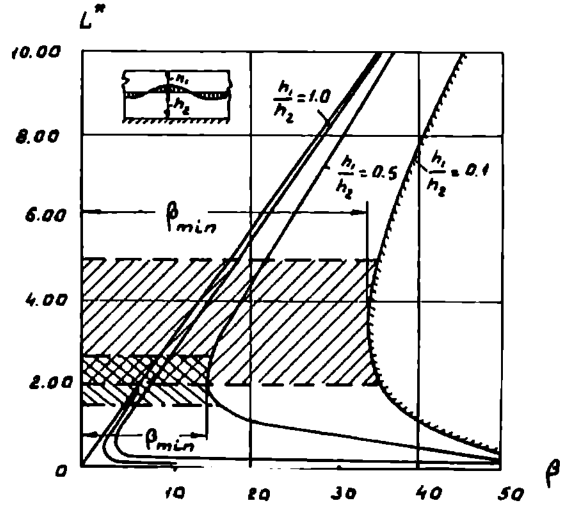
$$a_{11} = \left[A_1 + \frac{L}{2\pi h_2}\left(1 + \frac{\rho_2}{\rho_1}\right)\beta\right], \qquad a_{12} = a_{21} = B_1$$

$$a_{22} = \left[A_1 + \frac{\mu_2}{\mu_1}A_2 - \frac{L}{2\pi h_2}\beta\right].$$

$$\beta = \frac{(\rho_1 - \rho_2)\,g\,h_2}{\mu_1}.$$

for determination of instability areas.

On the slide N 3 are shown the graphs to determination of the length of the seismic wave by which the instability occurs. These graphs are calculated for a two layered foundation as function of . On the left of each curve there is a stability area, on the right the instability.



Vice-Chairman of the Specialty Session VIII

Prof. D.D.Barkan (USSR)

The influence of the soil properties in bases of structures on the designed seismical loads is estimated by the increase of seismic intensity which depends on the acceleration of seismic soil vibrations, i.e. upon its amplitude and period. For a constant wave period the change in the seismic intensity is determined by the change of the vibration amplitude. Thus, from the point of view of the influence of the soil on the designed seismic loads soils should be classified in relation to properties that influence only the amplitudes of seismic waves, everything else being constant.

The modulus of elasticity of the soil and the soil density are the main characteristics effecting the wave amplitudes.

When waves originated by the spherical pulse are propagated in an infinite space the amplitude are inversely proportional to the elastic shear modulus μ of the soil, the distance between the considered point and the source being large as compared with the wave length. For the short distances from the source, soil amplitudes due to the Rayleigh waves originated by the surface source are inversely proportional to the shear modulus of the soil and depend slightly upon the soil density. As the distance to the source increases, the relationship between the amplitudes of Rayleigh waves and the shear modulus, and the density varies somewhat. But as the distance increases (in comparison with the wave length) the amplitudes became inversely proportional to $\mu^{3/2}$.

In other cases the relationship between the amplitudes of waves propagated near the surface of the homogeneous soil and the modulus of elasticity, and the density may somewhat differ from the above mentioned laws. It seems still that it can be assumed that the amplitudes are at least inversely proportional to the shear modulus. This means that the relationship is the same as for the static case.

Thus soil classification in accordance with the increase of the seismic intensity must be based on a comparison of soil moduli of elasticity.

The existing opinion that the groundwater level has a significant influence on the increase of the seismic intensity is doubtfull. The doubts are verified by the absence of appropriate reliable data in technical literature. For some soils it may be that the groundwater level influences the seismical intensity as a consequence of the change in the thickness of the saturated layer and hence of the change of its physical and mechanical properties (but this is not true for all soils).

The physical and mechanical properties, (and hence the seismic intensity) of coarse, medium end even of fine grained sands slightly vary with the moisture content. The void ratio is the main factor effecting the elastic and strength properties of sands. The soil characteristics of dust like materials, especially of silty and clayey soils are worst when these soils are below the groundwater level in comparison with when they are above groundwater level. Therefore, from the point of view of seismic intensity the first case is more dangerous.

Thus, the groundwater level cannot be regarded as a unique seismic characteristic of a building site.

Numerous experimental data shows that a proportional relationship exists between the elastic and strength properties of soils.

Thus soils can be classified according to the increase of seismic intensity based on the strength properties of the soil, instead of the elastic ones.

The increase of seismic intensity on building sites may be determined by a "coefficient of ground condition" K, which is equal to the ratio of the strength characteristics of a soil taken as a mean soil (according to the seismic intensity) to the strength characteristic of the soil on the building site. Such a mean soil can be represented by a soil with an average value of the strength characteristic R, equal to 2,5 kg/cm^2.

The value of the coefficient of ground condition K for soils with a strength characteristic $R \neq 2,5$ kg/cm^2 is determined by the formula

$$K_s = \frac{2,5}{R}$$

A table of K_s values vor various soils classified according to the USSR building codes, and based on the abovesaid set forth

NN	Soil			The K_s value
1	Rocky soils, fissured-erupted, metamorphic and sedimentary: granite, gneiss, limestone, sandstone, conglomerate etc;			0,3
2	Semirocky soils: marl, flinty clay, argillaceous sandstone, tuff, shell, gypsum etc;			0,4
	Coarse debris:			
3	Crushstone (coarse gravel) with voids filled with sand:			0,4
4	Gravel from the debris of the crystalline rock;			0,5
5	Gravel from the debris of sedimentary rock;			0,9
	Sandy soils:		dense	semidense
6	Coarse sand of any moisture;		0,6	0,7
7	Semicoarse sand of any moisture;		0,7	1,0
8	Fine sand:			
	a) of little moisture content		0,9	1,3
	b) very moist and saturated		1,0	1,7
9	Silty sand:			
	a) little moisture content		1,0	1,3
	b) very moist,		1,3	1,7
	c) saturated;		1,7	2,5
	Argillaceous soils:	Void ratio	Consistency index	
			B=0	B=1
10	Sandy clayey soil	0,5	0,9	0,9
		0,7	1,0	1,3
11	Loam	0,5	0,9	1,0
		0,7	1,0	1,4
		1,0	1,3	2,5
12	Clay	0,5	0,4	0,6
		0,6	0,7	0,9
		0,8	0,9	1,3
		1,0	1,0	2,5

For bases of structures consisting of several soil layers with different strength characteristics, the change of the seismic intensity may be estimated by the average value of the ground condition coefficients.

Very loose soils (loose sand with a void ratio of more than 0,7-0,8; the argillaceous soils in the liquid state with a consistency index exceeding 1; peat and mud, etc) are not included in the above given table. Such soils cannot be used for building up without improvement of their properties even in nonseismic regions.

Improvement of properties of these soils is attained mainly either by means of piles or by means of deep foundations or otherwise..

Driven piles compact soil around them so that its strength properties are improved and as a consequence, a week soil together with the piles forms a base with better deformation and strength properties.

Preliminary tests show that everything else being equal, the vibration amplitudes of the soil with the driven into it piles diminish from 2 to 3 times. Thus the ground condition coefficient for soils compacted by piles driven under the whole area of the structure may be taken approximately as above in the ratio as compared with natural soil.

In conclusion it may be said that the aforesaid relationship between elastic and strength characteristics is very usefull not only for the estimation of the effect of the soil properties on the seismic intensity, but it also permits essentially alter the method for determining soil strength characteristics. By means of a geophysical prospecting methods we can determine the longitudinal and transversal wave velocities in a certain soil layer in the base of the structure. By means of these values we can easily calculate the modulus of elasticity, modulus of shear and even the Poisson ratio. Employing correlation between the elastic modulus and the strength characteristics, the last can be found without boring, sampling and soil testing. The great efficiency of such a method (classified as geophysical correlation) for the determination of soil strength properties is obvious when compared with the existing methods (especially for the sandy soils, where undisturbed sampling is practically impossible).

Chairman Prof. Sh. Prakash (India)

Thank you Prof. Barkan. The first contribution will be in charge of Mr. Stavnitser. (USSR) Mr. Stavnitser will you please.

Mr. L.R. Stavnitser (USSR)

When the results of triaxial dynamic tests of sand specimens in a vibrostabilometer are processed by means of the theory of limiting equilibrium, they create the illusion that the angle of internal friction is reduced by vibrations. But the concept of an "effective" value of the angle of internal friction at a limited level of vibrations enables the apparent reduction in the strength of the sand to be explained by the periodic changes in its stressed state. Here the true angle of internal friction remains constant.

In testing soils in a vibrostabilometer, the dynamic pressure, varying with time according to a harmonic law, is added to the value of the static lateral pressure. During one half of the period, the lateral pressure is reduced with respect to its mean value. This proves to be sufficient to lower the failure load deviator in comparison to static conditions. The results of such dynamic tests can be interpreted statically on the basis of the theory of the limiting equilibrium of soils by introducing the "effective" value of the angle of internal friction. This value is less than the true value and depends upon the parameters of the dynamic effect.

The obtained formula for the "effective" angle of internal friction

$$\sin \varphi_e = \frac{(\pi - k_0) \sin \varphi - k_0}{\pi - k_0 (1 + \sin \varphi)}$$

is valid only for values of the dynamic-response factor $k_0 < 1$, i.e. for amplitudes of the dynamic component of the stresses not exceeding their static values. This formula has been confirmed by numerous experimental data for fine-grained sand in the frequency range from 0 to 10 hertzes, which is typical of seismic vibrations.

Chairman Prof. Sh. Prakash (India)

Thank you Mr. Stavnitser. The next will be Dr. Jinkin from the USSR.
Dr. Jinkin will you please

Dr. G.N. Jinkin (USSR)

The soils of the railway bed are subjected to multiple action of the vibrodynamic loads. resulting in a thixotropic decrease of their strength. That decrease depends on the character of railway traffic, on the value of vibrodynamic load and on the state and the characteristics of the soils.

By the experimental researches carried out in the Leningrad Institute of Railway Transport the following results have been obtained.

1. Some decrease of the strength characteristics of the soil is registered when the train approaches to the experimental section to 300-350 m. When the train approaches to 70-80m, an essential strength decrease is registered. The maximum strength decrease is observed at the level of the upper surface of the railway bed when the freight trains go; in this case the strength decrease reaches 40-50 per cent.

After departure of the train from the experimental section the soil strength is quickly restored. In 2-3 minutes the strength increases to 70 per cent. In 60-70 minutes the soil strength is practically completely restored.

2. The results of experimental investigations proved that there is an exponential correlation between strength decrease and the vibration intensity. It is determined, that the quick decrease of strength characteristics to nearly 45-50 per cent lasts to a certain intensity. The exceeding of this intensity does not lead to a considerable strength decrease.

3. When the speed of the trains is not great, the vertical soil vibrations in the railway bed exceed the horizontal soil vibrations across railway line. With the speed of 120-140 km/h these vibrations are equal. When the speed of the trains becomes greater, the horizontal vibrations exceed the vertical vibrations.

The action of the vibrations extends to a depth of 4,0-4,5 m and in the horizontal direction to a distance of 500-600 m.

4. The maximum relative strength decrease under vibration is registered in interval from 0,25 to 0,45 of the plasticity number.

5. With an increase in the natural density of the argillaceous soils from 0.92 to 0.96 of their maximum strength there is an intense increase of their dynamic resistance and strength. When we take the soils with a density more than 0.96, this process damps.

6. The vibrodynamic loads cause a rupture of the structural lincs of the argillaceous soils in the state of plastic congelation. This phenomenon determines the decrease of their strength in the relatively small temperature interval from 0 to -2,4°C.

The maximum loss of the strength of the argillaceous soils in the state of plastic congelation takes place because of a decrease of the specific cohesion and in the degree less considerable-because of a decrease of the friction.

The mineralogical composition of the soils has no influences on a value of the coefficients of the relative cohesion and friction loss, but changes essentially a temperature range. That is why when we choose the bed, it is necessary to prefer the kaolinite soils with the content of 25-30 per cent of the argillaceous particles and with a minimum content of the univalent cations.

The increase of the ice content in the congealed soils under a negative temperature invariable leads to the decrease of the strength characteristics.

Chairman Prof. Sh.Prakash (India)
Thank you Mr. Jinkin.
The next speaker will be Mr.Shvets (USSR)
Mr.Shvets will you please.

Mr.N.S.Shvets (USSR)

Here I should like to mention the results of the vibration study of massive foundations for large ball-and-rod grinding mills having a short drum of over 5 m diametre.

These mills are usually installed on monolithic or frame foundations up to 15m high.

The operation of such mills at floration works revealed that considerable dynamic stresses arise and act on the foundation though these mills are considered to be balanced. As a result mill foundations are often induced to vibrate which sometimes results in foundation deformations and imperfection of grinding process.

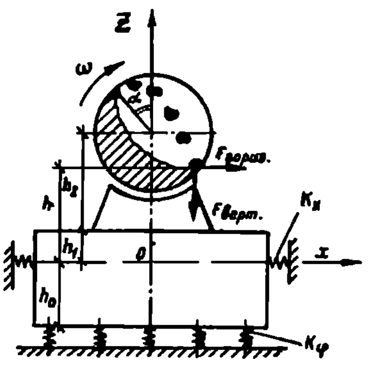
The investigation of foundation vibration of the acting ball-and-rod grinding mills carried out at fifteen floation works and also the use of a large model having similar physical dynamic properties permitted to discover that the reason of the foundation vibration was uniform movement of grinding bodies which get together in groups in the mill drum and travel in free falling trajectory (1).

To specify the vibration incitement of the "mill-foundation-footing" system statistic computation of foundation vibration oscillograms (output signal) was carried out according to a special program with computer M-220.

The results of the computation permitted to consider the process in question stationary.

The study of power spectrum of input signal for the mill foundation permitted to discover that the incitement of the "mill-foundation-footing" system according to the incidental functions theory was impact.

As a result of the solution of differential equations of the system movement which is regarded as a one-mass system with two degrees of freedom we have obtained a theoretically grounded relation for defining amplitude of vibration of the upper surface of the foundation for the grinding mills.



The comparison of the amplitude of vibration defined according to out formular with the actual changes indicated that the difference in their values does not exceed 20 per cent.

The studies at the acting grinding mills revealed that the value of admissible amplitude of vibration must not exceed 0,5 mm.

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Chairman Prof. Sh.Prakash (India)
Thank you Mr. Shvets.
The next contribution belongs to Mr.Ilyichev from the USSR
Mr. Ilyichev will you please.

Mr.V.A.Ilyichev (USSR)

There are different trends in the above mentioned investigations carried out in the NIIOSP soil dynamic laboratory. Here I would like to concider some singularities of these studies. There are many problems in the soil dynamics developing very intensively. Up to now there are no practical recommendations for many of these problems among which are predictions of the vibration level of soil, foundations and the structures upon them due to vibrations generated by the foundation-source in their neighbourhood. It must be noted that in many cases the real vibration parameters of a mashine foundation are far from the calculated ones. All these problems are closely interconnected and to make clear these relations is the purpose of our investigations. The theoretical investigations are based mainly on the hypothesis that soil may be represented by the model of elastic,isotropic homogenous semy space. The experimental data are to be used for the construction of the transfer function of weithless stamp-soil system. Such method for interpretation of the test data give more information as compared with the usual methods of determination of rigidity and damping coefficient. The transfer function of stamp-foundation receiver may be determined in the similar way. This method will allow to have a more adequate approach to the real dynamic model and to formulate practical recomendations for the aforesaid problems.

In our laboratory the theoretical and experimental investigations concerning the earthquake proof foundation engineering are also carried out.

The main purpose of studies was to elaborate the practical recomendations for design of foundations with account of soil and building properties. The fondation and base are the parts of the total soil-structure dynamic system and therefore while designing the same principles are to be used as for any other part of the system:

1. The total system under seismic forces is calculated as elastic one with known dynamic properties of soil and foundation.
2. Bearing capacity of foundation must be determined and then compared with the loads found in the previous item.

The practical recomendations developed in the soil dynamic laboratory of the NIIOSP are destined for flat and usual pile foundations with an intermediate gravel and sand cushion.

The theoretical studies concerning the liquefaction of sand soils which have been carried out in the soil dynamic laboratory allow to evaluate settlements as a function of time and the compaction zone.

The lack of time does not permit to give more details for aforesaid researches and to enumerate some other problems. For some of them there are special communications.

In conclusion it must be noted that the most part of the experiments have been made in situ with foundations on a large scale.

Chairman Prof. Sh.Prakash (India)
Thank you Mr.Ilyichev
The next contributor will be Mr. Singh and Mr.Neville C.Donovan from USA
Mr.Singh will you please report your contribution.

Mr. Singh S. and Mr. Neville C.Donovan (USA)

A review of the state of the art indicates that there exists no easy method by which to account for the effect of slope on the seismic liquefaction potential of soil. From a study of Lower San Fernando Dam failure and Sheffield Dam failure, preliminary indications are that for a 2 1/2 to 1 (40 per cent) slope on the upstream face of Lower San Fernando Dam, the stress ratio required to induce liquefaction would be higher (maybe as much as 50%) above that required on level ground. How this relation will vary with the inclination of the slope is a question which seems to have not been resolved completely, based upon results of cyclic loading triaxial tests in laboratory it has been shown (Lee and Seed, 1967) that the cyclic deviator stress required to cause liquefaction increases with an increase in the value of Kc-the anisotropic consolidation. At very large values of Kc, liquefaction just does not occur in laboratory samples; large strain can, however,take place. The writers believes that at high values of Kc,significant stress reversals do not occur and liquefaction, therefore,does not develope. It is therefore valid to assume that liquefaction is a function of stress reversals. This leads to a simple conclusion that the effect of a slope in reducing the liquefaction potential of a soil is essentially due to a reduction in the stress reversals. The problem, however, of determining this reduction for different slope types and inclinations,is not so simple. Elaborate laboratory testing together with stress analysis (finite element) to take into account the effect of sloping boundries on the soil response,are required to develope some reasonable relationship. The problem in general appears to be quite complicated when the randomness of earthquake motion has also to be considered.

Recently a stochastic approach to the seismic liquefaction problem has been introduced by Donovan (Donovan,1971), and a cumulative damage analysis has been developed to evaluate liquefaction potential of soils. The analysis, in its principle is analogous to fatigue (cumulative effect of stress reversals) failure in other engineering materials. Based after a study of the distribution of cyclic stress peaks in observed earthquake records, the distribution of stress during the design earthquake is obtained. This is done by using statistical procedures where the parameters for earthquake motion are expressed in random vibration terms and Rayleigh and wide band spectrum stress distribution options are used. This allows a direct evaluation of liquefaction without the requirement that a seismic responce analysis be made on the soil profile. (Reader is referred to the original paper by Donovan for details).

If liquefaction is assumed to be directly proportional to number of reversals,then the reduction of stress reversals due to slope effect can be easily considered by reducing the area under the stress distribution envelope in the cumulative damage analysis. To produce reversal seismic stress must exceed the downward slope component. This established limits between which the stress envelope distribution will be integrated and effects evaluated on the development of liquefaction using the cumulative damage analysis. The effect of a slope in reducing the liquefaction potential of a soil, can then be expressed in terms of a number relating the increased resistance to liquefaction due to slope to the resistance of the soil on level ground.

This incidently introduces and or calls for a concept of factor of safety against liquefaction failures. This, however, is a subject requiring separate studies and discussions. For the purpose of the present discussion, the number or the ratio defined above will be termed as factor of safety against liquefaction.

Incorporating the concept outlined above in the cumulative damage analysis and adopting it for use on a digital computer, detailed study of the slope effects on liquefaction characteristics of soil was made and results plotted in Fig.1 and 2. It will be noted from Fig.1 that the capability of a given soil type to withstand liquefying tendency or to almost eliminate stress reversals increases rapidly as the slope inclination reaches beyond a certain limit which depends upon the level of earthquake excitation and relative density of the soil. Fig.2 shows how an embankment of a moderately dense soil can perform satisfactorily against liquefaction when subjected to strong motions as compared with the same soil on level ground.

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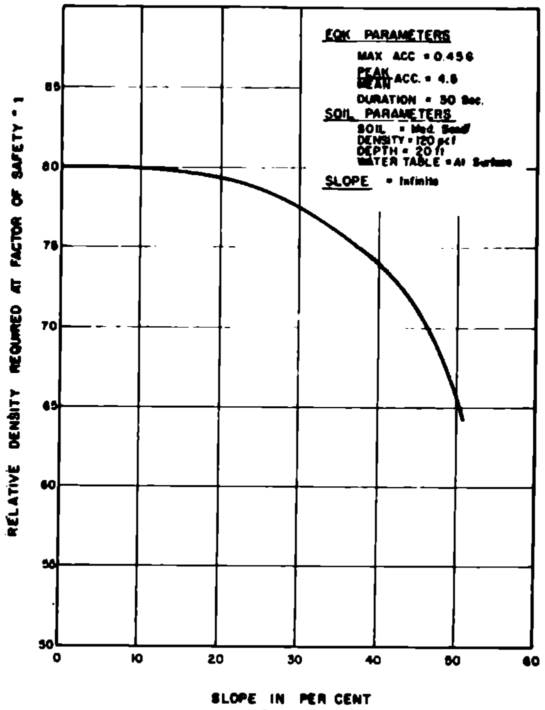

Fig.2

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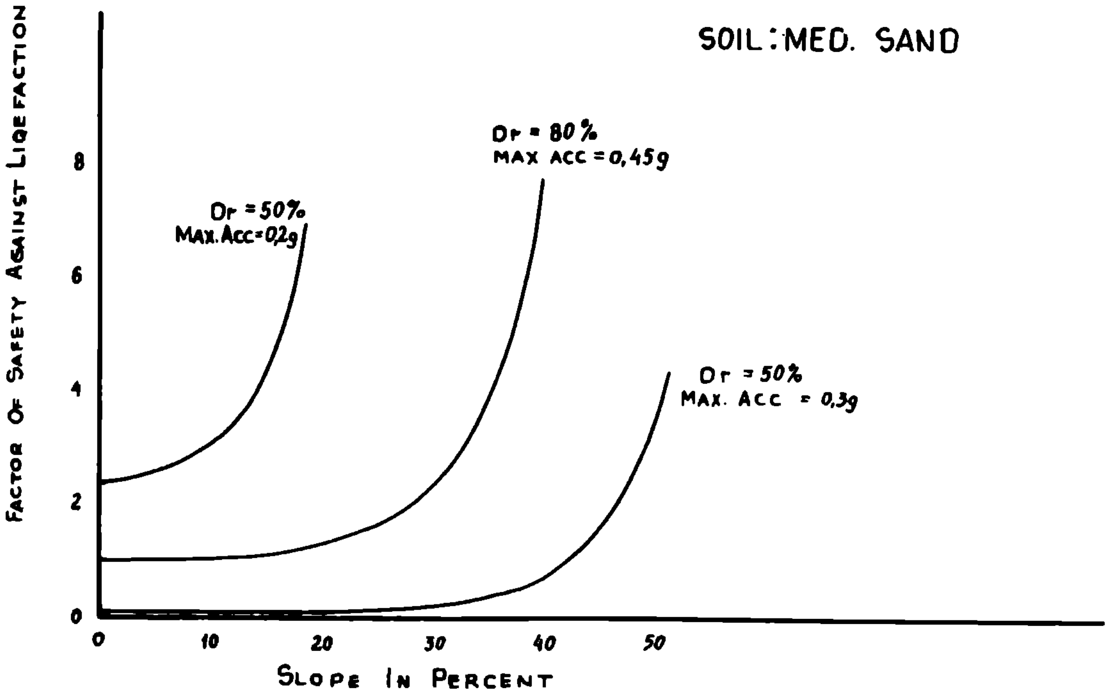

Fig.I

Chairman Prof.Sh.Prakash (India)
Gentlemen, Mr.Singh will report the next contribution also.

Mr.Singh S. (USA)

Current procedures of evaluating seismic liquefaction fail to take into account the followings:

1. The effect of vertical acceleration on the development of pore water pressure. Vertical acceleration, though always less than the horizontal acceleration, can be quite significant in some earthquakes. As vertical acceleration cannot be produced by shears waves propogating upwards, the assumption made in the current procedures that the pore pressure developes only as a result of shear stresses set up by a series of upwardly propogating shear waves, fails to recognize that the build up of pore pressure can occur in more than one way during the ground shaking.
2. Coupling effect of soil water system and its contribution to the development of pore water pressure during earthquake excitation.
3. Velocity of earthquake motions has been shown to greatly affect the slope movements during earthquake loading (Newmark, 1965). In case of liquefaction of soils, it is yet to be shown or proved that the velocity does not affect liquefaction more than acceleration.

Considering the uncertainties involved and unaccounted contributions of some important variables affecting liquefaction, and in the absence of a universally accepted explanation of the phenomenon of liquefaction, following concept/hypothesis is proposed.

The concept is based upon the response of a multi-phase system to seismic wave propogation. The progressive increase of pore water pressure and the consequent loss of strength of soil, though forms the basis of this hypothesis as in others, but the development of pore water is assumed to develope as a result of a phase lag in the components of the system. When seismic waves propogate through the soil water system, the inability of pore water to undergo strains compatible with the strains undergone by the soil phase (soil grains and soil structure) results in a response lag which, depending upon how opposed the direction of vibration of the two phases are, give rise to the development of pore pressure. It is interesting to note that this hypothesis implies development of phase lag and consequent development of pore pressure by all types of seismic waves as opposed to currently held concept that it is only the shear wave, propogating vertically upwards, which causes the development of shear stresses and hence, the rise of pore pressure. By making an anology with an electrical circuit made up of resistance and inductance, work is under way (Singh, 1973) to study the behavior of soil water system subjected to seismic wave propogation. The development of inductance with a consequent decrease in the resistance as the electrical current passes through the system is considered analogous to the development of pore pressure and loss of strength of the soil water system when seismic waves propogate through it.

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Chairman Prof. Sh.Prakash (India)
Thank you Mr. Singh.
Now I pass the word to Mr. Richart (USA)

Mr. Richart will you please.

Mr. F.E.Richart (USA)

In the resonant column test, cylindrical samples of soil are set into longitudinal or torsional vibration at their resonant frequencies to evaluate velocities of propagation of the longitudinal or shear wave in the sample. Several models of resonant column devices are available to evaluate the effects of confining pressures, amplitudes of vibration, stress history, and time of loading on the wave velocities for a variety of soils. A discussion of the types of machines and some of the test results are included in ref (1).

This discussion treats the effects on shear wave velocity produced by extended time of hydrostatic loading. Initially the sample undergoes primary consolidation as the confining pressure is applied. Development of primary consolidation is indicated in the resonant column test by a relatively rapid increase in shear wave velocity with time. After 100 to 1000 min of continuous pressure application, the increase of shear wave velocity settles down to a constant rate, which is represented by a straight line on the semi-log plot of shear wave velocity vs. log time. Figure 1 illustrates the change in shear wave velocity as a function of time for a cohesive soil. Primary consolidation was essentially completed in 1000 min., then the straight-line increase continued to 10,000 min in this test.

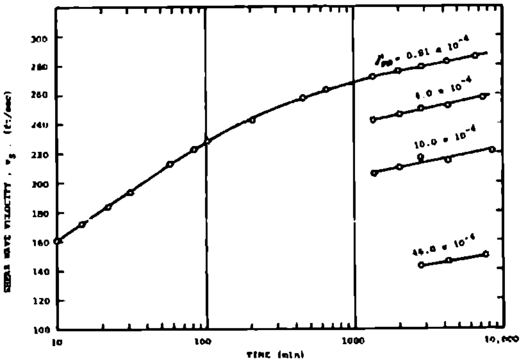

Fig.1. Shear Wave Velocity vs. Time for Resonant Column Test of Cohesive Soil.

Earlier tests (see Ref.2) have indicated that this straight line relation continues for at least 10^6 minutes.

Fig.1 shows the rate of shear wave velocity increase during the primary and secondary phases for low amplitude vibrations corresponding to shearing strains of 0.8×10^{-4}. For larger amplitude shearing strains of 4×10^{-4}, 10×10^{-4}, and 46×10^{-4} only the secondary increase is shown. As the shearing strains increase, the shear wave velocity decreases, but the time rate of secondary increase in shear wave velocity is essentially constant.

This rate of secondary increase in shear wave velocity with time is a function of material properties. At the present time the best indicator of the rate of secondary increase is the median particle size, D_{50}. Figure 2 shows data on cohesive and cohesionless soils using D_{50} as the abscissa. The ordinate of the diagram is the rate of increase of the shear modulus, $G(= \rho v_s^2)$, per log cycle of time, expressed as a percent of the value of G at time equal to 1000 min. Fig.2 indicates that for $D_{50} > 0.04$mm the ratio $\Delta G/G_{1000}$ is less than about 3 percent, whereas for the fine-grained soils ($D_{50} < 0.004$) this ratio may become as much as 10 to 15%.

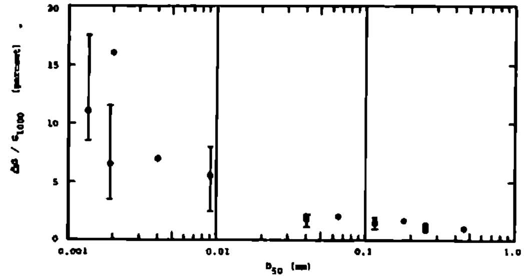

Fig.2. Time-Dependent Increase in Shear Modulus for Cohesive and Cohesionless Soil.

These results from resonant column tests indicate that laboratory tests might be expected to give lower values of shear wave velocity than those obtained from field tests.

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Chairman Prof.Sh.Prakash (India)
Thank you Mr.Richart.

The next will be Mr.Woods from USA
Mr.Woods will you please.

Mr. R.D.Woods (USA)

In this discussion I only wish to up-date our progress in Holographic Interferometry since the submission of the Paper, "Holographic Interferometry in Soil Dynamics".

All of the results presented in that paper were applicable only to the vertical component of particle motion. For a complete description of a surface wave phenomenon, the horizontal component is also required. A nearly horizontal component of motion of the surface of the half-space model has recently been recorded by re-orienting the optics of the experimental set-up, and interferograms showing fringes which represent nearly horizontal displacement components have been successfully made.

Another hologram interferometry technique not described in the paper has also been used recently. This technique does not yield contours of displacement but produces an illusion of a three dimensional view of the displaced shape of the half-space surface. To produce the 3-D effect, it is first necessary to make a static hologram of the ahlf-space surface. Then, an optical wedge is placed in the object beam of the interferometer. Now, viewing the static model surface through the hologram, shows closely spaced parallel fringes all across the half-space surface as in Fig.1. By vibrating the sand surface and pulsing the laser at the same frequency, the

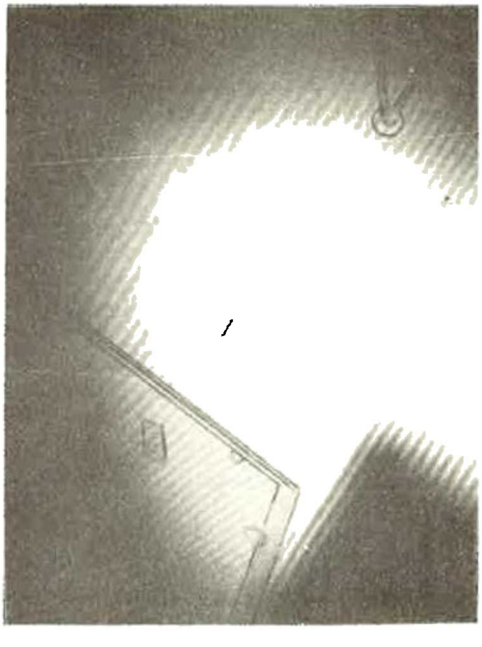

Fig.1. Static Interferogram with parallel fringes.

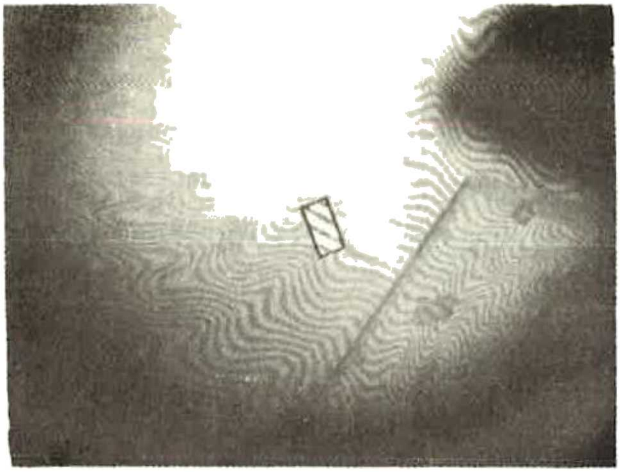

Fig.2. Dynamic Interferogram with 3-D illusion

formerly parallel lines on the half-space surface become wavy and produce the 3-D illusion shown in Fig.2.

Chairman Prof. Sh.Prakash (India)

Thank you Mr.Woods.
Now I want to call on Prof.Ortigosa (Chile)
Mr. Ortigosa will you please.

Prof.Pedro Ortigosa (Chile)

One of the common problems in soil dynamics is to estimate the liquefaction potential of saturated sandy soils subjected to earthquakes. One of the most popular methods to approach the prediction problem is to run undrained cyclic triaxial tests on saturated samples isotropically consolidated. In order to represent the liquefaction mechanism which occurs inside the sample,a model was developed including: (1) the relation between the compaction of the soil skeleton and the cyclic shear stresses applied to the sample; (2)the relation between volume changes and pore pressure increments for the boundary condition introduced by the membrane used to cover the sample. The first relation was obtained using cyclic shear test data on dry sand published by Silver and Seed. The second one was obtained increasing the back pressure in saturated samples confined isotropically. Typical results for these back pressure increments, u', are plotted in Fig.1. In the sketch the

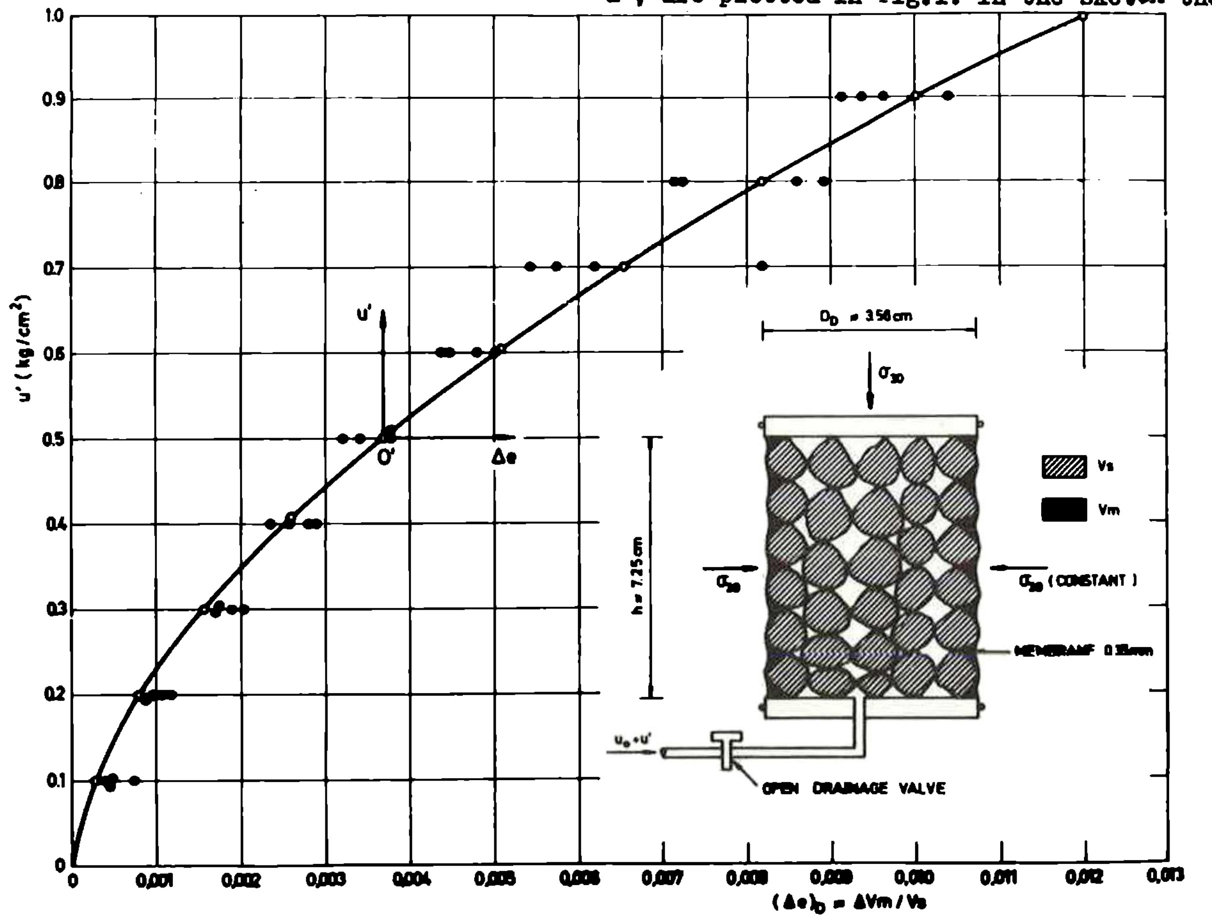

Figure 1. Void ratio increment of an isotropically confined sample as a function of water backpressure. Drained test on medium sand, DR = 45 – 50 °/o.

black area represents water pushing out the membrane as long as the pore pressure increases, which is equivalent to a "volume increment" of the sample. Because cyclic triaxial tests are conducted under undrained conditions, the "volume increment" has to be equal to the compaction of the soil skeleton. By using analytical relations to represent those volume changes in terms of sand characteristics, cyclic shear stresses applied to the sample, confining pressure, number of cycles, membrane rigidity and pore pressure, it is possible to predict tests results.
A plot of $u'/\bar{\sigma}_{30}$ vs N/N_f is shown in Fig.2 for a fine sand, where $\bar{\sigma}_{30}$=initial confining pressure, N= number of cycles and N_f=number of cycles to get failure (failure is defined for a condition $\bar{\sigma}$ =0); n, m, K in this figure represent soil properties and the parameter B the membrane rigidity. The shaded zones correspond to experimental results.

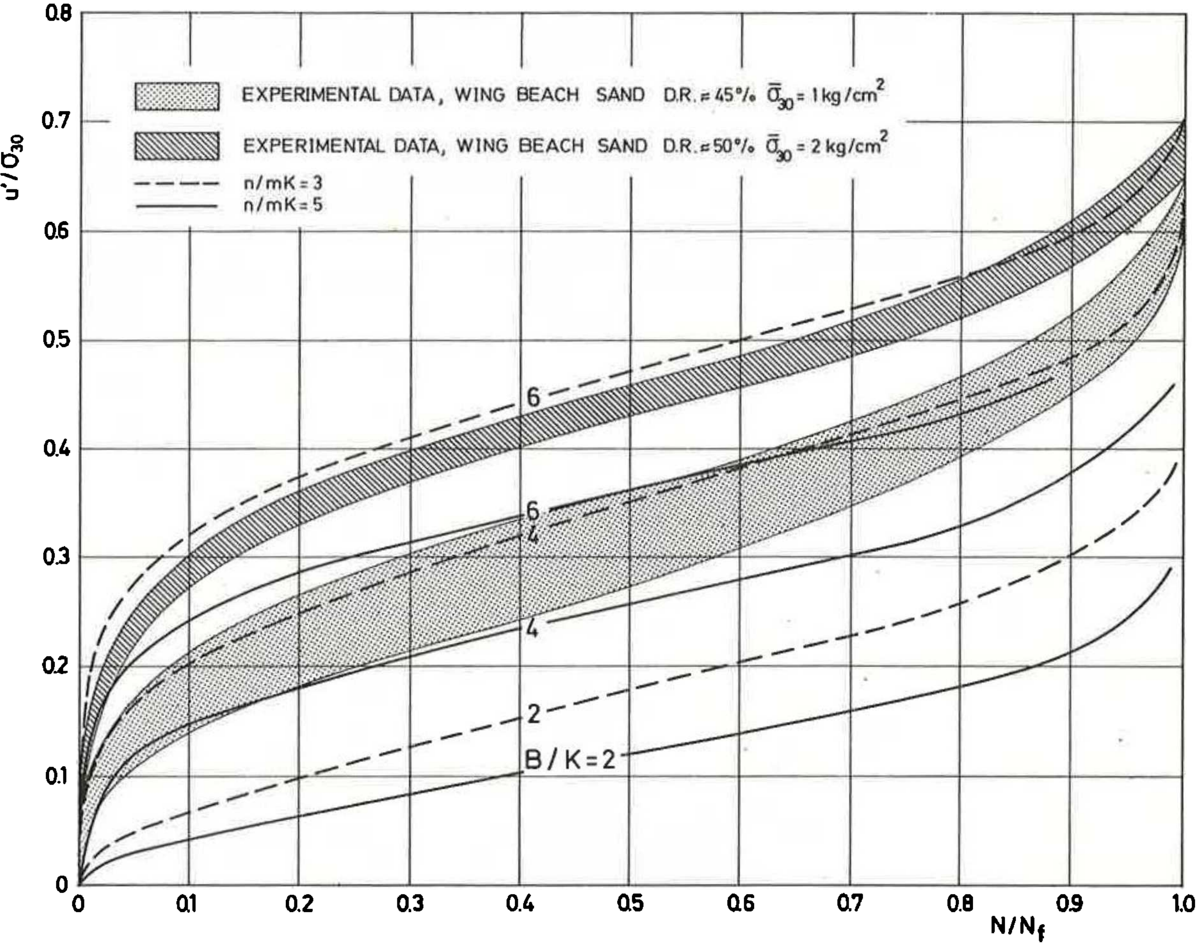

Figure 2. Theoretical and experimental curves giving pore pressure increment versus number of cycles for a cyclic triaxial test.

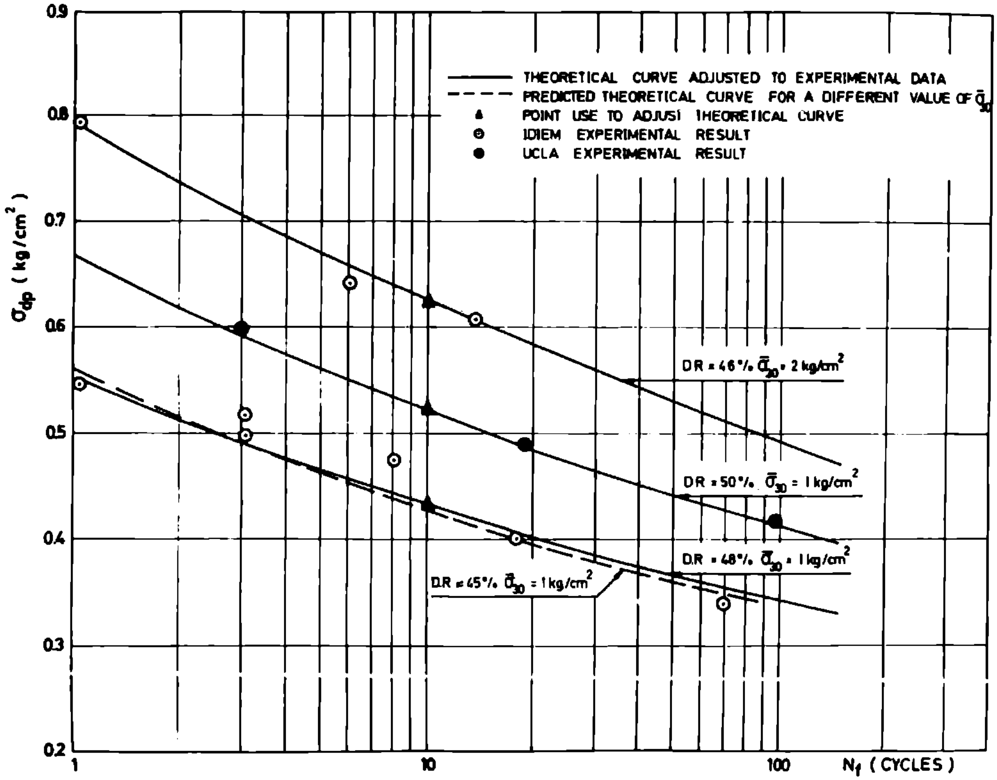

Fig.3.Curves of cyclic deviator stress versus number of cycles to failure.For different sets of experimental results theoretical curves are given,both "adjusted" for a given value of $\bar{\sigma}_{30}$ and "predicted" for a different value of $\bar{\sigma}_{30}$.

In Fig.3 it has been plotted for the same sand the deviator stress σ_{dp} vs N_f. The white circles represent experimental data obtained at IDIEM using samples with the same relative density but different values of $\bar{\sigma}_{30}$ (black points are data from UCLA). The dashed line is a curve predicted using the σ_{dp} and N values associated to one point of the curve $\bar{\sigma}_{30}$=2 kg/cm^2.

The "theoretical" approach presented here which includes the membrane effect,checks pretty well with the test results. The writer thinks that a similar "theoretical" approach could be used for predicting the liquefaction potential for a saturated sand stratum in the field. Actually this type of approach has been already used by Professor Maslov in the USSR.

Chairman Prof. Sh.Prakash (India)
Thank you Prof.Ortigosa.
The next speaker will be Mr. Menard from France.
Mr. Menard will you please.

Mr. M.Menard (France)

Nous avons developpe une technique de consolidation dynamique qui consiste a faire tomber des masses de 10 a 40 t avec des hauteurs de chute de 10 a 40m;cette technique permet d'ameliorer les sols sur une grande epaisseur, 10 a 30 m.

Or cette technique agit sur le sol d'une maniers tout a fait analogue a une serie de tremblements de terre; aussi les observations que nous avons effectue sur de dizains de chantiers ont un interet scientifique de caractere general.

La surface du sol est donc soumise a une series d'efforts dynamiques atteignant 200 t.M.a 2.000 t.M (par coup), les chocs etant concentres en des points particuliers distants de 4 a 8 m on au contraire regulierement repartis a la surface du terrain. Les observations effectuees sont les suivantes:

1° Les impacts de l'ordre de 5 a 10 millisecondes provoquent des trains d'ondes,onds de Raylaeigh de grande amplitude presentant les caracteristiques suivantes:

- Frequence 3 a 10 Hertz
- Amplitude: decroissante en fonction de la distance selon une fonction 1/r- (5 a 100m a 30 m de distance).
- acceleration: decroit tres rapidement avec la vitesse 10 a 20g (impact),quelques centiences a 30-40 m.
- l'amplitude de la vibration est une fonction lineaire avissante de la hauteur de chute du film jusqu'a 5-6m de hauteur puis reste ensuite constante quelque soit la vitesse d'impart.

- la vitesse de propagation de l'onde dans la souche compressible est tres faible pres de l'impact (20-30 m/sec), forte (200-400 m/sec) a 50 m de distance.

2° Des que l'energie transmise au sol atteint un seuil qui selon la terrains varie de 5 a 20 t.M/M3 on observe une liquefaction totale du terrain; la pression intentitielle croit et attend la valeur maximum possible c'est-a-dire la pression verticale; le terrain se comporte comme un liquide visqueux sans aucune resistance.

Tous les terrains satures ou proche de la saturation sont liquefiables, mais les plus sensibles sont les sables en la limons.

3° Ce phenomene s'accompagne d'un tassement immediat important, de l'ordre de 2 a 5% de l'epaisseur de la couche compressible consideree et de resultat est valable pour tous les terrains quaternaires; ceux-ci ont en effet la particularite (saur peut-etre les argiles sensibles dont nous n'avons pas l'experience), de posseder un pourcentage non negligeable de zag (quelque %) provenant de l'evolution chimique des matieres organiques.

4° L'eau et le gaz mis sur pression s'echauffent vers la surface en emprentant les fissures crees dans le terrain au moment de l'enfoncement du pilon; il en resulte souvent l'apparition de geysers, et des emanations de gaz (eau artesienne).

5° La resistance du terrain devient quasi nulle immediatement apres le choc, la diminution pouvant atteindre 80 a 90% de la resistance originelle; apres un periode de temps qui en fonction de la granulometrie du terrain, la resistance evolue dans le sens d'une croissance tres importante, (100 a 150% d'augmentation par rapport a la valeur originale).

6° Le phenomene de liquefaction est d'autant plus facile a obtenir que la densite relative du materiau est faible, l'on peut considerer comme regle de base que l'energie necessaire pour liquifier le sol decroit avec la densite relative.

7° Il est apparu que certaines technique de sol tel le standard Penetration Test sont de veritable essai de liquefaction du sol; on observe une destruction de la structure du materiaux sur la pointe s'accompagnant de la creation de forte pressions intentielle.

8° Apres un traitement de consolidation dynamique le terrain, pour une meme energie applique, devient de moins en moins sensible au phenomene de liquefaction. D'ou l'interet de cette technique lorsque l'on doit fonder des ouvrages tres important sur des sols sensibles au phenomene de liquifaction: reservoirs de methane liquefie, centrales atomiques-barrages en terre ou en enrochement.

9° Nous avons remarque sur de nombreuses autoroutes, que les vibrations provenant du passage de vehicules lourds, provoquaient une augmentation appreciable du tassement (de 30 a 60%) s'ajoutant au tassement normal sa obtanu sous le poids du propre du remblai et reaffernant a partie de la mise en service. L'on peut eviter ce phenomene, defavorable pour la bonne tenue de la route, en traitant le terrain au prealable par consolidation dynamique, les energies requerees par m^2 etant de l'ordre de 50 a 100 t.m. pour une circulation normale et 100 a 150 t.m. pour une circulation tres dense.

Chairman Prof.Sh.Prakash (India)
Thank you Mr.Menard.
Now I pass the word to Prof. Yoshimi (Japan)
Prof. Yoshimi, will you please.

Prof. Y.Yoshimi (Japan)

In my discussion, I would like to present some results of our recent study of liquefaction of level, sandy ground (Yoshimi and Kuwabara, 1973). In our problem, a soil stratum at some depth, marked II in Fig.1 , is completely liquefied during an earthquake. Subsequently, the excess pore water pressure in the liquefied stratum is dissipated through the overlying stratum, marked I.

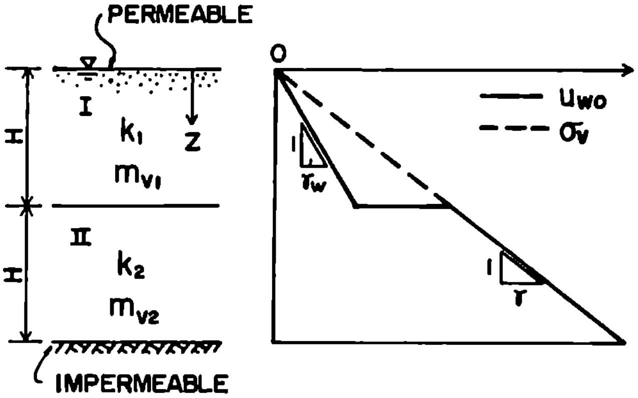

Fig.I

The Terzaghi consolidation theory was applied to the problem to determine the pore water pressure as a function of time and depth, and numerical solutions were obtained for a variety of initial and boundary conditions using the finite element method. Thus, our study is an extension of the work of Prof. Ambraseys (Ambryseys and Sarma, 1969). The initial pore water pressure distribution for the simplest case is shown in Fig.1, in which k is the coefficient of permeability, and m_v denotes the coefficient of volume change. In Stratum II, the pore water pressure undergoes nearly monotonic decrease with time. In Stratum I, on the other hand, the pore water pressure first rises, and then falls after reaching a peak value. The peak pore pressure corresponds to the minimum effective stress which is related to the stability of foundations located within the stratum.

Fig.2 shows the ratio of the minimum effective stress to the initial effective stress, plotted against the ratio of the coefficient of permeability, k_1/k_2, and the ratio of the coefficient of volume change, m_{v1}/m_{v2}. As one would expect, the minimum effective stress decreases as the surface soil gets less permeable and less compressible.

In a situation where soil is susceptible to liquefaction to a depth of, say 20m, it will be eonnomically desirable if we need not compact the soil to the full depth. But if we want to compact only the top half, allowing the bottom half to liquefy, we must make sure that the surface soil retains sufficient strength during the upward seepage. Two methods of compaction are compared here. Point A in Fig.2 is for a case in which the vertical permeability of the surface soil is increased in the process of densification-for example, by vibroflotation or the sand compaction piles using coarse backfill. The minimum effective stress is 22% of the initial value in this case, which makes it possible to design shallow foundations. On the other hand, Point B corresponds to a case in which the surface soil is densified without changing the grain size. In this case, the minimum effective stress is only 4% of the initial value, and the soil is not suitable to support a foundation.

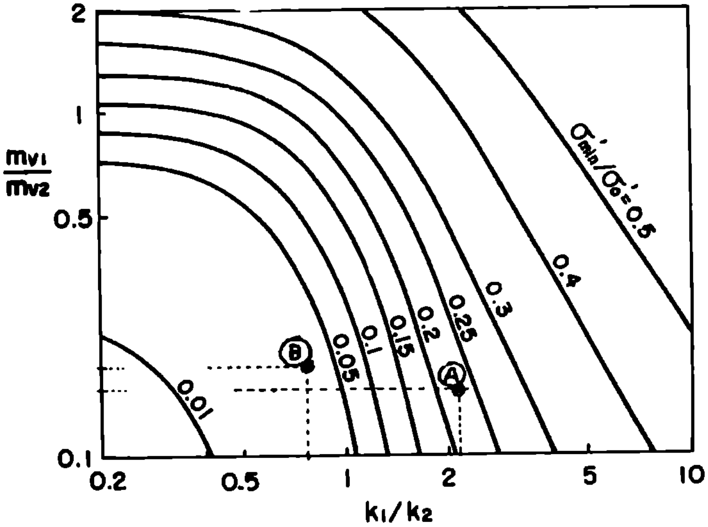

Fig.2

The effectiveness of the vibroflotation method in minimizing damage due to liquefaction was witnessed during the Niigata earthquake of 1964 (Watanabe, 1966) and the Tokachioki earthquake of 1968 (Ohsaki, 1970). There are indications that the effectiveness can be better explained by the drainage effect than the densification effect.

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Chairman Prof. Sh.Prakash (India)
Thank you Prof. Yoshimi.
The next will be Aoopian from the USSR

Mr. Acopian will you please

Mr. Acopian K.A. (USSR)

The work considers the questions of increasing of construction seismostableness from the point of reducing of wave effect of seismic loading with the help of shielding layers.

The aim of this investigation is to determine the optimum parameters of the shielding layer fixed on the way of seismic wave in order to reduce the strength of seismic wave blow on construction in many times due to repeated reflections within the layer and due to the material characteristics of the shield.

This investigation of wave spreading in layer plates helps to clerify the question of effectivness of shielding layer work which estimated by the coefficient of longitudinal waves passing.

The investigations showed that the amplitude for all considered layer thicknesses considerably reduced and the configuration of passed impulse differs from that one which falls.

The shielding effect of layer in respect of the wave passing normally towards upper edge of the layer consists of the following factors: firstly, the wave configuration sufficiently distorts and secondly, the intensity (amplitude) of wave reduces.

The quality analysis of films relfecting the interference pictures of stripes shows that the increasing of the layer thickness as well as the angle of falling impulse reduces the impulse intensity.

In case when acoustic rigidness of layer is more than the rigidness of main surroundings the amplitude considerably reduces for all considered layer thicknesses, and the configuration of passed impulse changes. The thin layers stretches the impulse in time and the layers of inter in capacity shortens the passed impulse in comparison with the falling one.

In contrast to the case of highmodule layer the form of passed impulse is analogical to the form of falling one. In this case the presence of the layer reduces only the size of passed impulse amplitude and shortens the time of conciseness phase not changing the form of the pulse at the same time.

Experimental material in films of interference pictures of stripes received in the result of this experiment helped to get the rates of coefficients of wave passing for the cases of different correlation of layer acustic rigidnesses and for the main surroundings.

Received regularities of the coefficient changes of passing of the longitudinal wave helps to estimate the efficiency of the chielding layers.

It shows also that the amplitude of passed wave under different falling angles changes slightly after passing the layer.

The coefficient of wave passing through the layers shows that from the point of reducing of wave influence the thickneing of shielding layers up to the level $\delta=0,2\lambda$ is quite effective:

- reducing upto 45% for the longitudinal wave in case of horizontal layers.
- reducing upto 60% the intensity of surface wave P.
- reducing upto 75-80% the intensity of surface wave R.

Chairman Prof. Sh. Prakash (India)
Thank you Mr.Acopian.

The next in our discussion will be Mr. Jakovlev from the USSR

Mr. Jakovlev will you please.

Mr. P.I.Jakovlev (USSR)

In modern practice soil pressure on the retaining walls under seismic conditions is commonly determined on a basis of Coulomb's theory. Engineering standards of many countries permit additional simplifications, the essence of which is as follows: the existing soil pressure is assumed equal to a product of a corresponding value acting at conditions excluding seismic activity and a certain factor which depends on the strength of an earthquake. Thus, there are even no attempts made to take into account the variations of kinematic and static phenomena connected with seismic forces origination and apply the data for the problem involved at given boundary conditions. This practice does not conform to the present state of loose medium theory all the more that the error of Coulomb's method reaches sometimes, for example, when determining passive coil pressure, more than 100%. Taking into account seismic influence these errors can be even higher.

The accuracy of engineering calculation under seismic conditions can be significantly improved using the modern theory of safe stress state. This theory was developed, first, by Rankine, Kötter, Prandtl and then completed by soviet scientists prof. V.V.Sokolovsky and prof. S.S.Golushkevich.

Currently, on a basis of the theory of safe stress state prof. F.M.Shikhiev and the author of the presented report (1973) elaborated a method of engineering calculation of soil pressure in back-fills. This method takes into account seismic forces in a general case of the retaining wall with inclined rough back side and inclined surface of back-fill under uniformly distributed load. The solution obtained using curvilinear surfaces of sloping contains finite expressions for the dimensionless factors of active and passive pressure which were singled out. In the present time the computers are used to compile the tables of the factors for different wall and back-fill surface angles of slope, coefficients of the angles of inner friction of soil and coefficients of friction between soil and wall surface. These tables can efficiently simplify the process of calculation. Solutions are also obtained for the case of gently sloping walls and walls with relieving slabs (Jakovlev, 1971). The results proved that on a basis of this theory seismic forces can be taken into account both in case of the walls with broken back side or broken surface of back-fill and in any other cases with complicated boundary conditions (Jakovlev, 1966, 1973; Shikhiev, Jakovlev, 1972).

It should be noted that the theory of limiting balance treats equally all problems arising in the process of interaction between structure and soil, which is its great advantage. In the USSR on a basis of this theory the seismic activity can be taken into account more accurately while determining bearing capacity of foundations, stability of slopes and other problems (Shikhiev, Jakovlev, 1968).

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Chairman Prof. Sh.Prakash (India)
Thank you Mr.Jakovlev
The last in our contribution will be Mr.Bourges. Mr. Bourges will you please.

Mr. F.Bourges (France)

L'augmentation des charges et de la vitesse des convois sur les voies passant sur des remblais edifies sur des sols compressibles peut conduire a des deflexions incompatibles avec l'exploitation normale de la voie. Ce probleme se pose pour les voies nouvelles des trains a grande vitesse traversant des zones marecageuses. Il se presente aussi sur les anciennes voies qui ont supporte sans dommage un trafic moyen pendant plusieurs decennies et qui ont subi des deformations tres superieures a celles admises habituellement (qui sont de l'ordre de 2mm) lorsqu'on y a fait passer des trains lourds atteignant des vitesses de 120 a 140 km/heure.

Dans le Nord de la France, pres d'Abbeville, nous avons recemment etudie le comportement d'une ancienne voie, sous l'accroissement du trafic en charge (utilisation de motrices a 6 essieux de 15 a 20 tonnes chacun) et en vitesse (passage de 80 km/heure a 120 km/heure).

La voie repose,par l'intermediaire d'un ballast de 1 m d'epaisseur, sur une couche de 10 m de sol tourbeux (melange heterogene d'argile,de sable et de tourbe,dont la teneur en eau varie de 100% a 500%). Sous cette couche compressible, on rencontre la craie alteree, peu compressible,sur 10 m d'epaisseur environ, puis la craie franche a 20 m de profondeur.

L'observation du phenomene a consiste a mesurer au passage d'un convoi les deplacements verticaux, a differentes profondeurs sous la voie, ainsi que les contraintes dans le rail. Nous nous contenterons de donner ici quelquesuns des resultats les plus caracteristiques de ces mesures et d'en tirer quelques premieres conclusions. L'interpretation theorique, en cours actuellement, permettra de mieux cerner le phenomene et, nous l'esperons, de proposer des solutions.

L'experience a ete conduite avec une machine a 2 boogies, chacun d'eux comportant 3 essieux de 17,5 tonnes. On a fait varier la vitesse de la machine de 40 km/heure a 120km/heure. Les deplacements ont ete mesures a differents niveaux par rapport au rail: 0,2,50 m, 4,50 m et 8 m de profondeur. Les contraintes dans le rail ont ete determinees a l'aide de jauges de deformation collees sous le patin du rail.

Les principaux resultats peuvent se resumer ainsi:

1. La deflexion maximale mesuree en surface croit bien sur avec la vitesse (figure 1) mais on s'apercoit qui meme aux tres faibles vitesses,cette deflexion doit etre importante (alors qu'a vitesse nulle,c'est-a-dire pour la charge statique, elle est tres faible). A 40 km/heure, on a deja une deflexion de 19 mm soit 77% de celle mesuree a 116 km/heure. La vitesse critique conduisant a une deflexion prohibitive est inferieure a 40 km/heure. Avec de telles machines, on peut dire que le probleme existe meme aux faibles vitesses. Notons que la deflexion est entierement reversible et s'annule apres le passage de la machine.

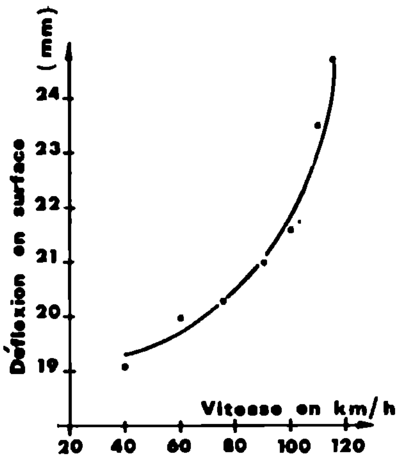

Fig. 1

2. On constate (figure 2) que,par rapport a la deflexion en surface, on n'a plus que 33% de cette deflexion a 4,50 m de profondeur et 10% a 8 m. On peut conclure que la demi couche superieure de tourbe encaisse la plus grande partie des deformations.

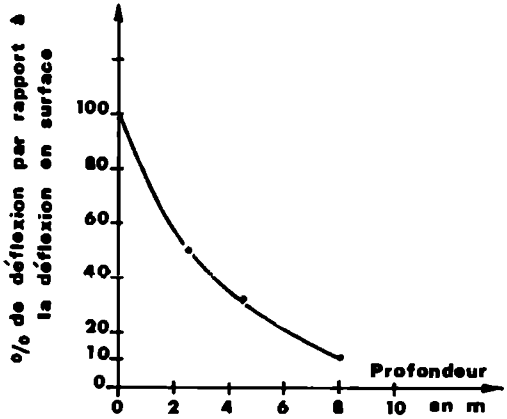

Fig. 2

Or, l'etude de la couche tourbeuse n'a pas permis de deceler une amelioration des caracteristiques de la tourbe en profondeur. On note que les deflexions mesurees en surface sont plus de 10 fois celles qui sont couramment admises.

3. Les contraintes mesurees dans le rail sont tres fortes: 10 a 12 hbars.

Les mesures ont fourni la ligne d'influence des differents essieux. On recherche actuellement un modele bi-couche representant les deux demi-couches de tourbe: 0 a 4,50 m et 4,50 m a 10 m, avec des modules respectifs E_1 et E_2 differents ($E_1 < E_2$): en ajustant E_1 et E_2, on essaiera de retrouver la ligne d'influence experimentale. On etudiera ensuite la variation de cette ligne d'influence avec E_1 et l'epaisseur h_1 de la demi-couche superieure. On pourra ainsi definir l'epaisseur minimale a traiter et la resistance mecanique (representee par le module E_1) a obtenir par ce traitement pour parvenir a des deflexions acceptables. Le module statique non draine de la couche de tourbe est de l'ordre de 12 bars.

Chairman Prof.Sh.Prakash (India)

Thank you Mr. Bourges for your discussion.

Professor Barkan, Professor Sinitsyn, ladies and gentlemen,
You have participated in very stimulating discussions this after-noon on a subject which is still in its infancy. The main questions which need further investigations may now be listed:
(i) Effect of vertical component of ground motion on the stability of high earth and rockfill dams.
(ii) Dynamic stresses in soils due to fast moving trains.
(iii) Evaluation of cushions for vibration isolation of machines
(iv) Development of simplified procedures for evaluation of liquefaction.
(v) Liquefaction of soils on slopes.
(vi) Stochastic approach to solution of liquefaction.
(vii) Limitation of existing methods e.g. vibration table studies, cyclic triaxial and oscillatory pure shear tests, and correlation of results of one type with the other.
(viii) Need for newer interpretation of existing test data on liquefaction.
(ix) Field study of liquefaction phenomenon.
(x) Vibration transmission due to collapsing structures
(xi) Soil testing by resonant column method
(xii) Use of holography for horizontal and vertical displacements in soil dynamics studies.
(xiii) Dynamic compaction of soil upto 10 m to 30 m depths to improve their properties.
(xiv) Liquefaction of stratified deposits.

I believe it is a good inventory of problems to work upon till we assemble again in Tokyo for IX International Conference. I am sure you would support the idea of recommending to the Conference to allot 1- main session to the question of "Liquefaction" of soils. This is clearly indicated by the interest shown by the house in this topic.
And finally, although to-day is Friday (TGIF, thank God it is friday), please do not oversleep. We assemble to-morrow morning for the concluding sessions. Before we disperse, I shall like to extend our hearty thanks to the Organizing Committee for excellent arrangements for this session, to Dr.Barkan and Professor Sinitsyn, our co-chairmen and Mrs.Medvedeva our Secretary.

VIBRATIONS OF EMBEDDED FOOTINGS.

M. Anandakrishnan and
N.R. Krishnaswamy (India)

INTRODUCTION

A single-degree-of-freedom mass-spring-dashpot analogue with the inclusion of a Coulomb friction damper has been proposed to describe the dynamic response of an embedded footing. A simple procedure by which the constant frictional force of the Coulomb friction damping can be evaluated has been described. The report, in its present form, deals exclusively with vertical vibrations.

THE PROPOSED ANALOGUE

The vertical vibrations of a rigid footing embedded in the soil can be approximated, for example, by the simplified analogue suggested by Lysmer (3), which in principle apply for a footing placed on the surface of soil. With the addition of a Coulomb friction damper F, the differential equation of motion for an embedded footing can be written as,

$$M X + C X + K X \pm F = Q(t) \qquad (1)$$

where the coefficient of viscous damping,

$$C = (3.4 r_o^2 \sqrt{\rho G}) / (1 - \mu)$$

the spring constant,

$$K = (4 G r_o) / (1 - \mu)$$

the constant frictional force of the Coulomb friction damping that is likely to be mobilized as a result of embedment = F, the mass and equivalent radius of the base of the vibrating footing are denoted by M and r_o respectively. G, μ and ρ = shear modulus, Poisson's ratio and mass density of the soil below the footing.

A steady-state solution of Eq. 1 for the case of a constant sinusoidal exciting force was given by Den Hartog (1). The steady-state solutions for the cases of exciting force due to eccentric rotating masses and constant type of exciting force has been presented by Krishnaswamy (2) in a convenient form suitable for purposes of foundation design and analysis. The solution of Eq.1 under transient excitation is possible by techniques such as the phase-plane analysis. The solution procedure and the analytical expressions are not presented here, but can be found in Ref.(2). However, the final results for the case of rotating-mass-type of excitation are illustrated in Figures 1 and 2, for various damping factors, D.

The proposed theoretical model can predict the dynamic response of an embedded footing provided the values of M, r_o, G, μ, ρ and F are known for a given footing and soil conditions. Accepted procedures are available to estimate all these parameters except the constant frictional force, F. A simple procedure is described below by which, F can be estimated for an embedded footing.

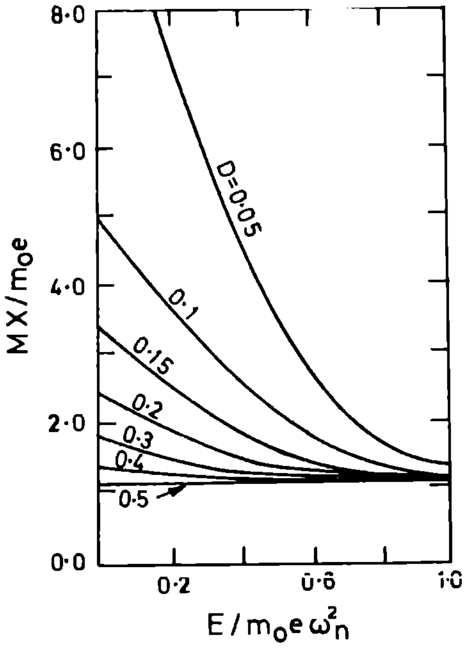

FIG. 1 DECREASE OF MAXIMUM AMPLITUDE AT RESONANCE WITH COULOMB FRICTION FACTOR

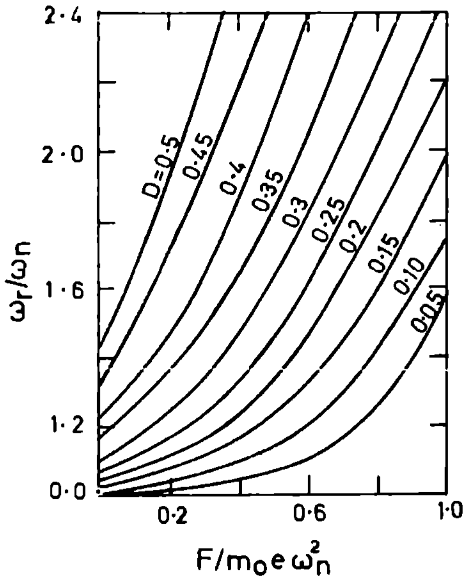

FIG. 2 INCREASE OF RESONANT FREQUENCY WITH COULOMB FRICTION FACTOR

EVALUATION OF CONSTANT FRICTIONAL FORCE

Experimental evidence from Novak(4) and Krishnaswamy (2) suggests that the dynamic response of an embedded footing depends on the physical characteristics of the interface between the walls of the footing and the surrounding soil besides the lateral earth pressure acting normal to the surface For a footing embedded in a general 'c-ϕ' soil, the total amount of constant interfacial friction that would be mobilized during vibrations can be written as,

$$F = (\tfrac{1}{2} K_o H^2 \gamma \mu_f + C_a H) L \qquad (2)$$

where K_o = coefficient of lateral earth pressure at rest, γ = bulk density of the surrounding soil, H = depth of embedment, μ_f = coefficient of kinematic wall friction between soil and foundation walls, L = perimeter length of the footing, c and ϕ = cohesive strength and angle of intergranular friction of the surrounding soil, respectively.

It has been shown by Krishnaswamy (2) that a good correlation between the predicted and observed response of massive concrete footings embedded in soils is possible by adopting the values of $K_o = 0.4$, $\mu_f = \text{Tan}\ (\phi/3)$ and $C_a = 0.01\ c$, in Eq. 2.

CONCLUDING REMARKS

Steady-state vibration tests on massive concrete footings reported by Krishnaswamy (2) indicated a decrease of maximum amplitudes of motion and increase of resonant frequencies as a consequence of embedment. This trend is in agreement with the proposed theoretical model. The correlation between the field test data and the predicted values by the proposed theoretical model is satisfactory.

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INVESTIGATION OF PILE FOUNDATION UNDER DYNAMIC AND SEISMIC LOAD.

Barkan D.D., Megevoy G.N., Mongolov Ju.V., Stavnitser L.R. Shaevich V.M., Shekhter O.Ya. (USSR)

The main defect of the driven piles lies in the small resistance to the horizontal loads which can be large during the earthquake. Therefore the piles needed to resist the seismic horizontal loads are taken often in much more quantity as compared with the vertical loads determined by calculation So these piles are far from saving economy.

The improving of pile behaviour concerning the horizontal forces may be reached either by using the piles with great resistance in horizontal direction, or by reducing the horizontal loads on the pile. For the first purpose the piles with considerably extended cone part may be employed and it appears to be expedient when the relatively short piles are used. For reducing the horizontal loads transmitted to the piles it is necessary to eliminate the rigid connection between the piles and foundation, and instead of that to place between them an intermediate soil cushion, made of well compacted sand crush stones, gravel or the soil taken in situ. With this arrangement the horizontal loads transmitted to the pile are substantively reduced because the surrounding ground participate in bearing capacity.

The study of 25 test piles with the intermediate cushion was carried out on the loess loam in situ on a large scale.

Besides this 15 model pile foundations (scale 1:5) have also been tested under different soil conditions. The results of these tests show that the static and dynamic horizontal pile displacements and hence the horizontal forces on the piles are considerably reduced with increasing of the cushion thickness. With the thickness equal to 40cm the horizontal pile displacements (as well as the loads transmitted to the piles) are equal to the 0,25-0,40 of horizontal foundation displacements values (the same is for the loads) The pile length and the cushion material do not influence considerably on the horizontal stresses in piles and their displacements. Of course the extension of cross section dimensions reduces the horizontal displacements. The horizontal vibration as it was established has a very little effect on the pile settlements.

The experiments with vertical pile loadings made it possible to establish the character of the load distribution between the piles and the ground and the influence of the cushion thickness, its material, pile length and its cross section on the elastic and plastic settlements. The results of these tests have permitted to establish the approximate methods for pile designs and their testing. At the present time these foundations with intermediate cushion are used for buildings under different soil condition of the seismic regions.

The study of pile rigidity has a great significance for antiseismic designs. Therefore the set of the tests on different soils were carried out in situ. The driven piles were tested by applying upon them the static loads, forced and free vibrations. The results have shown that the relation between the elastic horizontal displacements and the load is far from being linear and is of soft type character. Thus the value of the rigidity coefficient must be corresponding to the limits of the known displacements. For the seismic conditions the assymptotic values must be used. By the use of the tensopiles it was established that the elastic soil pressure increases with depht in accordance with the non linear law having a greater value for the clay soils that for the sand soils. But the natural frequencies and hence the horizontal rigidity of pile are influenced by pile depht only when it does not exceed 4-5m and they diminish with the depht in this range. Further it was established that the horizontal rigidity coefficient in a great degree increases when the distance between piles diminishes. In time it also increases significantly. But when there is a free part of a pile (above the ground) this coefficient became much smaller and for instance when the free part is equal to one meter the natural frequency does not exceed 20% of the value corresponding to the pile completely embedded into the soil.

The theory for a cylindric pile in a soil subjected to the plane horizontal harmonic waves has been developped. The vertical load from a building is supposed to be a concentrated mass applied to the upper end of the pile. This end is rigidly connected with a raft foundation. The approximated dynamic calculation of a pile is developped on the basis of a discrete model by dividing a pile into the equal parts. The pressure applied to the pile from the seismic wave is represented by a step function. The problem of finding out the forces acting on the each part of the pile is connected with the solution of a dynamic plane elastic problem about the interaction between the elastic waves and the rigid inclusion embedded into a soil. This inclusion represents the pile cross section. Thus we have deduced the expressions for dynamic displacements, bending moments and transversal forces as functions of the depth and time. The results of the calculations show that the pile length has a little influence on the above ground building vibrations.

ON AN EXTRA-QUICK SOIL OF VOLCANO-SEDIMENTARY ORIGIN.

Cozzupoli D.,R.Mortari (Romania)

SUMMARY- In the Vulsinian volcanic district (Latium, Italy) the presence of a number of clastic dykes has been related to the liquefaction of an extra-quick soil.The assessment of some geotechnical and mineralogical properties has pointed sensitivity values as being particularly high, this owing possibly both to the presence of diatom shells in the sediment and to the low percentage of true clay minerals in the fraction finer than 2 microns.

In the Vulsinian volcanic district (Central Italy) lacustrine sediments are found intercalated to the local volcanites.These sediments-mainly composed of volcanic ashes and diatom shells- in the pumice quarries of Case Collina constitute clastic dykes which cross the mass of the pumices. The peculiarity of this field situation suggested to the authors an investigation of the technical and mineralogical properties of the original sedimentary material, in the attempt to explain the formation of these structures.

The original sediment of these dykes was recognized near the edge of an ancient lacustrine basin, under a pumiceous bank. It often shows a clear stratification because of slight grain size variations; it can be defined as a sandy silt with a small "clay" fraction. The percentage of the particles larger than 2 mm may reach 4 ÷ 5 % in the coarsest levels; between 2 and 0.062 mm, the particles were found to be about 40 %, between 62 and 2 microns about 50 %, while the fraction finer than 2 microns was generally represented by less than 10 %.

For what regards the mineralogical composition of this sediment, in the fraction coarser than 2 microns we found: feldspars (sanidine and plagioclase), clinopiroxenes (prevailing diopsidic and egirinaugitic), garnets, vesuvianite, biotite, anphibole, zeolites (mostly analcime), titanite and magnetite; in the larger sizes there are frequent glass **fragments with a pumiceous structure and lithic fragments that can be related to volcanites of various types. In the fine and medium silty fraction we also find numerous diatom shells.**

Though the particles finer than 2 microns did not show evident crystal features at a diffractometric analysis, they enable us to recognize a presence, even though a scarce one, of clay minerals, concerning the groups of montmorillonite, illite and halloysite. This has also been confirmed by the results of the thermal analysis which pointed out a presence of clay minerals not higher than 20% of this fraction, including incidental clay amorphous species. The remaining 80 % of the "clay" fraction is formed by not yet argillified glass fragments.

This soil is found in situ to be saturated at 100 %, with low densities ranging from 1.20 to 1.24 g/cm^3, a porosity of nearly 80 % and a moisture content ranging from 134 % to 146 %; some levels, a few mm thick, may show higher moisture contents up to 160 %. The ranges of the Atterberg limits (see table 1) are:

$w_L = 63 \div 69$, $w_P = 46 \div 49$, $IP = 17 \div 22$.

In two samples the percentage of "clay" fraction resulted 7 % and 10 %, when the analysis was performed so as to preserve the shells of the diatoms; such percentage rose up to 12 % and 15 % respectively, when on the contrary the material was remoulded with the fingers for a long time.

In fig.1 we reported the cumulative grain size curves of sample n.9a obtained with the two different treatments: we can see the curves coinciding for values higher than about 10 microns and splitting only for lower size values.

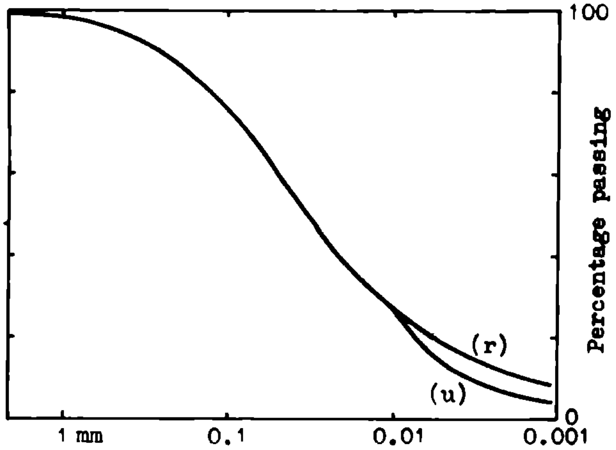

Fig. 1 - Grain size curve of sample n. 9a with (r) and without (u) breakage of diatom shells.

In order to calculate the values of activity we referred to the lower percentage values of the "clay" fraction, assuming that plasticity comes only from the fraction finer than 2 microns being present in the samples before their remoulding. In fact samples of pure diatomites, like those of Riano, North of Roma, result to be not at all plastic; according to that the silica of the shells of these algae behaves like quarz that has no plasticity in the finest sizes (von MOOS, 1938) and therefore has no activity (SKEMPTON 1953). In this case, for samples n. 7a and 9a we obtained the two activity values 2.2 and 2.4 which are very high and certainly not due to the clay minerals, which appear in the samples in a very low quantity. Such activities are therefore due almost completely to the **tiny volcanic glass fragments being in the fraction finer than 2 microns.**

It is to be pointed out that deposits of volcanic origin are frequent in Latium and they are considered as tuffs, showing a clear plasticity, activity values like those described, and high sensitivity to remoulding.

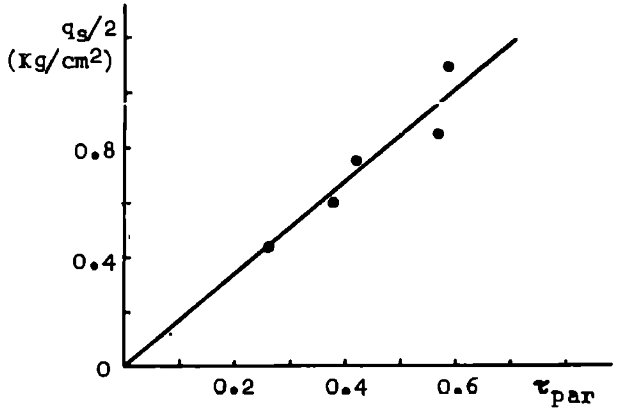

Fig. 2 - Correlation between unconfined compression strength and strength parameter.

In order to determine the sensitivity of the examined soil, we measured the shear strength on undisturbed samples showing the highest uniformity, using unconfined compression tests. From comparison with the results obtained by the pocket penetrometer, we noticed both the uniformity of the tested material and the coincidence of the values obtained in the two kinds of tests. That allowed us to extend the measures, by the pocket penetrometer, **evento those levels not sufficiently thick to carry on the unconfined compression tests. For those levels too thin to be tested** even by the pocket penetrometer, we used a fall-cone weighing 400 grams with a cone angle of 30 degrees; in fig 2 diagram the strength parameter τ_{par} (HANSBO, 1957) of fall-cone tests are compared with the results of unconfined compression and pocket penetrometer tests. The uniformity of the ratio

$$k_{30} = \frac{q_s/2}{\tau_{par}} = \frac{c_u}{\tau_{par}}$$

between the two series of values, equal to 1.7, justifies the use of the fall-cone. In such a way we could compute c_u maximum values equal to 2.0 Kg/cm^2 in a finer grain size level with a moisture content of 160 %, into which the cone penetrated repeatedly for 4.5 mm.

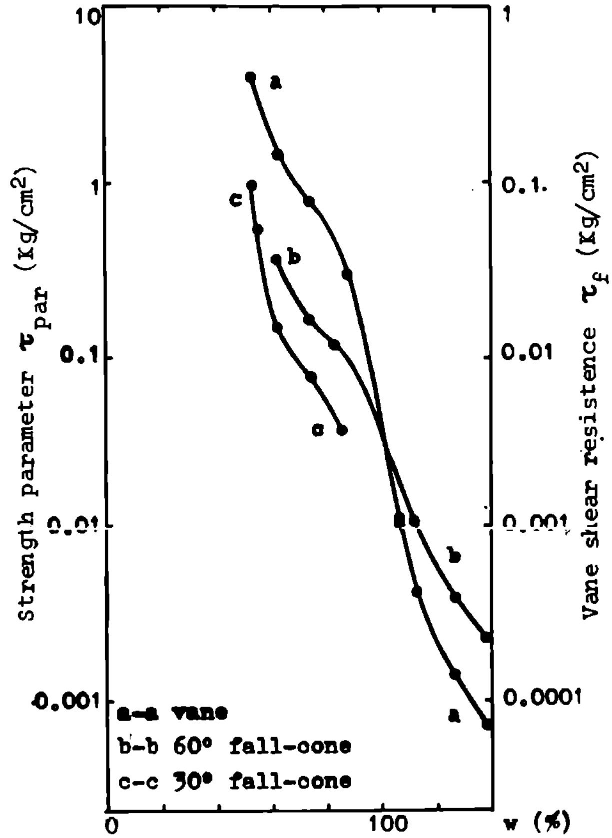

Fig. 3 - Consistency curves from vane and fall-cone tests at various moisture contents for sample n. 9b.

The shear tests were always performed perpendicularly to the stratification; unconfined compression tests performed on samples taken parallel to stratification showed a reduction of the resistence, e.g. from 0.88 to 0.45 Kg/cm^2.

The shear strength values on remoulded samples were obtained by laboratory vane tests and compared with the results of a fall-cone weighing 6.6 grams with a cone angle of 60 degrees. The shear strength in the vane test was 7.5x10^{-5} Kg/cm^2 for the sample n. 9b, with an average moisture content of 139 %; the corresponding strength parameter resulted to be 2.35x10^{-5} Kg/cm^2, with a very low ratio k_{60} of 0.03. Considering these results, the two kinds of test were repeated at lower moisture contents leaving the sample to dry in the air and without adding water. Results are reported in figs. 3 and 4. We can notice that k_{30} and k_{60} **have** very different values at dif ferent moisture contents, reaching maxima in correspondence with the liquid limit.

The remarkable fluidity in the remoulded material can at least partially be referred to the breakage of the diatom shells.

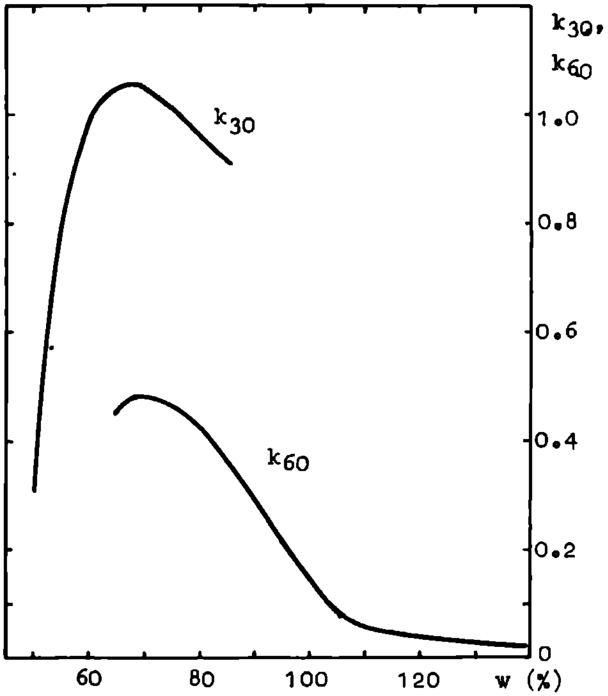

Fig. 4 - k_{30} and k_{60} values versus moisture for sample n. 9b.

Because of the difficulties due to the fluidity of the material, the falling of the cone was measured observing the mark of the material on the cone after a very quick deepening; the following table 2 shows the sensitivity values obtained in tests performed on various parts of the sample n. 9.

sample n.	7a	7b	9a
w_L	69	69	63
w_P	49	47	46
IP	20	22	17
w	139	140	134
IL	4.5	4.2	5.2
% < 2 μ (undisturbed)		10	7
% < 2 μ (remoulded)		15	12
A		2.2	2.4

Table 1 - Physic properties of samples n. 7a, 7b and 9a.

The particularly high sensitivity of the te tested soil allows us to suppose that the lake deposits lying under Case Collina pumiceous materials intruded into the upper masses in the form of clastic dykes along discontinuity surfaces, in relation to a sudden volcanic event.

It is worth pointing out here that we find quite a different sedimentation environment for the extra-quick soils, considering that these soils can generally be found in the periglacial environments. The common element of the two kinds of geological environment is the presence, of different genesis, of a high percentage of non-clay minerals of less than 2 microns in diameter.

w	c_u (Kg/cm^2) (undisturbed)	c_u (Kg/cm^2) (remoulded)	S_t
139	0.9	7.5×10^{-5}	12000
146	1.1	5.5×10^{-5}	20000
160	2.0	2.5×10^{-5}	80000

Table 2 - Sensitivity values of various parts of sample n. 9b.

It is shown that sensitivity increases, for a given sediment, with decreasing values of plasticity index (BJERRUM, 1954); thus this increase can be explained by the presence of non-clay minerals into the "clay" fraction. The extremely soft structure observed can be related to this presence.

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BEHAVIOUR OF SUBGRADE SOILS UNDER SURFACE PULSING LOAD.

M.Goldstein,L.Lapidus,O.Reznikov, V.Storojenko,N.Sinaevsky.

SYNOPSIS. The influence of the pulsing load on the shear strength of different soils and the swelling and consolidation processes was investigated. The design coefficients for the transition from the static shear strength to the weariness strength limit are established. The dynamic load action results in reducing the swelling process speed and in stopping the clay consolidation. The loamy soils are compressed only partly in comparison with the static conditions.

The investigations were conducted using the electromagnetic pulse-loading apparatus. A frequency and duraction of the loading was program controlled.

The angle of dynamic internal friction of saturated clays under triaxial UU-test was much less in static tests and often was equal even to zero. In this tests the frequency was equal to 1 herz and the loading duration - to 0,2 sec. The results remained the same under 2-3 times frequency variation.

The consolidation and swelling were investigated in the odometers. The regime was just as in the real railway subgrade under the live load action: each 200 cycles of the pulsing load were followed by the 15 minuts rest. The consistence limits of the tested soils were: the clay- w_T=58,3%; w_p=31.8%; the loam - w_T= 25,6%, w_p=15,7%.

The relative swelling was equal to 13-15% under static tests and 5-7,5% under pulsing pressure equal to 1,2 kg/cm^2. The clay swelling pressure was equal to 1,0 kg/cm^2. When the load was not more than 0,7-1,0 kg/cm^2 the swelling was going on both during the rest period and during the pulsing load action. Under the greater load the swelling took place only during the rest periods.

The consolidation of the clay was absent when the pulsing load as large as 2-3 kg/cm^2 was acted during 50-100 hours. From date received in these experiments the reliable conclusions could be drawn about the values of design parameters for the dynamic of soils analysis.

The dynamic test of loam soils showed quite different results. No swelling occured under pulsing load, in spite of great swelling under static conditions. On the contrary the consolidation of the loam under pulsing load come to an end wore quickly than under the static load. But the soil density values after the dynamic consolidation are less than after the compression under the static load of the same magnitude. It was noted that additional compression under smaller static pressure took place after the dynamics compression was finished.

Due to the short duration of the each load cycle the density of the subgrade cohesive soils don't increase. So the spherical component of the dynamic stress tensor don't influence on the internal friction and the soil resistance to the deviator depends only on the cohesion. Really, our experiments have confirmed that the angle of internal friction of clayey soils under the pulsing loads was practically equal to zero

The loading cycles number results in decreasing the unconfined dynamic strength (dynamic cohesion) diminishing to the weariness limit. The weariness limit for the loam is approximately equal to (0,5-0,6 σ_{st}) and for the clay -0,4 σ_{st} (σ_{st}-the unconfined quick strength).

Accordingly in the dynamic stability analysis of the road subgrade the dynamic soil strength must be taken equal to the weariness limit.

The ultimate bearing capacity analysis of the railroad subgrades based on the theory of limiting equilibrium and the above described data on the dynamic soil strength have shown satisfactory coinsidence with the field observations.

THE STUDY OF RESONANCE PROPERTIES OF SOIL ON BUILDING SITE. Barkan D.D.Golubtsova M.N.(USSR)

To determine the seismic properties of soils lying near to the surface and considerably influencing upon the intensity of seismic forces the experiments in situ have been carried out.For this purpose the steady state harmonic waves with different frequencies (of 4 to 20 Hz) were excited by means of the inertia type vibrator placed on the soil surface and producing the vertical vibrations. The vibrator exciting force was 5 tons.

The soil was represented by different layers of Macroporous loams of hard consistency, of sand and of sandy clayey soils.The waterlevel was below 20 m.

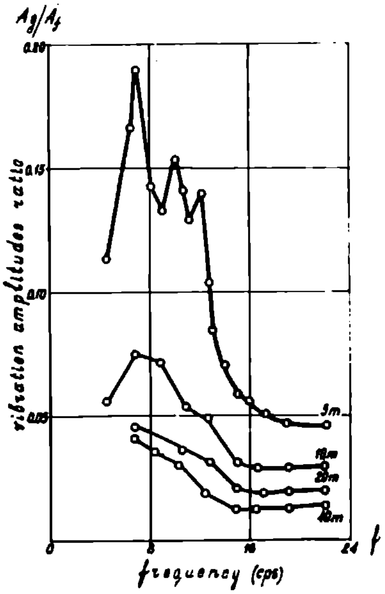
The vibrations were registered by means of the vibrograph VEGIK on the vibrator foundation and on the soil surface (vertical and horizontal components) at the different distances from the source (5 to 100 m). The maximum amplitude of the vibrator foundation was 1 mm.

As result of these experiments the wave propagation velocity as well as their damping in function of a distance from the source have been found.

The principal attention was paid to the resonance properties of the test site soils.The test data allowed us to plot the resonance curves of the foundation-soil system and as consequence to determine the relationships between the vibration amplitudes of the soil at the different points from the source and the frequencies.Besides the values of the soil amplitude and the source foundation amplitude ratio (A_s/A_f) as function of the frequencies have been determined.

The relationship between the ratio A_s/A_f at the different point show that the soil surface layer has the resonance frequencies which do not coincide with the resonance frequency of the soil foundation system.In our case this frequency is equal to 15Hz.The tests show that the greatest values of the ratio A_z/A_f correspond to the interval of 4-12Hz, maximum being at 7Hz. Referencing to the Figure it can be seen that the general shape of the curves A_z/A_F at the different points of the soil surface is quite the same, but this ratio decreases with the distance.

The above-mentioned results may be used for the evaluation of the effect of the soil condition on the intensity of the seismic forces developped on the soil surface and they confirme the perspective of the application of the steady-state wave to the investigation of the soil seismic properties.



Dr. Gorbunov B.P., Stepanowa E.W. (USSR)

Nous avons recherché eu théorie la mésure R, dependée de la pression en l'injecteur et de son rayon. Cette mesure est un rayon dirijé en haut de l'injecteur, qui nous applerons "ort"

$$R=\sqrt{\frac{P}{\gamma}r_0} \qquad (1)$$

Ou γ - poids volumetrique de coilis.

Le changement temporel de la nature de coulis nous pouvons exprimer à l'aide de viscosité de coulis, augmentée pendent le temp de formation du gel (de T á h_0).

Il est fixe la dependance du temp t et de la viscosité (pour KM-3)

$$\theta^2+\mu^2=1 \qquad (2)$$

$$\theta=\frac{t}{T}\,;\quad \mu=\frac{\eta_0}{\eta}$$

Le metode de superposition et la metode de changement des regimes stationnaires nous ont donnée les formules des rayons de la diffusion coulis depend du temps et de la pression constant dél injecteur

$$\frac{k_b\eta_b T}{n\,\eta_0 R}=\frac{\tau}{\vartheta} \qquad (3)$$

ou K_b - coefficient de permeabilite du sol indiqué à l'aux

η - visocisité d'aux

n - porosité de sole

τ - fonction de diffusion du rayon de coulis

ϑ - fonction de changement de viscosité

Soit $R \gg r_0$ r pour le rayon dirigé en haut

$$\tau_1=\frac{1}{2}\ln\frac{1+\xi}{1-\xi}-\xi \qquad (4)$$

pour le rayon dirigé en bas

$$\tau_2=\xi-\mathrm{arctg}\,\xi \qquad (5)$$

art

pous le coulis impondérable

$$\tau_1=\tau_2=\tau_3=\frac{\xi^3}{3} \qquad (6)$$

ou $\xi=\frac{r}{R}\,;\ \vartheta=\frac{1}{2}\left(Q\sqrt{1-\theta^2}+\mathrm{arc\,Sin}\,\theta\right)$ (7)

Soit $\theta<0{,}3$ $\vartheta=0$ suffisament precise.

Il est nécessair donner le rayon pour que determiner le temps, ou donner le temp pour que determiner le rayon (3).

Pour cela il faut d'abord tabuler et representer grafique les fonctions (4,5,6,7).

La difference entre le rayon pratique et le rayon calcule est négligeable en ces, si la pression en l'injecteur n'est pas demoli le sole.

En utilisant les injecteurs perfores, nous devons changer la pression P par la pression P_1 pour calculer l'injection

$$P_1=\frac{N r_u}{2 r_0}P \qquad (8)$$

N-nombre d'orifices

r_u- rayon d'orifice

A l'aide de fonctions et de "orte" nous avons recherché: la formule, le rendement de pompe, assurée la pression donnée.

$$Q=\frac{4\pi k_b\eta_b}{\eta_0}R^2 \qquad (9)$$

et la formule de volume des sols consolidés autour de l'injecteur

$$V=4\pi R^3\tau_3 \qquad (10)$$

Cettes formuls peuvent etre utilisé pour le projet de la consolidation des sols et, nottament, pour l'injection des coulis à l'aide d'injecteures haurisontaux.

This was a [illegible] for N7 Specialty Session

CALCULATIONS OF EXCESS PORE WATER PRESSURES IN SATURATED SOILS UNDER DYNAMIC EFFECT

P.I.Gorelyshev, A.J.Smiltnek, L.A.Eisler (U.S.S.R.)

The paper presents a calculation example of excess pressures under vertical vibrations of a saturated soil layer taking into account variations in the stress-strain state of the soil skeleton under dynamic loads and test data on the accumulation rate of irreversible deformations in the above condition. Analysis was based on the data obtained from testing a saturated sand specimen of 1 m diameter and 3 m high at a value of ρ_{sc} averaged over the height and equal to 1.49 gm/cm^3 ($D = 0{,}3$), the tests being performed on a large shaking table with vibrationfrequency $f = 10$ cps and acceleration $\alpha = 0.2$ g at the Dneprodzerzhinsk Branch of the All-Union Research Institute of Hydraulic Engineering. A value of the skeleton reversible strain, ε_0, was taken as a fundamental parameter characterizing the soil stress-strain state in different points of the specimen over the height. For the case under investigation (a small value of the permeability coefficient and an almost complete saturation) the above value of ε_0 is defined chiefly by water elastic properties and is independent of the skeleton static stress state. In the conditions corresponding to the above $\varepsilon_0 = 0.16 \cdot 10^{-5} x$, where x - depth of a point in the soil specimen in m . Investigations of accumulation rate of irreversible volumetric strains ($\dot{\varepsilon}_p$) in soil skeleton for the prescribed values of $\varepsilon_0(x)$, $f = 10$ cps and $\rho_{sc} = 1.49$ g/cm^3 according to static load (σ_x (and vibration time (t), were carried out on a small-size shaking table with specimens of 16 cm in diameter and 6 cm in height The static load was simulated by loads ensuring the necessary values of σ_x and ε_0 by an appropriate choice of the load mass and acceleration of the shaking table vibration. This testing procedure reproduces more closely natural conditions of a soil element dynamic loading than the currently used method of non-inertia loads. The experimental results can be presented as a relation $\dot{\varepsilon}_p = \dot{\varepsilon}_{p0}(\sigma_x,\varepsilon_0)\exp(-\lambda t)$ where $\lambda =$ (0.04 ± 0.01) $^1/$sec and the relation $\dot{\varepsilon}_{p0}(\sigma_x,\varepsilon_0)$ is plotted in Fig. 1.

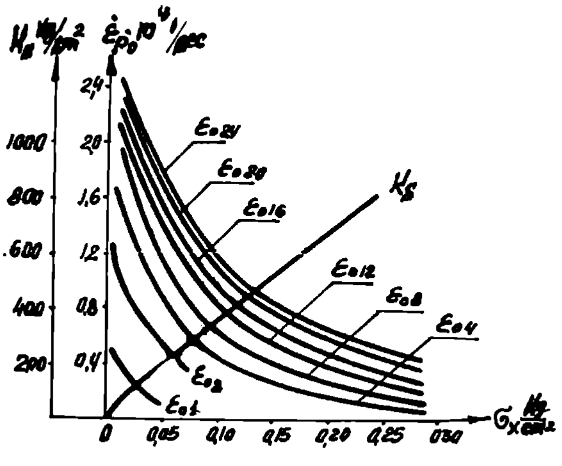


Fig. 1 Curves $\dot{\varepsilon}_{p0} = \dot{\varepsilon}_{p0}(\sigma_x)$ and $K_s = K_s(\sigma_x)$ for different values of $\varepsilon_0 \cdot : (\varepsilon_{0n} = 0{,}2 \cdot 10^{-6} \cdot n)$

Due to the presence of water in soil pores and the impossibility of attaining irreversible strains in the soil skeleton under short-term loads /1/, the rate of decrease in the static stresses in the skeleton (σ_x) and the consequent increase in the pore water pressure (P) are represented by the relation $\partial P/\partial t = \partial \sigma/\partial t = K_s \dot{\varepsilon}_p(\sigma_x, \varepsilon_0, t)$. Experiments on a small-size shaking table have shown that the elastic module, K_s , is dependent on σ_x . This relation is presented in Fig. 1. Surplus pore water pressure (U) for a 3 m thick specimen of sand were calculated using the non-uniform consolidation equation

$$\frac{\partial U}{\partial t} - a^2 \frac{\partial^2 U}{\partial x^2} = \frac{\partial P(\sigma_x, \varepsilon_0, t)}{\partial t} \qquad (4)$$

The average value of a^2 which proved to be equal to 1000 cm/sec was defined from tests performed on a large scale shaking table (Dneprodzerzhinsk Branch of the All-Union Research Institute of Hydraulic Engineering) according to the consolidation time of saturated sand after the removal of dynamic load. The value $U(x,t)$ was calculated by the finite difference method under zero initial and upper boundary conditions. On the boundary $x = 3$ m it was assumed that $\partial U/\partial x = 0$. Changes in σ_x were defined for each time step, and values of $K_s(x)$ and $\dot{\varepsilon}_{p0}(x)$ were corrected using the plots shown in Fig.1. Calculation results are presented in Fig. 2 together with the surplus pressure values obtained by testing a 3 m thick sand soil specimen.

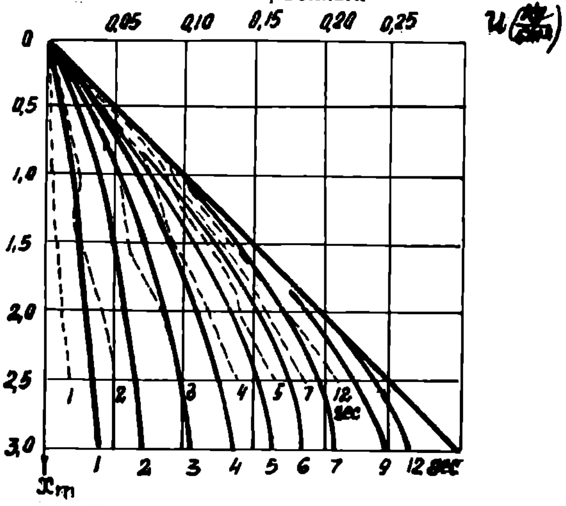


Fig. 2 Surplus pressure distribution over the depth of a 3 m thick specimen for different time moments. (——— calculated, - - - - - tested)

The analytical and experimental data are in fair agreement. Some discrepancy in the data may be attributed to a possible non uniformity of the specimen and the time necessary to set oscillations of the shaking table (~ 1.5 sec) which were not included in computation. The calculation method proposed may also be modified for non-uniform soils and more complicated cases of static and dynamic loading.

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VIBRATIONS OF MACHINE FOUNDATIONS. S.K.Guha, A.V.Wedpathak. P.J.Desai, S.C.Marwadi (India)

Central Water and Power Research Station, Poona. had undertaken during the last decade or so extensive prototype vibration survey on about thirty massive machine foundations (both block and frame type) founded on various types of soil in India in order to compare the experimentally observed vibration intensities with those theoretically calculated ones (Barkan, 1962) and also to evaluate, in quantitative terms, the influence of type of soil on observed foundation vibrations.

Suitable experimental set-ups comprising Philips Electrodynamic pick-ups with recording oscillographs, three-component Sprengnether Seismographs (x50 and x500) etc. were used to measure the vibrations at the foundations when machines were working under normal conditions. Vibration measurements on block foundations founded on soft soil, weathered and compact rock etc. have conspicuously revealed predominant influence of type of soil on vibrations Vibrations of foundations resting on soft soil are accentuated several times more than those resting on compact rock specially in case of low frequency machines (Reissner, 1936; Barkan, 1962). Similar extensive prototype vibration survey on frame foundations in thermal power stations has revealed that the observed amplitudes of vibrations in the horizontal components are comparable to those in vertical component. On the contrary, theoretically computed amplitudes of vertical vibrations are generally much larger than those experimentally observed while the computed amplitudes of horizontal vibrations are much smaller than the experimentally observed ones (Guha et al., 1973). These descripancies may suggest that the various assumptions involved in theoretical analysis of frame foundations may not be fully valid in assessing dynamic behaviour of frame foundations. However, in such cases theoretical computations have been done assuming non-yielding rock formations and hence do not provide any rational method for obtaining influence of type of soil on the vibrations of frame foundations. The results of theoretical computations would thus reasonably correspond to compact rocky formations. As the frequencies of these machines are generally high (50 cps) vibration intensity does not vary so conspicuously as the low frequency machines on block foundations.

Earthquake intensity on very soft soil could accentuate upto ten times or so of its value on compact rock formations. Kanai et al. (1966) had made extensive theoretical and experimental investigations on amplification character of soil layer during earthquake and had developed quantitative method for estimating amplification factors for various types of soil. Earthquake response spectra are also largely dependent on soil character and could thus quantitatively explain influences of soil layers on damages to structures. Theoretical and experimental results of vibrations of rigid foundation block resting on elastic half-space by Reissner (1936), Barkan (1962), Agarwal et al. (1971) etc. had amply corroborated the influence of elasticity of half-space. Enhancement of vibrations of block foundation on elastic half-space could also be explained from soil amplification factor (Kanai et al., 1966) following analogy in earthquakes. In absence of exact theory of frame foundation founded on elastic half-space, the following soil amplification factor, G(T), of Kanai has been used and could reasonably explain the relative enhancement of vibrations of frame foundation founded on elastic half-space :

[illegible]

where, T_G is the predominant period of site or microtremor period, T = period of earth waves, $= \rho_1 V_1 / \rho_2 V_2$ and V_1, V_2 and ρ_1, ρ_2 are velocities and densities of overburden soil layer (subscript-1) and of sub-stratum rock (subscript-2). When $T = 4H/V_1$, $G(T) = \rho_2 V_2 / \rho_1 V_1$ at resonance.

In order to verify the efficacy of the soil amplification factor, G(T), in explaining enhancement of vibration intensity of both block and frame type foundations, G(T) values computed from above equation, for various assumed parameters of local soil conditions at respective sites and the amplification factors observed from the actual prototype measurements on both block and frame type foundations, are found to be of similar order (between 1 and 3). Corrected vibration amplitudes of foundation could thus be A x G(T) where 'A' is the computed amplitude of frame foundation according to present theory of non-yielding type of rock. Thus, the soil amplification factor, G(T), as obtained from the above method can easily be applied to assess the relative increase in vibration intensity of machine foundations resting on various types of soil (Guha et al., 1973).

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MEASUREMENTS OF DYNAMIC PORE PRESSURE BY PILE DRIVING

K. ISHIHARA (JAPAN)

INTRODUCTION

Most of the works ever done for assessing the likelihood of liquefaction of the ground during earthquakes are based on the information of sand behaviors obtained in the laboratory test on disturbed samples. Since there are several factors, such as relative density and cementation effect, which make it difficult to simulate exactly the field conditions in the laboratory samples, it would be necessary to develop some vibratory test procedures that can be operated in the field. In view of this, an attempt was made, in a reclaimed sand deposit in Chiba, Japan, of measuring the increase in pore pressure caused by the vibration due to driving of compaction piles (Ishihara-Mitsui, 1972). Subsequently, the similar tests were done in Tokyo, but this time in the sand deposit of alluvial origin. This report describes the performances and results of the test.

TEST PERFORMANCES

The playground of Takasago Middle School located in the lowland area of Tokyo was chosen for the test site. The place, consisting of a natural levee deposit, has a soil profile as shown in Fig.1. The whole deposit consists of fine sands except that a layer with greater silt parting is sandwiched around the depth of 4.5 m. The grain size distributions of the sands at three depths are shown in Fig.2.

The test project involved the use of a 30 cm in diameter pile driven by a vertically acting vibrator to produce vibration in the ground. At the distance of 1.0 m from the location at which the pile was to be driven, three bored holes were drilled, as shown in Fig.3(a), each to the depth of 4.5, 7.0 and 10.0 m, and a cylindrical capsule whose profile is shown in Fig.4 was placed at the bottom of these holes after washing out the fines and then backfilled with the sand. The capsule weighing 16.2 kg encases a piezometer and a vertical accelerometer and has many small holes round the side wall to permit free drainage of water to and from the adjacent sand deposit. A cone-shaped tip is attached to the end to help the capsule pushed upright into the bottom soil. After all these arrangements had been finished, pile driving was started using a 30 ton vibrator operating with a frequency of 17 Hz. The pore pressure and vertical acceleration as they changed during pile driving were picked up electrically and recorded on oscillograph papers. Similar tests were performed repetitively four times under identical conditions at the same test lot.

TEST RESULTS AND DISCUSSIONS

Fig.5 shows the variation of excess pore water pressure and accelerations with time. The pore pressure fluctuates within each cycle with the mean value shifting away from the static pressure. It is this shift divided by the initial effective overburden pressure that is plotted in Fig.5. The depth of pile penetration is also shown in Fig.5 against elapse of time. Fig.5 shows that the acceleration reached its maximum when the pile was driven down to about 1.0 m above the point of measurement and then decreased faily acutely as the pile passed by. The pore pressure changed almost in accord with the acceleration. There are good reasons to believe that, once the pore pressure decreases the densification of the sand occurs and the virgin state of the deposit is destroyed. Therefore, if the liquefaction potential in the virgin state is of concern, it is necessary to pay attention to the pore pressure behaviors at the stage where it still is increasing. The amplitude of vertical acceleration at the time when different pore pressures were reached for the first time was read off from Fig.5 and replotted in Fig.6 against the corresponding pore pressure. The average data points in Fig.6 are approximated by straight lines, and the blow counts obtained at the depths where pore pressures were measured are also indicated. Similar data obtained in the previous test under approximately identical conditions on a reclaimed deposit in Chiba (Ishihara-Mitsui, 1972) are also plotted in Fig.6. It is seen that the magnitude of acceleration required to induce a certain pore pressure increases as the blow count of the layer increases,

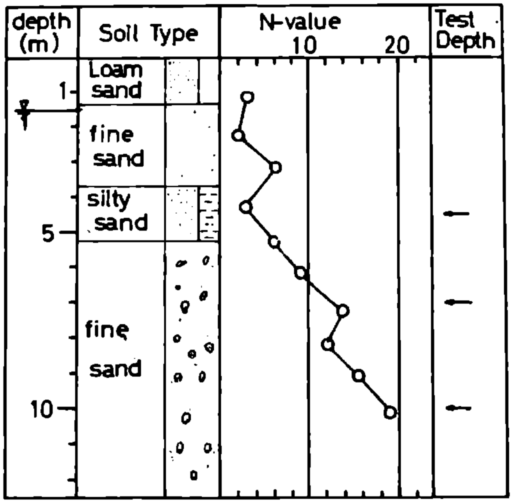


Fig.1 Soil Profile at the test site

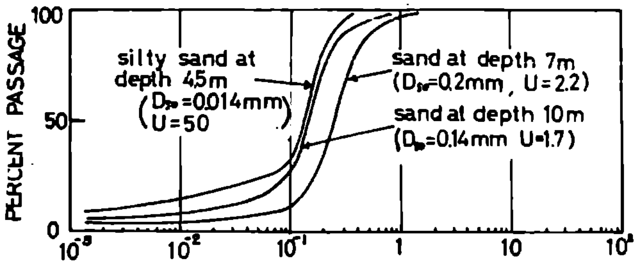
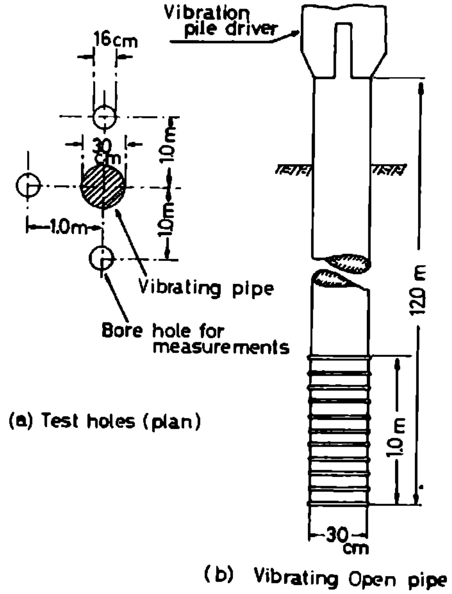


Fig.2 Grain size distribution of the sands



(a) Test holes (plan)

(b) Vibrating Open pipe

Fig.3 Layout of the test

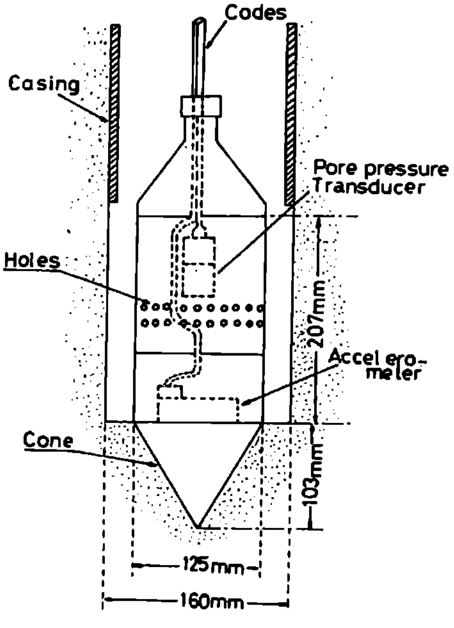


Fig.4 Capsule for pore pressure and acceleration measurements

but the rate of increase in required acceleration appears to be different between the two series of tests being considered. To clarify this point further, the accelerations required to raise the pore pressure by an amount equal to say 10% of the effective overburden pressure were read off from Fig.6, and plotted in Fig.7 versus the corresponding blow count. Good correlation between the blow count and the acceleration seems to exist within each of the test series, but the relationship of this kind is quite different between these two series. In explanation, a greater acceleration is required in Tokyo test site than in Chiba site to raise the same proportion of pore pressure, although the density of the deposit as represented by blow count is supposed to be the same. This consequence seems to indicate that there are some factors related with the pore pressure characteristics which can not be accounted for by the N-value alone. At the current state of knowledge, it is difficult to define the factors, but it will be interesting to see the apparent differences which prevailed between the two test conditions. First of all, the alluvial deposit in Tokyo is considered several hundred years old or even older, whereas it was only about one year after reclamation that the test was run in Chiba. The age effect as possibly represented by cementation and the like may account partly for the difference. Secondly, the ground water table in Chiba site was located about 0.5 m below the ground surface, whereas at the Tokyo site the water table was about 1.5 m deep. The difference in the level of water table seems to come out more predominantly than it is taken into account by dividing the pore pressure by the effective overburden pressure.

CONCLUSIONS

Pore pressure measurements performed during vibratory pile driving into a saturated alluvial sand deposit have shown that there exists an unique relationship between the blow count and the acceleration level required to develop a given pore pressure near the driven pile. This relationship was established for the alluvial sand at Tokyo test site and compared with the similar relationship obtained in the previous test on the reclaimed sand in Chiba. The comparison has disclosed a difference in pore pressure development potential between these test sites which can not be explained by the difference in blow count alone.

ACKNOWLEDGEMENTS

The field test described herein was sponsored by the Tokyo Metropolitan Government and performed by the contractor, Token Geological Investigation Company. The author is much grateful for this support. He is also greatly indebted to Professor Masami Fukuoka of University of Tokyo for his useful and encouraging comments.

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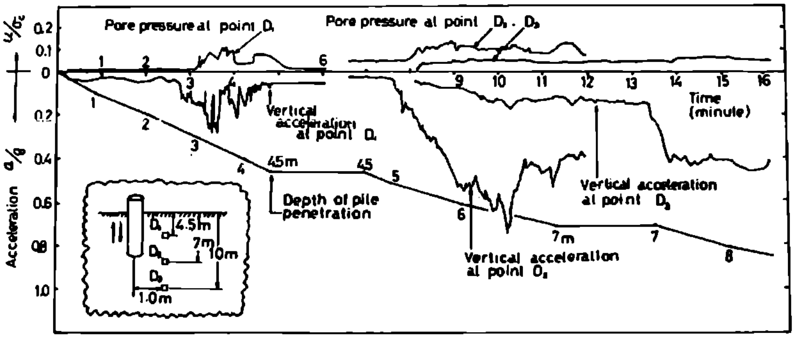
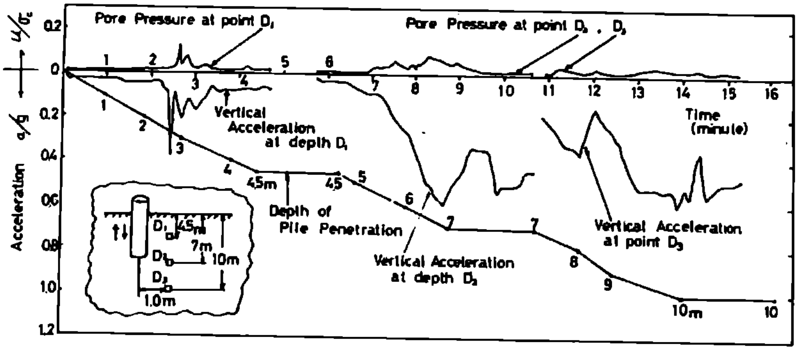


(a) Test No.1



(b) Test No.2

Fig.5 Change with time of pore pressures and acceleration

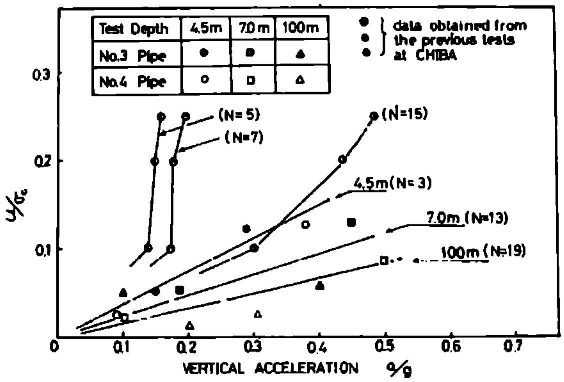


Fig.6 Relationship between pore pressure and vertical acceleration

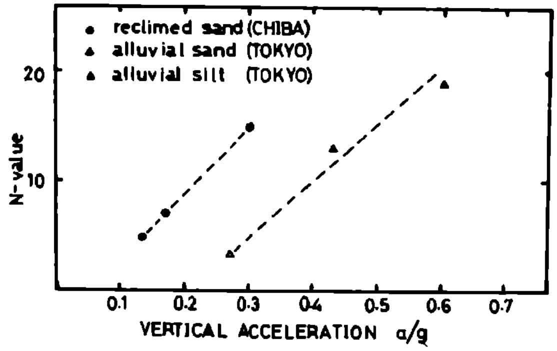


Fig.7 Vertical acceleration required to increase pore pressure by 10% of the effective overburden pressure

SAND LIQUEFACTION AND ITS ANALYSIS AS FATIGUE PHENOMENON, Y.Tomozawa Iwasaki (Japan)

CHARACTERISTICS OF SAND LIQUEFACTION

Some characteristics of the sand liquefaction obtained by laboratory tests by many investigators show that the liquefaction is a special type of the fatigue failure of undrained saturated sands under cyclic loading condition. These characteristics of liquefaction common to general fatigue phenomenon are;

1. Without significant plastic deformation, saturated sands fail under cyclic loading.1)
2. Cyclic shear stress with an amplitude smaller than the static shear strength causes failure.1)
3. The stress amplitude becomes smaller, the number of cycles required to cause failure becomes larger.1)
4. With the same sand, by the same method of testing, there are some scatterings in the test results.

S-N CURVE FOR UNDRAINED SATURATED SAND AND ITS FAILURE

The results of cyclic simple shear tests by Seed and Peacock1) are replotted in Fig. 1 as an example of S-N curve for liquefaction and can be expressed by the following equation (1).

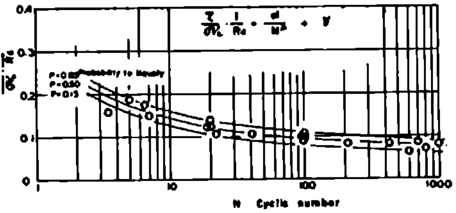


FIG. 1. S-N Curve for Liquefaction Obtained from Cyclic Simple Shear Tests — with lines of probability to liquefy.

$$\frac{\tau}{\sigma_v'\cdot R_d} = \frac{\alpha}{N^{\beta}} + \gamma \qquad (1)$$

where,

τ ; cyclic shear stress applied on the potential failure surface.

σ_v'; vertical effective stress applied on the potential failure surface before loading cyclic stress.

R_d ; relative density of the sand.

N ; number of the applied cyclic stress required to cause failure.

α, β, γ ; parameters.

Assuming that α and β are constant and that the scattering depends mainly on a probablistic parameter γ which shows normal distribution. Those constants and parameter are estimated by nonlinear regression analysis. The standard diviation of γ is 0.013 and the S-N curve is expressed by the following equation (2).

$$\frac{\tau}{\sigma_v R_d} = \frac{0.23}{N^{0.52}} \quad 0.071 + 0.013k \qquad (2)$$

where k= 0.0 for probability of failure 50%
k= 2.0 " 95%
k=-2.0 " 5%

In this study of liquefaction analysis applied for Niigata, the value of k=0 is adapted, however for design purpose for important projects k=-2.0 may be used with much larger safety.

Since those S-N curves are obtained through constant stress amplitude tests, it could not be applied directly to random stress case like in earthquake motions. To analyse such random stress effects on the fatigue failure using S-N curves obtained by constant stress amplitude tests, it is common and convenient to define the Life of the material to be 1.0 before loading and become to zero at the failure. If the material is loaded n_i times with a constant stress amplitude S_i with which N_s times loading causes failure of the material, the Life of the material is assumed to decrease from 1.0 to 1.0-n_i/N_s. If some random stresses with various amplitudes of $S_1, S_2, \ldots, S_k$ are applied with number of loadings of $n_1, n_2 \ldots n_k$ respectively, the Life of the material left after these random loadings is assumed by the following equation(3).

$$\text{Life} = 1.0 - \sum_{i=1}^{k} \frac{n_i}{N_{si}} \qquad (3)$$

When the value Life decreases to zero, the fatigue failure is assumed to occur.

AN ANALYSIS OF LIQUEFACTION IN NIIGATA OF 1964 AS FATIGUE FAILURE.

Using shear stress history in the ground based on the accelerograms recorded at the liquefied area in Niigata city, an analysis of sand liquefaction as fatigue failure is shown below.

In Niigata city the area was divided into 3 zones according to the earthquake damage; A-zone: No damage, B-zone: Light damage, C-zone: Heavy damage, shown in Fig. 2.2)

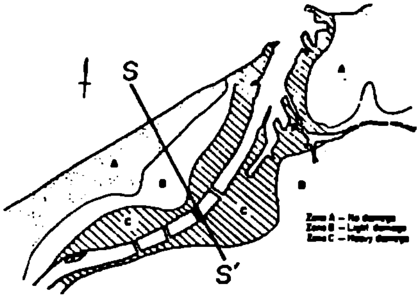
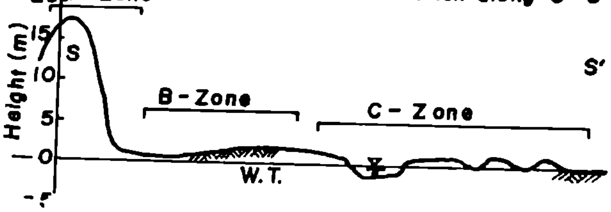


Fig.2 Damaged Zones in Niigata

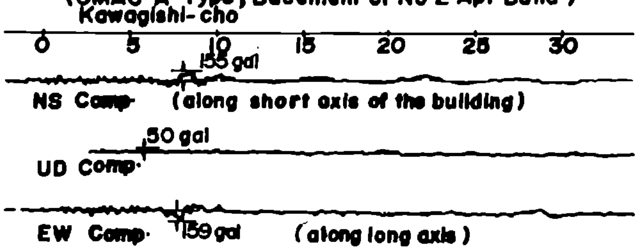
The profile of the section S-S' is shown in Fig.3. The difference among these zones are that while C zone shows very high water table with very loose sandy ground, A and B zones consist of loose sand with different water tables. Water table in B zone was very high as in C zone but A zone showed rather low ground water table(see Fig.3).

Fig.3 Profile of the Section along S-S'



Accelerograms are shown in Fig. 4 recorded at the

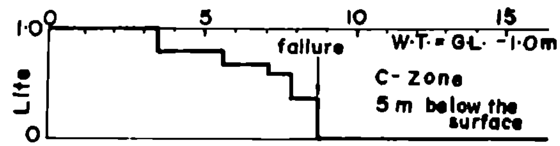
Fig·4 Record of Ground Accelerations during Niigata Earthquake (SMAC A-Type, Bacement of No·2 Apt·Build·) Kawagishi-cho



base of an apartment building at Kawagishi-cho, Niigata, where many buildings were tilted heavily without structural damage due to liquefaction of the ground. It should be recognized that the accelerograms change their shapes after 10 sec.. The dominent high frequency components suddenly disappear and then rather low frequency movements take place especially for NS component which coincides the short axis of the building. This abrupt change of the motions may be explained as the rocking motion about the long axis of the building was strongly induced after the ground was liquefied and become "very soft foundation". These surface acceleration records were used to estimate shear stress history in the ground of Niigata area. Assuming horizontal motions are mainly due to vertically transmitted SH-waves, the wave motions in the ground may be calculated from known surface motions by multiple reflection method. An example of the calculated shear stress history for the depth of 5 m below the surface in C zone is shown in Fig.5. Water table in C zone was about 1 m below the surface. Standard penetration test results in Niigata shows that subsurface of 5 -10 m thick has relative density about 40 - 70 %.

Fig·5 Shear Stress History at Depth 5m in C Zone

Assuming unit weight of sand 1.8g/cm^3 above the water table and 2.0g/cm^3 below the water table, effective overburden pressures are calculated. Since it was proposed1) that the cyclic stress required to cause liquefaction under level ground condition in the field would be about 40 -50% higher than those shown in Fig. 1, 45% higher level of S-N curve than in Fig. 1 is used to analyse the liquefaction in this study. The estimated Life of the saturated sand 5 m below the surface in C zone is calculated and shown in Fig. 6. The Life decreases with time from 1.0 to zero when the fatigue failure takes place about 8 sec.



Fig·6 Decrease of Life with Time

In C-zone, the same procedures are applied to various depths to obtain Life. The variation of Life with depth at several times are computed and shown in Fig . 7. Also in the Fig. 7, variation of Life for B-zone and for A zone are shown. They indicate that C zone of heavy damage area liquefied at about 8 sec. at the depth between 4 and 8 m, B zone had almost liquefied, while A zone did not liquefy, which corresponds well with the degree of damage observed in the field.2)3)

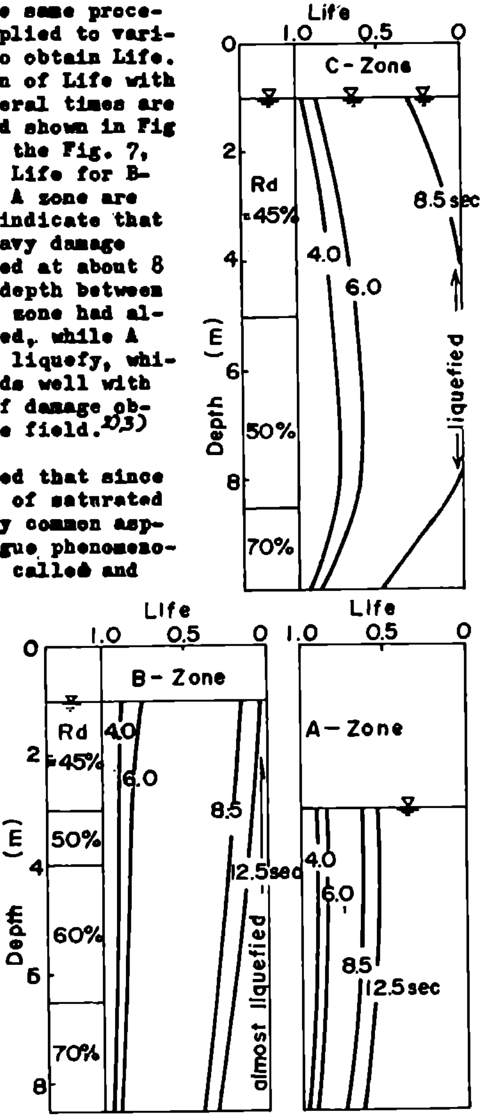


Fig.7 Change of Variation of Life with Time in three Zones

CONCLUSION.

It is proposed that since liquefaction of saturated sand has many common aspects to fatigue phenomenon, it may be called and understood as more general term "Fatigue Failure". It is fortunate that the mechanics of fatigue in saturated sand has been researched and clarified as the result of gradual increment of pore water pressure. Through nonlinear regression analysis, S-N curves with probability of failure for a simple shear test results are obtained.

Life of the saturated sand under cyclic loading is defined and a method to estimate the fatigue analysis for liquefaction is proposed.

Fatigue analysis for liquefaction applied for Niigata earthquake of 1964 showed good agreements with observation in the field.

The author is grateful for useful discussions with Prof. H.B.Seed.

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LONG MACHINE FOUNDATIONS ON FLEXIBLE SOILS.

P.Karasudhi (Thailand)

INTRODUCTION

The author participated in two studies (Karasudhi et al, 1970 and 1972) on the dynamic response of long machine foundations on flexible soils. The first study concerns with thick foundations directly supporting the machines, while the second one discusses single-story machine frame foundations. The interaction forces and torque between the supporting soil and the foundation is incorporated by utilizing the solution of the steady state vibration of a long rigid plate on an elastic half space obtained by Karasudhi et al (1968). The system is excited by a harmonic centrifugal force induced by the rotation of the unbalanced mass of the machine rotor.

THICK FOUNDATIONS

For this case (Karasudhi et al, 1970), there are three modes of vibration, vertical, horizontal and rocking; the horizontal and rocking motions are coupled. The amplitude of each mode of vibration are plotted against arguments of the machine operating frequency for various values of parameters corresponding to the mass of the foundation plus the mass of the machine, the foundation thickness, the height of the machine centroid, and the soil Poisson's ratio. These charts facilitate the analysis and design of actual foundations. A design example for a 1000 rpm machine is presented in the paper.

SINGLE STORY FRAME FOUNDATIONS

For machinery, frame foundations have many advantages over the massive ones (Barkan, 1962). Karasudhi et al (1972) study the response of long machine frame foundations, and show that their vertical vibration is a single-degree-of-freedom motion and the coupled horizontal, rocking and flexural (interfloor column bending) vibration is a three-degree-of-freedom motion. Two frames which are commonly used in practice, one supporting a 200,000kw turbogenerator running at 3000 rpm and the other a 500 kw motor generator running at 750 rpm, are analyzed.

RESULTS AND CONCLUSIONS

The response of the system depends heavily upon the soil shear wave velocity and the mass ratio $\bar{m}$ which is defined as $m/\rho b^2$, where m = mass of the foundation plus the mass of the machine, ρ = soil mass density, and b = half width of the foundation base. The influence of the soil Poisson's ratio is less prominent For a low mass ratio ($\bar{m} < 5$) which is very common in practice, there is at most one resonance for the coupled motion.

In the study of Karasudhi et al (1972), the most severe condition, i.e. resonance, of a high speed machine is not at its operating frequency but when it is either building up its speed or slowing down. It is also found that the assumption of a rigid soil yields a more conservative design of the frame than that of a flexible soil; and, for the latter case, varying the column stiffness of the frame is not an effective mean for avoiding resonance in the design.

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VIBRATION INSULATED FOUNDATIONS UNDER MACHINES

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Vibration-insulated foundations under machines are one of the most widely spread ways of protecting buildings from vibration which permit to prevent settlements of foundations of machines and structures due to the sustained effect of vibration as well as to avoid the strengthening of bases, i.e.the usage of piles, arrangement of compacted sand layers etc.

The issue of the instruction developed by V.S.Martyshkin /1/ in 1956 promoted wide-scale usage of vibration insulation. Basing on the experience gained in the application of vibration insulation /1/ has been further developed and a new guidance /2/ has been worked out. In the guidance particular attention is paid to active vibration insulation at periodic effects, antishock vibration insulation and passive vibration in insulation.

The problem of active vibration insulation is studied best of all. In the dynamic analysis it has become common now to use simplified schemes in standard cases, and to consider in detail oscillations of vibration-protected arrangement as a system with six degrees of freedom when, on one hand, higher accuracy of design is needed, and, on the other hand, the system is not symmetric enough. The article /2/ discusses in detail a more accurate estimate at the stage of service. Consideration of transient regimes, that is passing through resonance at the starting and stopping of machines, plays a significant role in the theory of active vibration insulation.

These problems are solved in /2/ as well as in /1/ on the basis of considering a single-degree-of-freedom system passing through resonance. In the course of further development of the foregoing analysis some clarifications are feasible associated with considering systems with several degrees of freedom (A.P.Philippov, E.P.Goloskokov /3/ various laws of passing through resonance (A.P.Philippov, B.G.Korenev), account of specific measures aimed at suppressing starting and stopping resonances.

The application of new constructive solutions, that is new damping arrangements and plastic pliable members described in /2/, usage of dampers to suppress starting-stopping resonances /4/ are of particular importance in further development of vibration insulation.

In practical application of antishock vibration insulation it is of particular importance to make more accurate and even to extend the analysis with a view to take into account wave processes in the ground /5/, to consider in detail the impulse resonance.

The application of spring vibration insulation regarded in /2/ has also brought about some modifications in the dynamic analysis /6,7/.

The problems of passive vibration insulation in /2/ are considered rather widely. Along with the determinate displacements of foundations, it is for the first time that random displacements are analysed. In studying determinate displacements of the foundation a system with several degrees of freedom is considered with sufficient detail; in the stochastic presentation of the problem a single degree-of-freedom system only is analysed. The effect of the dynamical sources placed on the foundations upon the passively insulated arrangements is also discussed.

Though engineering problems of vibration insulation are rather well studied, there still remain many problems to be solved. Among them are the theory of non-linear vibration insulation, development of new effective damping arrangements, deeper study of transient regimes, etc.

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SOME ASPECTS OF DYNAMIC STABILITY OF COHESIONLESS SOIL AND MEASURES AGAINST POSSIBLE LIQUEFACTION DUE TO SEISMIC EFFECT.

N.N.Maslov, Yu.P.Shkitskiy.

The problem of a saturated sand stability under dynamic (seismic) conditions remains at present one of the most important issues and its role becomes especially significant in connection with the fact that the favourable effect of a sand stratum dead weight under these conditions is limited. This was proved by H.Rasoulov in 1968.

The present report covers results of the recent investigations aimed at solving some practically important problems and basing on the Maslov "filtration theory" (1953). According to this theory disruption of a saturated cohesionless soil stability results from degradation of its shear strength properties under dynamic conditions due to development of the uplift pressure in the soil stratum.

The development of a positive pressure head (h_z) becomes feasible provided the acceleration of an oscillating motion reflecting intensity of the seismic effect ($\alpha_{seismic}$) is bigger than the critical acceleration (α_{cro}) i.e. the acceleration below which densification of horizontal saturated sand layers takes place.

Then the shear resistance of the soil under seismic conditions (S_{din}) can be expressed as follows $S_{din}=[(p_o+\gamma_s \cdot z)-\Delta_w \cdot h_z]\cdot tg\varphi$, (1)

where P_o - external load;
$\Delta_w; \gamma_s$ - volume weight of water and bulk density of suspended soil;
φ - angle of internal friction in sand;
z - flowing coordinate through stratum.

The fact which was recorded by many investigators (N.Maslov 1959, Se-Din-I,1962,P.Ivanov 1962 etc). that the sand constituting the submerged slopes turns out to be less stable than the horizontal layers made up of the same sand,was theoretically substantiated by V.Kanan (1969) who corraborated the Maslov's idea (1959) about effect of the tangential stress developing in the slope,on the maghiture of the critical acceleration ($\alpha_{cr\beta}$) which characterizes the seismic stability of soil in the slope set at an angle (β).V.A.Kanan derived the following relationship

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$$\alpha_{cr\beta}=\alpha_{cro}\left(1-\frac{\tau_{max}}{S_{st}}\right) \qquad (2)$$

Thus a slope will be seismically stable if $\alpha_{cr\beta}>\alpha_{seism}$ condition is satisfied for the soil constituting the slope. In practice the condition is met by using varions measures that improve the dynamic stability of soil (increase of $\alpha_{cr\beta}$). One of the most frequenfly used methods is surcharging.Favourable effect of the surcharging on the dam slope for example a layer of riprap, was proved by many investigators (V.Radina 1953, M.Goldshtein 1953, N.Maslov 1953, V.Ershov 1953 etc) Nevertheless methods of determining the required thickness of surcharging have not been adequately developed yet.

The method of determining the thickness of surcharging (d), suggested by us, calls for two possible alternative solutions:

a.to preculde completely possibility of soil liquification in the submerged slope and hence,preclude development of a positive pressure head h_z due to seismic effect ($\alpha_{cr\beta}>\alpha_{seism}$);

b. the weight of a surcharging layer neutralizes the uplift pressure $\Delta_w\, h_z$ that can develop in the slope subjected to seismic effect $\alpha_{seism}>\alpha_{cr\beta}$ (condition $\Delta_w \cdot h_z = d \cdot \gamma_{sg}$)

The following relationship was derived proceeding from the I-st alternative

$$d=\frac{1}{u}\cdot\frac{\alpha_{seism}-\alpha_{cr\beta}}{\gamma_{sg}\cdot Cos\beta}, \qquad (3)$$

where u- factor of normal stresses effect on $\alpha_{кр\beta}$; γ_{sg}- dry bulk density of surcharging layer;
$d \cdot \gamma_{sg} \cdot Cos\beta$- component of surcharging layer weight normal to slope surface.

Basing on the 2-nd alternative we derived several relationships,one of which is given herein after

$$d=\frac{1}{6}\cdot\frac{\Delta_w}{\gamma_{sg}}\cdot\frac{1}{Cos\beta}\cdot\frac{\nu_n^p}{K_f}\cdot\mathcal{L}^2 \qquad (4)$$

Relationship (4) satisfies the case when

$$h_z=\frac{1}{2}\cdot\frac{\nu_o^p}{K_f}\cdot\left[\mathcal{L}\cdot z-z^2+\frac{z^3}{3\mathcal{L}}\right] \qquad (5)$$

Other possible cases of pressure head h_z distribution are discussed in (L_z).

In formula (4), ν_n^p-stands for coefficient of dynamic liquification (L_z) with due regard for the surcharging; k_f- coefficient of soil permeability; L-thickness of active zone to be determined from the expression derived by H.Rasoulov (1968) $-L=\xi.\nu_o^p.H.t$ (6) where H-thickness of sand stratum, ξ -factor of unstable conditions, t-duration of seismic effect.

Expression (6) shows that the role of the soil dead weight decreases with adequate duration of seismic effect.

The comparative analysis based on formulas (3) and (4),showed that the surcharge layer thickness for the submerged slope 100m in height should be 13,2m and 6.73 m respectively.

Investigations into consequences of heavy earthquakes reveals that in some cases the cause of considerable and nonuniform settlement of the structures lies in decrease of the structure setting effectiveness due to uplift pressure ($h_z . \Delta_w$) and decrease of the dead weight role in the zones bordering on the structure with the uplift pressure rise,decrease of the setting effectiveness takes place.This results in a considerable developments of zones with the critical and super critical states of soils in the structure fase.The structure will be subjected to significant and nonuniform settlement and in some cases the structure loses completely its stability.

Basing on the performed investigations and analysis of specific examples (consequences of the earthquake in Niigata Japan,1964), R.Tchaplanova (1972) found out as follows.

I.Decrease of the structure setting effectiveness can occur when the saturated cohesionless soil in the structure base is subjected to earth quakes.

2. Stability of the structure can be ensured provided the following conditions are satisfied.

a) Condition $\alpha_{cr}>\alpha_{seism}$ is met at all levels of the stratum in the structure base.

b)Depth of structure setting is designed with due regard for possible soil uplift in the zones bordering on the structure.

ENERGETIC CORRELATION OF CONSTRUCTION AND SOIL UNDER VIBRATIONS CAUSED BY SEISMIC FORCES. E.S.Medvedeva, /USSR/

Let us discuss the problem of energetic correlations of construction and soil for different ratios of velocities, densities and rigidities in constructions and soil. When seismic forces are acting the soil transfers it's kinetic energy to the construction. The value of kinetic energy depends on the ratio of seismic rigidity of soil and construction. The construction cannot always absorb the energy of elastic deformations. That's why some residual deformations can appear in structure, and some parts of construction can wreck.

The amount of energy, that the construction has got, depends and some factors. One of the most important of them is the seismic characteristic of soil, on which the building is constructed. Much depends on the construction itself. If the construction can absorb the great amount of energy in connection with plastic deformation, then it will be able resist seismic forces without being destroyed.

For the solution of the problem the construction is treated as the layer situated on the elastic half a space. They are parted with a plane a border, to which a flat seismic wave comes from downwards. It's spreading front is parallel to the border of the construction and the soil. The energy of spreading wave consists of kinetic E_k and potential energy E_n.

Physical conditions on the border are the following:

1. Entireness and impenetrability of the solid $$a_r(0,t) = a_3(0,t) \qquad (1)$$
2. Equality of action and counteraction $$\sigma_r(0,t) = \sigma_3(0,t) \qquad (2)$$

For the solution of the problem the construction is treated as inhomogeneous medium, which elements' dimensions are less than the length of the seismic waves. Then we can operate with effective values of waves' velocities V_p and V_s, densities ρ and seismic rigidities $V_p\rho$ and $V_s\rho$. The author made calculations and measurements of effective meanings of V_p, V_s and ρ for constructions. As a result constructions are classified in three types:

I type - buildings with walls of saman bricks, red bricks mortar's sort 10, floors of light concrete. $V_p\rho = 0{,}02 - 0{,}05$; $V_s\rho = 0{,}01 - 0{,}03$.

II type- constructions with thin walls of bricks; mortar's sort 50, with walls made of large blocks of light concrete.

Frame constructions with light filling

$V_p\rho = 0{,}05 - 0{,}1$

$V_s\rho = 0{,}03 - 0{,}06$

III type - large-panel constructions, made of volumetrical elements, with retaining walls of large blocks with reinforced concrete floors

$V_p\rho = 0{,}1 - 0{,}2$; $V_s\rho = 0{,}06 - 0{,}11$

Let us consider the propagation of flat seismic wave. For the moment t we can write down the following equation of equilibrium when the elementary part of soil with the length Δx is vibrating:

$$\rho_r F_\Delta x \frac{\partial^2 a}{\partial t^2} = F\sigma_r(x + \Delta x) - F\sigma_r(x) \qquad (3)$$

Making some transformations we get the following:

The mean value of energy $\bar{\mathcal{E}}_r$ of the part of the medium

$$\bar{\mathcal{E}}_r = \frac{1}{T}\int_{t_1}^{t_1+T} \mathcal{E}(t)\,dt = \frac{1}{2}\rho F \Delta x\, \omega^2 A_r^2 \qquad (4)$$

The mean density of energy $\bar{\mathcal{E}}_{rn}$

$$\bar{\mathcal{E}}_{rn} = \frac{1}{2}\rho\,\omega^2 A_r^2 \qquad (5)$$

From this formula one can obtain the mean density of energy flow of seismic wave in soil q_r $$q_r = \frac{1}{2} V \omega^2 A_r^2 \qquad (6)$$

When the seismic wave comes up vertically from the soil to the basement of the construction it is reflected and refracted. The mean density of the energy flow in reflected wave $\bar{q}'_r$ and that in refracted one (that is in the wave that the construction $\bar{q}_3$) are expressed by the formula

$$\bar{q}_3 = \frac{1}{2}\rho_3 V_3 \omega^2 A_3^2 \qquad (7)$$

The ratio of $\bar{q}_3$ and $\bar{q}'_r$ to the density of the approaching flow is the coefficient of reflection R_e and coefficient of seismic energy penetration

$$R_e = \frac{\bar{q}'_3}{\bar{q}_r}; \quad T_e = \frac{\bar{q}'_r}{q_r} \qquad (8)$$

$$R_e = \left(\frac{1-\gamma}{1+\gamma}\right)^2; \quad T_e = \frac{4\gamma}{(1+\gamma)^2} \qquad (9)$$

where γ - is the ratio between seismic rigidity of the soil and that of the construction

$$\gamma = \frac{\rho_r V_r}{\rho_3 V_3} \qquad (10)$$

when $\gamma - 0$ and $\gamma - \infty$ we obtain R=1, and T=0. That means that all the energy of seismic wave when it is reflected from the border is returned to the first medium, that is to when $\gamma = 1$ we obtain R=0, T=1. That means that all the energy of seismic wave passes to the construction from the soil.

The data above allow us to make some important conclusions about the peculiarities of seismic action upon the constructions:

- the more seismic rigidity of construction, is the more is the part of the seismic energy passed from the soil to the construction.
- on the same soil and at the same earthquake intensity the energy density in constructions of the III type can exceed the energy density in constructions of the I type;
- the part of seismic energy, transfered to the construction, with respect to the whole energy of the soil is becomming greater with the decrease of seismic rigidity of the soil. On constructions, errected on the dense limestone and sandy soils, the seismic energy is for several times less than that on mellow sandy-clayly soils. This is the characteristic feature of all the types of the constructions.

A METHOD TO EVALUATE DYNAMIC INSTABILITY OF SATURATED SAND. Nobuchika Moroto (Japan)

Since the catastrophic failures of saturated sand deposits occured during the recent earthquakes, a large number of studies of the liquefaction of saturated sand have been presented. Their eyes have been mainly focussed on the effective-time relation by laboratory dynamic tests. The author would like to examine the dynamic instability of saturated sand and sand deposit in a different manner from the current works.

Particulate material changes its volume in shear. The volume decreasing nature of loose sand under undrained condition can be directly related to the development of pore-pressure in saturated sand. When a sand sample is consolidated and then sheared, the sample first contracts and then tends to dilate. Soil in level ground which remained undisturbed since deposition is in the state of at rest. In this paper, the maximum amount of volume decrease of sand in shear becomes interesting. The maximum amount of decrease of void ratio can be roughly expressed as

$$e_o - e_c = C\,I, \quad I = K_o - K_c \tag{1}$$

where e_o is void ratio at rest,
e_c is void ratio at minimum volume condition during shear,
K_o is stress ratio at rest,
K_c is stress ratio at the minimum void ratio,
C is a constant and
K is expressed by O_3/O_1 (O_3 is lateral stress and O_1 is axial stress in the conventional triaxial test).

The relative density has been used as a measure of sand liquefaction potential. If the critical void ratio e_{cr} could be specified and easily determined, $e_o - e_{cr}$ will be the best index to estimate the liquefaction potential in principle. However the existence of the critical void ratio under dynamic condition seems to be uncertain. Then the author tries to use $e_o - e_c$ instead of $e_o - e_{cr}$ in estimating the dynamic instability of saturated sand. In Eq.(1), $e_o - e_c$ is expressed in terms of $I = K_o - K_c$. We will call this I as an instability index of saturated sand.

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The author conducted on constant stress ratios tests on sand and knew that the negative dilatancy was closely relating to the instability index. Theories and practices show that the earth pressure coefficient at rest K_o can be related to the angle of shearing resistance ϕ'. Constancy of K_c has been observed for various void ratio, grain size and stress path. Then the instability index can be written to be dependent almost on the earth pressure coefficient at rest K_o or the angle of shearing resistance ϕ'. We write, therefore, as

$$I = K_o - \text{constant} \tag{2}$$

Adopting the Jaky's empirical formula, We get

$$K_o = 1 - \sin\phi' \tag{3}$$

Using this relation, we have

$$I = \text{constant} - \sin\phi' \tag{4}$$

These expressins are very interesting. Because an earth pressure coefficient and a parameter of shearing resistance of sand have close relations to the dilatancy characteristics and consequently they would become attractive measures of the dynamic instability of saturated sand.

The angle of shearing resistance ϕ' is obtained as a function of void ratio e. Usually sand sample is prepared by pouring sand into sample former filled up with water. In this case, the ϕ'- e relation has been known to show smooth curve. While in the case that sample is obtained applying a very slow upwards flow of water through wet sand placed loosely in sample former, the ϕ'-e curve shows a peculiar aspect. Bjerrum, Kringstad and Kummeneje (1961) reported appearance of a sharp descent in the curve of ϕ'-e for the sand sample obtained by the latter sample forming method. Such a sharp bent of the curve was also observed at two laboratories in our country. It is noticeable that the sharp bent of the curves starts from points of $\phi'=32^\circ$-34°, and the pore pressure coefficient at failure A_f takes naught near the bent points of the curves. The author regards the values of $\phi'=32$-34° as an indicative criterion in estimating the instability of saturated sand.

Standard Penetration Test has been commonly employed in the field investigation. The penetration resistance N-value has been used to estimate stiffness and strength of soil deposits. Angle of shearing resistance has been correlated with the N-value. A number of engineers proposed ϕ'-N relations that show there are variances in their expressions. Angle of shearing resistance, in itself, has been known to vary some extent depending on types of apparatus and testing methods. Then considering summerily these proposed relations, we present the following formula:

$$\phi = \sqrt{13N} + 20 \tag{5}$$

This formula gives the values of $\phi'=32^\circ$-34° for N=11-15.

Reports on damage to structures by the Niigata Earthquake (1964, Japan) showed that saturated loose sandy deposits with less than N=12 at 8 meters deep from the ground surface were suffered from the heavy seismic damage (Osaki, 1966), and that harbour facilities rested on the layers of less than N=15 were seriously damaged, especially of less than N=10 (Hayashi, Kubo and Nakase, 1966).

Seed and Peacock (1971) collected many data of past earthquakes experienced in sandy deposits. Referring to the seismic intensity, occerence of liquefaction, ground condition in terms of average N-value of liquefied zone, the author can obtain the following formula of the critical N-value N_{cr} for sand liquefaction as

$$N_{cr} = 80\,k_s + 2 \qquad (6)$$

where k_s is the seismic coefficient. This give $N_{cr}=10 \div 14$ for $k_s=0.15 \div 0.20$. These N_{cr} are remarkably coincident with the N-values obtained by Eq.(5) for $\phi'=32^{\circ}-34^{\circ}$.

An effect of upwards flow of water on liquefaction of sand deposit is disregarded in usual treatments. Therefore let us check the effect of seepage force on the liquefaction likelihood of simplified saturated ground. To a depth of about ten meters from the surface, increment of stress ratio due to horizontal acceleration K_s at any depth can be assumed as follow:

$$K_s = A_s/g = k_s \qquad (7)$$

where g is the acceleration of gravity, k_s is the seismic coefficient and A_s is the maximum horizontal acceleration of seismic motion at the ground surface. Pore pressure u_d generated in the ground will be assumed to proportional to k_s. Taking into account inhomogeneity of natural ground, we write

$$u_d = D\,k_s\,z^b \qquad (8)$$

where D is a constant, b is an index of inhomogeneous soil profile and z is depth calculated from the top of ground water. The condition of one dimensional fully liquefaction can be easily written down as

$$\int_0^z (\gamma' - F_z)\,dz = u_d\ , \quad F_z = \gamma_w \frac{du_d}{dz} \qquad (9)$$

where γ' is submerged unit weight of soil, γ_w is unit weight of water. Substituting Eq.(8) into Eq.(9) gives the critical seismic coefficient k_{cr} as follows:

Case of $F_z=0$,

$k_{cr}=z/D$ b=0, $k_{cr}=1/2D$ b=1, $k_{cr}=1/2Dz$ b=2

Case of $F_z=0$,

$k_{cr}=z/D$ b=0, $k_{cr}=1/D$ b=1, $k_{cr}=1/Dz$ b=2

Then, the following liquefaction process may be supposed; (1) In case of b=0, liquefaction stars from the top of ground and then the lower parts lose the effective overburden pressure. In this way, the liquefaction zone will grow downwards successively. (2) In case of b=1, liquefaction likelihood is the same at any depth. (3) If b becomes 2, deeper part will be subjected to liquefy more readily than upper part.

If we consider the seepage force in the above model of ground, the liquefaction potential becomes twice of that in case of no seepage force. We can recognize that the effect of upwards flow of water on liquefaction is important.

Conclusive remarks cocerning the discussions in this paper are:

(1) Sand sample of laboratory test should be prepared by the way to reproduce sedimentation condition of sand deposit.

(2) The statically determined strength parameter ϕ' can be known to be an interesting index for dynamic instability of saturated sand. The values of ϕ' and e at which shoulder of sharp fall in the ϕ'- e curve locates and the pore pressure coefficient at failure takes naught in the $\phi'(e)$- A_f curve should be sharply paid attention to.

(3) The values of $\phi'= 32^{\circ}- 34^{\circ}$ in the conventional triaxial test are regarded as an criterion for instability of saturated sand. These values correspond to the values of N=11-15 in Eq.(5). The critical N-values of N_{cr}= which based on the observational data collected by Seed and Peacock remarkably coincide with those N-values of N=11-15. This coincidence is attractive.

(4) The loose sand deposit with $N < 10-15$ should be improved by vibro-compaction method for foundations of structures. When a work is carried out with no soil improvement, pilying may be planned. In this case, bearing capacities and lateral resistance of pile and piles should be estimated from the bottom of the liquefied zone. For case of $15 < N < 30$, the liquefaction potential may depend on soil type, soil profile, earthquake magnitude and etc.

(5) Inhomogeneous soil profile should be taken into account in the evaluation of liquefaction potential.

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DYNAMIC CHARACTERISTICS OF COLLAPSIBLE SOILS

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The prediction of additional subsidence, developed in some cases of seismic loads, presents certain difficulties due to the lack of a testing method, as well as instruments the simulate seismic action in determing the subsidence properties.

To obtain a tentative assessment of the possible deformation resulting from dynamic loads numerous cyclic tests were conducted (Musaelyan, 1971) according to various procedures (cyclic loads were applied: simultaneously with the development of subsidence, i.e. simultaneously with the moistening of the soils; upon partial deformation up to 50 per cent of that at static loads; or after the stabilization of subsidence). Independently of the method of applying cyclic loads, their duration was determined by the complete moistening of the soils and stabilization of the deformation.

Taking into consideration the various possible conditions under which dynamic loads may be applied, the proposed procedure is a more actual representation of the conditions of the moistening of the bases of buildings and structures when they are being used.

The use of different compacting loads, soils with various subsidence properties and different procedures for applying the cyclic loads revealed their influence on the nature of the development of subsidence with time, depending on the number of cycles of application.

Cyclic loads give rise to a further development of subsidence whose magnitude, depending on the soil properties, varies in a wide interval from the initial deformation to 20, 30 or more per cent (of the deformation at static loads). The time required for the stabilization of the additional deformation varied in a range from 50 or 60 to 300 or 400 or even more cycles. The main deformations were developed during the first cycles of load application.

As distinguished from the above-described cyclic tests (the compacting load is completely removed upon unloading), the somewhat different method, employed in the subsequent stages, extends the range of the conducted investigation.

In these tests with a given frequency only a part of the load is removed. This enables the observation of the degree of development of the additional subsidence depending on the magnitude of the removed load which we have called the drop in load and which is measured in kg/cm^2 (this varied in the tests from 0.5 to 1.5kg/cm^2 thereby constituting from 25 to 75 per cent of the static load).

Of most interest are the results of the dynamic tests (Musaelyan, 1972) in which the dynamic action at a given frequency was produced by alternating loads which we have also called the drop in load. This dynamic action imitates seismic forces acting on the soils of the bases of buildings and structures in seismic action.

Cyclic tests were conducted on modernized consolidometers. A dynamic consolidometer was employed for the dynamic tests. In both types of apparatus the parameters of action and the magnitudes of the loads was controlled by a dynamometer and the sensing elements of strain gauges with subsequent recording by an oscillograph.

In the tests, investigations were carried out in a range of frequencies from 1 to 5H and of load drops from ±10 to ±40 per cent (of the static load). The use of soils with various subsidence properties, different ranges of compacting loads (0.5 to 2.0 kg/cm^2) and various methods of applying the dynamic loads enabled the principal laws to be established for the development of additional subsidence upon dynamic loads.

An analysis of the numerous laboratory data obtained in the tests enabled the character of the development of additional subsidence to be revealed for dynamic loads of the indicated parameters which varied in a range from the initial deformation to 20 or 30 per cent (of analogical deformations upon static loads). Also determined to a considerable extent were the load drops and the physicomechanical properties of the soils. The frequency range within the limits investigated has no appreciable influence on the nature of the development of additional subsidence.

Cyclic and dynamic tests of collapsible soils conducted according to the procedure set forth enables changes in the dynamic characteristics of subsidence to be observed under definite conditions. This shows the expedience of taking them into account in certain cases when designing the bases of buildings and structures.

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LIQUEFACTION OF SATURATED SAND SUBJECTED TO COMPLICATED ALTERNATING SHEAR STRESS.

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Cyclic loading tests on saturated sand are commonly conducted under the idealized single shear stress conditions to obtain the knowledge of liquefaction. However, a soil element in the ground is subjected to complex system of shear stresses which will vary with time during an earthquake, and those shear wave conditions can not be simulated by repetition of idealized single wave form.

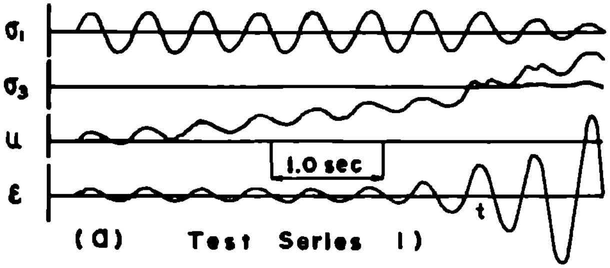
In an attempt to evaluate the effects of those complex system of shear stresses on liquefaction, Seed and Idriss(1971) proposed the average equivalent uniform shear stress corresponding to about 65 % of maximum shear stress occured in a soil element during an earthquake.

Shibata et al.(1972) conducted the triaxial tests in which different magnitude of shear stresses were applied to the specimen. Ishihara and Yasuda(1972) also conducted the triaxial tests in which they used the irregular shear wave form. From those test results, they obtained the smaller equivalent shear stress than that proposed by Seed and Idriss.

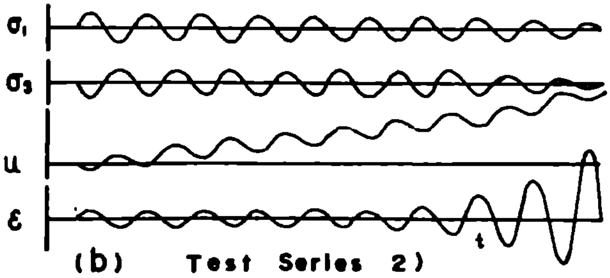
The auther also carried out the triaxial tests with application of alternating vertical and lateral stresses of different frequencies and magnitudes. As the results of these stress conditions, more complicated shear stress wave form on the 45° obliqued plane was obtained.

In this investigation, it was also discussed about the effects of over-consolidation on liquefaction.

A sample used in this test is Niigata sand. The specimens of saturated sand were consolidated under ambient confining stress σ_c = 2.0 Kg/cm^2 or 1.0 Kg/cm^2. After consolidation the confining stress was increased to σ_o = 2.0 Kg/cm^2 with undrained condition and back pressure u_b = 1.0 Kg/cm^2 was applied just prior to the start of test. Thus, some specimens were under over-consolidation; OCR=2.0, and others were normally consolidation; OCR=1.0.



(a) Test Series 1)



(b) Test Series 2)

FIG.-1 TYPICAL PATTERNS OF THE RESULTS OF TEST SERIES 1) AND 2)

Relative density of specimens was 55 % after consolidation.

Stress conditions used in these tests were as follows; 1) The magnitude of lateral stress was constant during the test and only vertical stress was alternated, 2) Vertical and lateral stresses were alternated with same frequency and difference of phase of 180°, 3) Vertical and lateral stresses were also alternated and ratios of their frequencies were 2:1 or 5:1.

Fig.-1 shows the typical patterns of the test results of test series 1) and 2). These results show that the behavior of sands in these shear wave test are same as those in squre wave test shown by Seed and Lee(1966).

Fig.-2 shows the typical pattern of the results of test series 3). Axial strain also increases suddenly and shear stress decreases at the onset of liquefaction. But variations of the pore water pressure do not correspond to that of shear stress wave owing to the influence of variation of confining stress.

From the results of the test series 1) and 2), it is evident that the relation between pore water pressure increment per stress cycle (Δu) and τ_d / σ_o' can be given by

$$\Delta u = a\left\{\left(\frac{\tau_d}{\sigma_o'}\right) - \left(\frac{\tau_d}{\sigma_o'}\right)_{crit}\right\}^2 \qquad (1)$$

where τ_d is an alternating shear stress on the 45° obliqued plane, σ_o is an initial effective confining stress which is consistent with consolidated confining stress at the normally consolidation test, and $(\tau_d / \sigma_o')_{crit}$ denotes the critical stress ratio below which shear stress has not influence on the occurrence of liquefaction. This critical stress ratio has same meaning as proposed by Shibata et al.(1972).

In the test series 3), it is very difficult to determine the magnitude of shear stresses and the number of alternation for obtaining the relation (τ_d / σ_o') - N_1 because different amplitudes of shear stresses are combined.

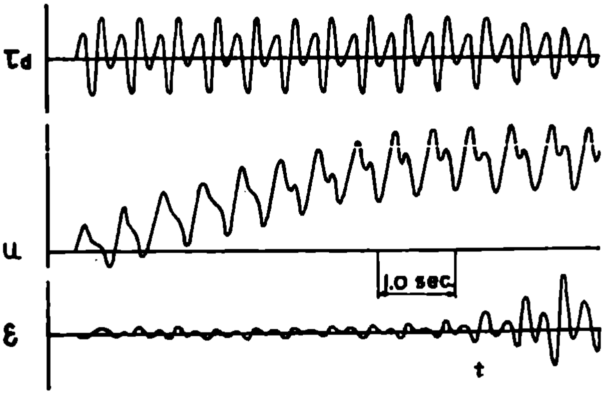


FIG.-2 TYPICAL PATTERN OF THE RESULTS OF TEST SERIES 3)

Therefore, to compare the results of test series 3) with those of test series 1) and 2), equivalent shear stress and number of application N_1 are assumed as follows;

$$ {}_a\tau_d = (\sum \tau_{di} \cdot n_i) / N_1 \qquad (2) $$
$$ N_1 = \sum n_i $$

where τ_{di} is the alternating shear stress which is larger than critical shear stress in equation (1), and n_i is the number of application of τ_{di} before initial liquefaction will occur.

The relation of ${}_a\tau_d / \sigma_0'$ and N_1 obtained on these assumption are shown on curves 1) and 2) in fig.-3, and they are quite close with those of single wave test. Thus, we can evaluate the liquefaction condition of sand deposit during earthquakes from the results of single wave test by using the relation expressed in equation (2)

As shown by curves (1) and (2) in fig-3, the alternating shear stress required to cause initial liquefaction under over-consolidation at a given number of cycles is higher than that under normally consolidation condition.

Assuming that the pore water pressure corresponding to back pressure is developed by application of alternating shear stress, a number of repetition of shear stress required to cause them can be obtained as follows;

$$ N_c = u_B / \Delta u $$

in which u_B is the value of back pressure. By adding N_c to the number of application of shear stress N_1 obtained from the test run under the over-consolidation condition, we can obtain the corrected number of application.

$$ {}_mN_1 = N_c + N_1 $$

The modified relations of τ_d / σ_c and ${}_mN_1$ are quite consistent with those of normally consolidated sand as shown on curve (2) in fig.-3.

The investigation described herein was conducted to determine the effects of irregularity of shear stress wave and over-consolidation of sand on liquefaction condition and the following conclusion were obtained.

1) The relation of ${}_a\tau_d / \sigma_0'$ and N_1 of saturated sand subjected to irregular shear stress wave form is quite close with that of single wave form by using stress and number of application expressed by equation (1).

2) The shear stress required to cause initial liquefaction of saturated sand at a given number of cycle is higher under over-consolidation condition than normally consolidation condition. These differences can be corrected by assuming that the pore water pressure corresponding to back pressure are occured by application of cyclic shear stresses.

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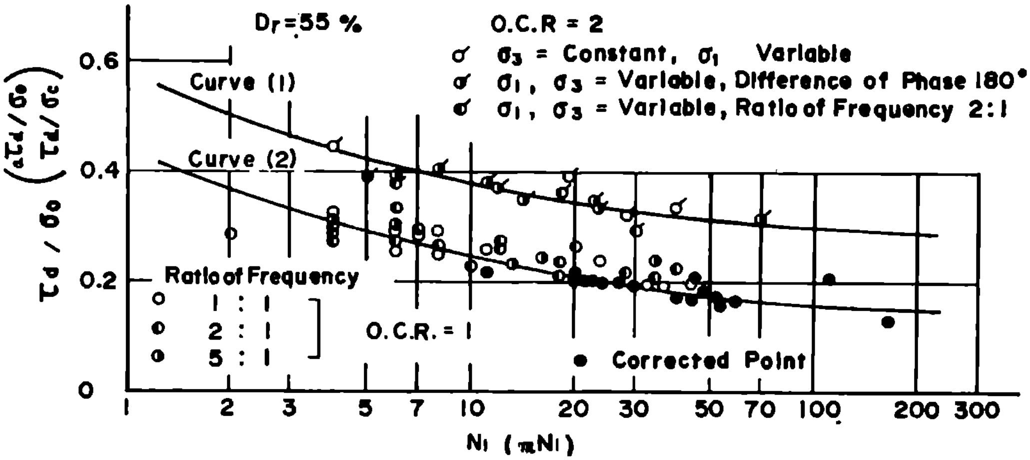

FIG-3 RELATION BETWEEN THE STRESS RATIO AND THE NUMBER OF APPLICATION REQUIRED TO CAUSE LIQUEFACTION

THE PROPAGATION OF RAMMING VIBRATIONS AND THEIR EFFECTS ON STRUCTURES.

W.Palloks /GDR/

If construction work takes place in regions densly built-up, we have to ask, whether pile driving near constructions may be permitted. At the "Forschungsanstalt für Schiffahrt, Wasser-und Grundbau" Berlin investigations on the danger of vibrations have been conducted for a number of special structural projects, such as the admissibility of pile driving immediately near subway tunnel, buildings and other structures /4/.

Some experiences we could gain are here reported.

Basic for these tests is the possibility to measure the vibration of the structure. The elastic waves in the soil which are produced by pile driving in general have frequencies in a range from 10 cps to 60 cps. For this range of frequency a measuring apparatus can ~~be set up with comparatively little expense.~~ We use either electro-dynamic vibration pick-ups or piezo-electric accelerometers with electric integration. In general we use the vibration velocity as measuring value. This appears reasonable for the estimation of damages from energetic considerations. Recordings are taken either by technical high-speed pen recorder, oscillographs on photographic paper or by tape recording. For paper recordings a frequency modulation device is used. This allows recordings to be taken in a frequency range from 1 cps to 1000 cps using normal tape recorders. Beside the pile the pile driving produces mainly transversal and surface waves, of which the vertical component usually is the greatest. Apparently the main part of the energy is transmitted as a longitudinal wave in the direction of the pile. This is so for vertical as well as for inclined piles. For the decrease of vibrations (vibration velocity v) with distance x the equation

$$v = a \cdot x^b$$

is suitable, which is proved by experiments for the distance range from 5m to 100m (Fig.1,2).

The value a is determined by the condition of ramming, essentially by the relation tup mass versus pile mass, and the sort of piles. The value b depends essentially upon the type of soil. In the "Forschungsanstalt für Schiffahrt, Wasser- und Grundbau" we try to solve this problem by exact tests.

Up to now we found, that the vibration in the ground is not proportional to the value of the ramming energy.

With the same type of pile and the same soil you may get larger amplitudes in the soil with less ramming energy if the relation $R=m_{tup}: m_{pile}$ is very low.

This means that a great part of the energy is lost for the ramming. Fig.3 shows such a result. The values given for one distance ~~were maximum velocities measured with the~~ same pile but with different depthes of the pile in the soil. Comparison between the Diesel Pile Hammers:
DELMAG D 22 und DELMAG D 12

Performance data:	D 22	D 12
ram weight (piston) t	2,2	1,25
energy per blow mkp	5500	3125
$R=m_{piston}: m_{pile}$	0,5	0,28

mass of pile 4,3 t
sort of pile: concrete, prestressed, 340x340x15000mm
soil : medium sand, dense
Fig.4 shows the position of this test

The same results were found when the ramming energy was produced by a falling weight. In all cases we found a relation between the movement s of the pile per blow and the vibration velocity v: $v \sim \frac{1}{s}$.
The frequency of the vibrations depends on the type of pile, the soil an especially on the relation $R=m_{tup}:m_{pile}$.

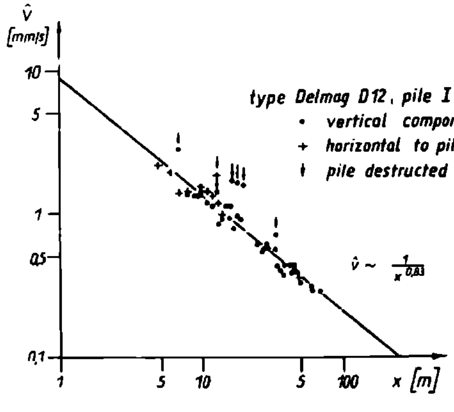

Fig.1Vibration velocity versus distance

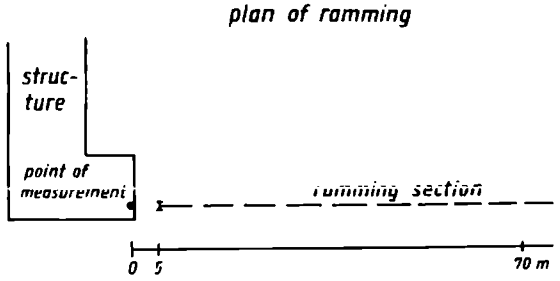

Fig.2 Plan of ramming evaluated in Fig.1

THE PROPAGATION OF RAMMING VIBRATIONS AND THEIR EFFECTS ON STRUCTURES. W.Palloks (GDR)

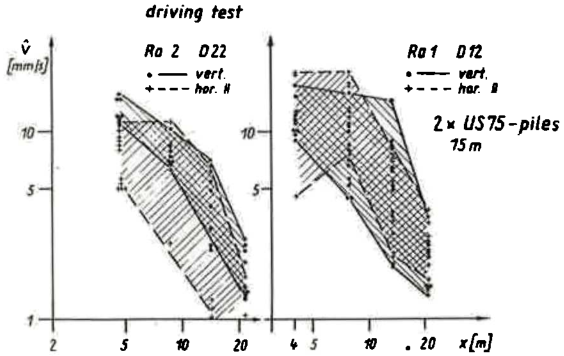

Fig. 3 Vibration velocity versus distance

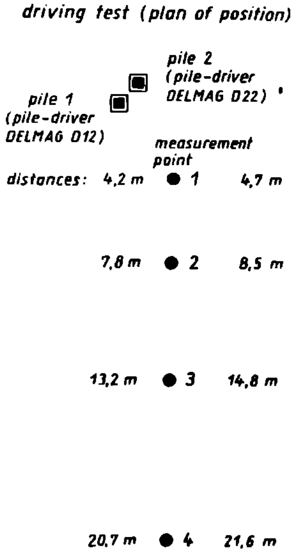

Fig. 4 Plan of ramming evaluated in Fig. 3

To evaluate the danger for buildings we tested the wellknown criteria, such as by CIESIELSKI /3/, BAULE /1/, /2/, DIN 4150 and others. It can be observed that, according to practical experiences, BAULE /1/, /2/ and DIN 4150used the best differentiation of building types. But the limits given by BAULE and DIN 4150 are too low for ramming vibration, because the frequency range in this case is very limited.
As a result of these researches of the author VEK Tiefbau Berlin, GDR now introduced the new "Werkstandard ITB-075". In this standard we also took into account, that the size of a building is one of the most important factors in all these vibrations problems.
Further investigations of this problem will require a consideration of the effects of vibrations on both constructions and bearing soil layers. /5/

5/64

Up to now our investigations mainly examined the effect of vibrations on constructions. These vibrations had to be accepted as given by technical reasons. The most important problem will be to avoid or to reduce vibrations at their source.

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GROUND MOTION OF SURFACE WAVES. S.M.Peng and J.D.Nelson (Thailand)

As part of an experimental program to investigate the isolation of surface waves by barriers, the orbits of motion of points on the ground surface were studied (PENG, 1972). The ground position at any particular time was determined by integration of horizontal and vertical velocity-time curves, measured using horizontal and vertical geophones. The soil was a compacted clay fill having a unit weight of 1.75 tons/cu.m. and a natural water content of 35%. The ground water level was at a depth of 0.40 m.

The theory of surface wave propagation predicts that the orbit of motion of particles located on the surface should be an ellipse having the major axis normal to the surface. The ratio of magnitudes of major to minor axes should be equal to 1.60 for $\nu = 0.37$ (corresponding to the soil used in this investigation).

Fig.1 shows the orbits of motion of soil particles recorded at different distances from the source of vibrations. The general shape of the orbits was elliptical although some deviations from an ellipse did exist. There were some instances in which at large distances the orbits of motion of the soil particles became a figure eight. This was also observed by BARKAN (1948). It can be seen that the major axis was not normal to the surface was predicted, and at some points it was parallel to the surface.

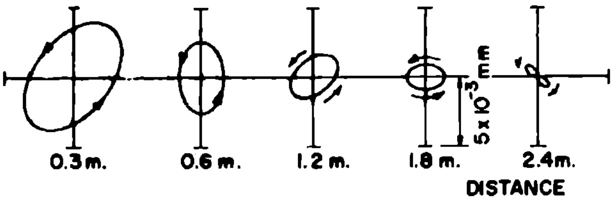

FIG.1 ORBITS OF PARTICLE MOTION
(f = 67 Hz, λ_R = 1.1 m.)

The distortions of the orbits observed at small distances from the source may be due, in part, to the influence of the P-waves and S-waves propagated simultaneously with the surface waves. Also, if the damping of the soil in the horizontal direction was different from the damping in the vertical direction, the phase angle between horizontal and vertical components of motions would change with distance. This would also explain why the inclination of the axis of the ellipse should have increased as the distance from the source increased, as shown in Fig. 2.

It can be seen from Fig. 2 that for all cases the inclination of the major axis rotated in a clockwise direction. The rate of change of inclination increased as the frequency increased, which is to be expected because the logarithmic decrement, and hence the energy absorption, is directly related to frequency (HARDIN, 1965).

The effects of trench barriers on particle motion orbits are shown in Fig. 3. The scale for particle motion amplitudes behind the trench is greater than that for in front of the trench. It can be seen that the amplitude of particle motion the ratio of vertical to horizontal axes and the angle of inclination of the major axis were changed by the barrier. In addition, within an area outside of the trench extending to a radius of about one wave-length the direction of particle motion was reversed.

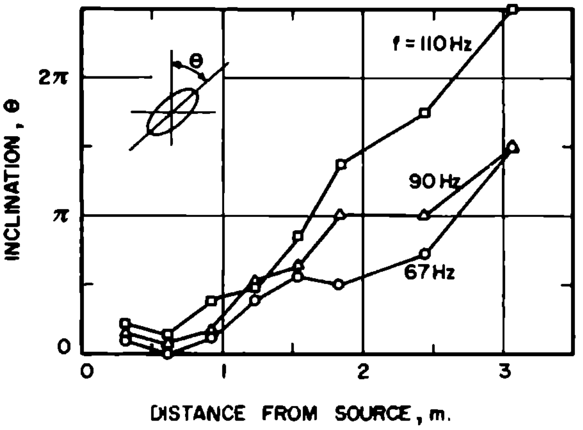

FIG.2 INCLINATION OF ORBITS

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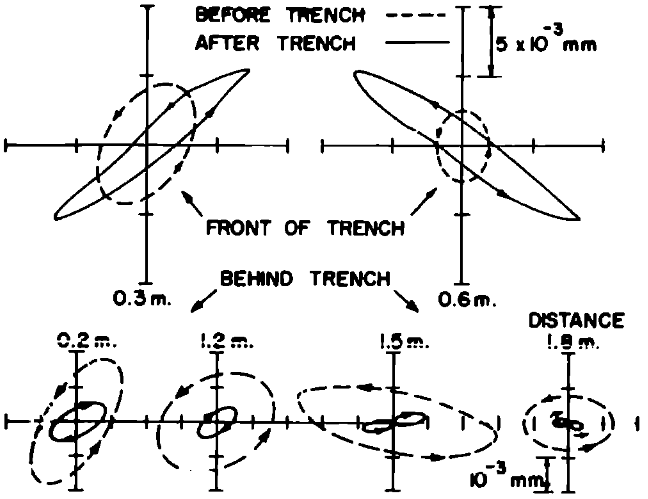

FIG.3 EFFECT OF BARRIER ON ORBITS
(f = 67 Hz, λ_R = 1.1 m.)

ON SOME COMPLICATED PROBLEMS IN THE DESIGN OF FOUNDATIONS.

O.A.Savinov, M.M.Klattzo, G.N.Stepanov, E.I.Tchasov
(U.S.S.R.)

The paper presents the results of the investigations into vibrations of a series of massive foundations for low frequency machines with periodic action as well as separate pile foundations for machines of different type.

1. Vibration calculations of a foundation series for low frequency machines

An approximate solution to the vibration problem of a foundation series resting on a common rigid base can be found out if soil inertia is neglected in the way adopted for a single foundation calculations. The following provisions make this assumption valid: a) foundation vibrations involve subresonance region so constrained vibration modes of the system are similar to displacements under disturbing force static action; b) lengths of the waves propagating in soil largely exceed the distances between foundations hence their vibration phase difference is usually small; c) mass of foundations (of usually basement type) are large comparing to a "reduced" soil mass involved in vibrations. A series of machine foundations situated on the same site can thus be regarded as a system of solid bodies resting upon a common elastic base, for example an elastic isotropic half-space. In this way a problem was formed and formulae received to determine vertical and horizontal rotatory vibration amplitudes for the foundations considering their mutual influence. The problems were solved using standard computer programs /1/. Vibration character variations of foundations installed in serios are as follows: vibration character of the loaded foundation significantly changes-natural frequency number of the system becomes equal to the sum of the foundation natural vibration frequencies. A region where resonance is possible is considerably expanded, particularly towards low frequency range; foundation vibrations of the source and the receiver may be comparable especially under the resonance. In the subresonance region vibration amplitude is similar to the product of the source vibration amplitude and the influence coefficientsof some value. Investigations of the foundation series for compressors and log frames showed the following: a) foundation vibration amplification accounting for the mutual influence of the foundations is possible when the machines are installed in a set; b) mutual influence of foundations becomes considerable in the zone equal to 5 times the foundations size; c) calculation data and experimental results show close agreement.

2. Pile foundation vibration design

In /2/ a pile foundation is reproduced by a distributed parameter system. The solution yields expressions for determining the modes and frequencies of free vibrations as well as the amplitudes of forced vertical vibrations (induced by both periodic and impulse forces). In the above papers this problem is also solved approximately where a compound system with distributed parameters is represented by a one degree-freedom system characterized by a reduced stiffness coefficient K_{zr} and mass M_r, both being functions of elastic material properties, the number and size of piles as well as the elastic properties of the soil in the pile penetration area and under the caps. The problem evaluating foundation behaviour under horizontal periodic forces is also solved approximately. The equations are solved versus the horizontal component of the displacement of the foundation base centre and its rotation angle round the axis through this centre and are expanded into natural vibration modes. Foundation characteristics are determined in the following way: a) Estimating for vertical vibration yields:

$$M_r = m + m_r; \quad K_{zr} \doteq K_z \gamma_{0},$$

in which m = mass of machine and foundation mat; m_r = pile and soil reduced masses taken approximately equal to the pile mass in the design; γ_0 = stiffness coefficient of a system with distributed parameters reduced to a one degree-of-freedom system.
b) Calculating for horizontal-rotational vibrations:

$$M_r = m;$$

$$\theta_{cr} = \theta_c + \frac{m_r}{n}\sum_{i=1}^{n} r_i^2; \quad \theta_{0r} = \theta_{0r} + m h_z^2$$

where n = number of piles in the foundation, r_i = distance from the ith pile axis to the rotation axis of the base of the foundation mat; θ_c - inertia moment of the foundation mat and the machine masses in relation to the axis through their common centre of gravity perpendicular to the vibration plane; θ_{cr} - reduced moment of the system inertia mass versus the same axis; θ_{0r} - the same versus the rotation axis of the foundation mat base. The stiffness coefficient of a pile foundation and the coefficient of energy absorption into the foundation are determined from additional expressions, diagrams and tables. Comparison of the test and analytical data is presented in Fig. 1.

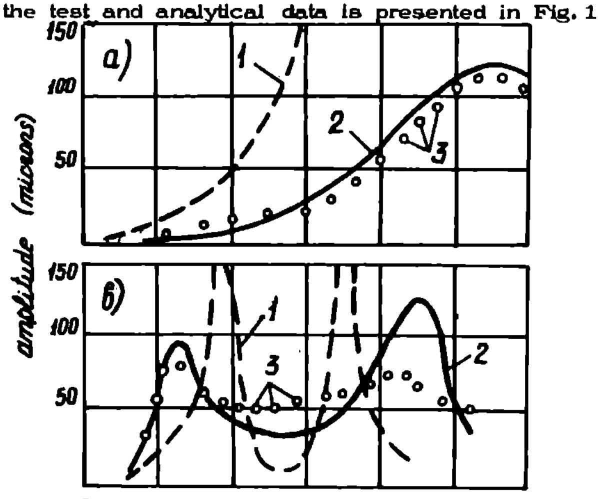

Fig. 1 Amplitude-frequency vibration curves of the experimental pile foundation (a - vertical, b - horizontal)
1-calculated by the design standards;
2-calculated by the method presented;
3-experimental points.

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THE DESIGNING OF THE PILE FOUNDATION WITH THE INTERMEDIATE CUSHION.

Shekhter O.Ya., Fayans B.L., Ukolov V.N (USSR)

The intermediate cushion (made of sand, or crushstone, or some other material according to the soil condition of the building site) being placed between the foundation block base and pile cups, the horizontal loads transmitted to the piles from the inertia forces of the building are small and they can been neglected while pile designing. At the same time such foundations (proposed in Chile) provide the necessary bearing capacity for the vertical loads, which however must be determined by calculation. In order to define more precisely the bearing capacity of the pile foundation with the intermediate cushion it is desired to make static experiments with one pile foundation, what is however can hardly be done in full details for making clear the influence of set of parameters of the soil, the cushion and the pile/ on the stress distribution in the cushion base, foundation block base and of the settlements of the foundation block and the pile. So the elaboration of the theoretical methods is needed.

For this purpose we propose a designing scheme, shown on Fig. . A rigid circular plate placed on the surface of the intermediate cushion, considered as an elastic infinite layer with thickness h, is vertically loaded The tangential stresses on the both surfaces of the intermediate cushion are supposed to be equal to the zero. On the contact surface between the intermediate cushion and the pile cap and the soil outside the cup the contact stresses are supposed to be proportional to the displacements, C_{z1} and C_{z2} being subgrade reaction coefficients, corresponding to the pile cap $(0 \leq r \leq 1)$ and for the soil outside of it $/1 \leq r \leq \infty /$. It must be noted that C_{z2} may depend upon the cushion thickness

The solution of the axialsymmetrical problem of the rigid plate is constructed by means of solutions for the set of the uniformally loaded rings (or more exact circular loadings) forming a rigid plate. the unknown loads of these rings then are determined by using prof. Zhemochkin's method. The approximate solution of the uniformally loaded circular plate is found by means of the integral equation

which is the result of the boundary conditions. The solution of this integral equation is obtained approximatelly by the expansion of the unknown function into series of some chosen functions with unknown coefficients.

At present there is the programme for calculations with the use of a computer. The parameters of soil, pile, cushion being given, this programme permits to evaluate all the data necessary for making clear the behaviour of the foundation type under consideration.

For example let us take the following parameters. The radius of the foundation block R=60cm, the cup pile radius l=30cm, the total load applied to the foundation block P=45 tons, the subgrade reaction coefficients C_{z1}= 5,10,20kg/cm^3, C_{z2}=1 kg/cm^3, the elastic modulus and the coefficient ratio of the cushion material are $E=300 kg/cm^2$, $\nu = 1/3$. The results of the calculations are shown on Fig. On the abscissa axis the values of h are given. On the ordinate axis the values of the ratio in % between the load acting on the pile and applied to the foundation block, as well as the settlements values of it are given. It can be seen that the effect of the yielding pile coefficient Cz1 on the foundation block settlements is significant when h is comparatively small. The layer thickness and yielding of the pile considerably influence the value of the load part transmitted to the pile.

The results are in good agreement with the data of the field experiments conducted under the guidance of prof. Barkan D.D.

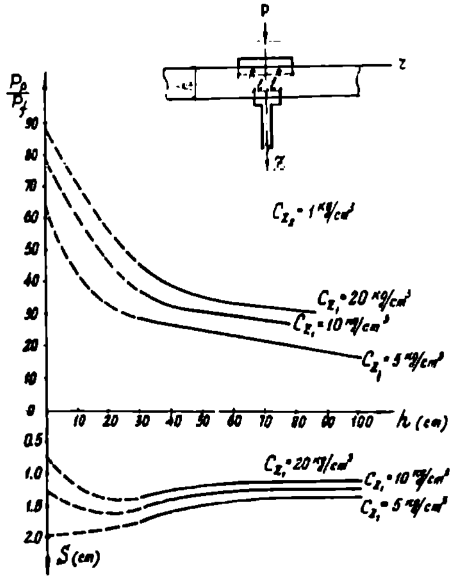

Fig.1

The dotted lines are the continuations of calculation data to their limit values.

SEISMIC ENERGY TRANSMISSION INTO TWO LAYERED SOIL FOUNDATION. Sinitsyn A.P./USSR/

The seismic effect of an earthquake hearth is propagated to the structure through a layered soil foundation. The propagation of seismic waves in layered media has been studied quite fully. But the latest investigations (1) have shown that the additional investigations must be carried on the soil foundation in which there is a stratum with low velocity. This stratum has the elasto-plastic properties and is described by bilinear diagram. The upper sedimentary stratum on which the foundations of the structure are placed is a stratum with low velocity. The energy flux is a good generalized parameter to determine the influence of the sedimentary stratum properties on the transmission of seismic effect to the structure foundation. The value of seismic energy flux transmitted through the sedimentary stratum depends on the physical and geometrical characteristics of the stratum. The essential influence on the distribution of displacements and accelerations on the surface of the sedimentary stratum exerts not only the thickness of the stratum but the inclination of the rock foundation which spreads the sedimentary stratum too. Using the finite method it is possible to determine the overload coefficients to the surface of sedimentary stratum. The overload coefficient is called the multiplier by which the wave effect propagated through the rock foundation must be multiplied for the evaluation of the wave effect corresponding to a given surface point of sedimentary stratum. This coefficient in some cases may have a great value. For example the value of accelerations on the surface of sedimentary stratum may be twice as much as the accelerations of rock foundation. The graph of alteration of the overload coefficient for the maximum accelerations of the surface of the sedimentary stratum is shown in fig.1. It is the case when sedimentary stratum has changeable thickness as it is in a valley. From the graph it is clear that the sedimentary stratum may both increase or decrease the acceleration values which reach a given point of rock foundation. Thus, the engineering geological structure of the region in which the earthquake energy propagates, influences greatly the value of effect which is transmitted to the structure during an earthquake. In fig.1 three points are shown in which a building may be placed. In point 1 the amplitudes of the accelerations will be twice as much as in point 3 and two and a half times as in point 2. That way in such conditions it is worth while to place the building in point 2 and the aseismic resistance of this building will be bigger as compared to the case when the building is placed in points 1 or 3. The increase of aseismic resistance of the building is the result of a good choice of engineering geological conditions of this region and no special expenditures must be made. At strong earthquakes in dep layers of the upper mantle the propagation of seismic waves is complicated by the complexity of the stress conditions. The seismic waves change this stress conditions. The detailed investigations of this problem show (2,3), that in such complex stress conditions there may exist two velocities of elasto-plastic wave propagation. The first velocity C_f is related to the fast plastic waves. These waves appear when main stresses are the normal stresses caused by "P" wave. The second velocity C_s - is related to the slow plastic waves which are connected with shear waves, they appe when the shear stresses are great.

The indicated velocities satisfy the following nonequalities:

$$C_s < C_2 < C_f < C_0$$

The values of velocities C_f and C_s, which correspond to the plastic deformation front propagation, depend on values of and . In table I the values of these velocities are given in dimensionless form.

By the complex stress conditions the two plastic waves arise, the first is related to the longitudinal and the second to the shear seismic waves. The fast plastic longitudinal wave has velocity -C_f, and the slow plastic shear wave has velocity-C_s.

To correct application of the data given in Table I for the designing of structure foundation on aseismic resistance it is necessary to consider the preliminary stress caused by dead load of soil. This stress influences the loading of structure during the whole period.

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Table I

τ/σ	β=0.0I		β=0.40		β=0.90		Note
	Cf/C2	Cs/C2	Cf/C2	Cs/C2	Cf/C2	Cs/C2	
$0/\tau=0$	I,00	0,I6	I,00	I,00	I,52	I,00	$\nu=0.$
0,5	I,42	0,II	I,47	0,64	I,58	0,97	$C_0=$
I,0	I,55	0,09	I,57	0,57	I,60	0,95	$1.62C_2$
I0,0	I,6I	0,08	I,6I	056	I,6I	0,94	$\theta=2$
$\infty(\sigma=0)$	I,62	0,06	I,62	0,55	I,62	0,93	

β -the coefficient of deformation modul on the plastic range of diagram.

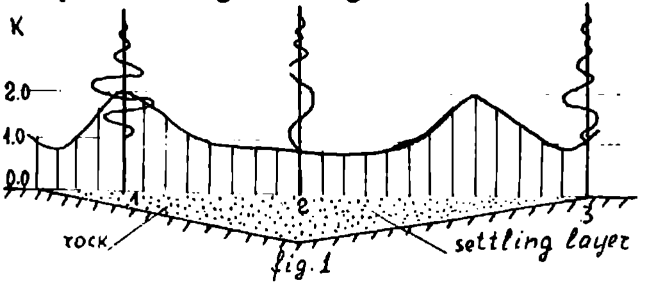

fig.1

B.O.Skipp (England)

Vibrations associated with collapse of structures

Between 1969 and 1972 seven large cooling towers were demolished in the United Kingdom by explosively removing their supporting legs. The energy available on impact was equivalent to a small earthquake of low magnitude and the duration of the impact and the ground response extended over several seconds. Because concern had been expressed on the integrity of adjacent structures and services when these cooling towers were demolished vibration monitoring of ground and structure response was undertaken. In addition on three occasions high speed cameras were mounted and the progress of impact followed.

The three sites on which these demolitions were undertaken had different geological conditions and the vibration results are interesting both from a seismological and engineering point of view since the services near to the points of impact included gas mains,oil filled 132 kv cable,and relay stations controlling major sectors of the power supply to a large city, and a main sewer. Structures near to the points of impact included bridges, railway embankments, roadways light buildings, cooling towers on piled foundations,and wharf frontage held back by tie bars.

The vibrations were measured generally by damped low frequency seismometers (geophones) feeding into U/V recorders and tape recorders together with strong motion seismographs (type SMAI).

At Thorn Hill. Yorkshire the cooling tower was 280 ft. high and weighed approximately 3,200 tons. The demolition was undertaken by removing supporting legs from 270° of the perimeter allowing it to fall by first tilting approximately 5°, whereupon impact took place upon the edge of the reinforced and thickened lower part of the shell and pond floor. The cooling tower itself was founded on a strip footing sitting on dense gravel some 40 ft. thick overlying Coal Measure sandstones and shales.

The levels of vibrations recorded were notable in that they were extremely high and the results are included as peak particle velocities in Fig. 1. Accelrometers installed at the points of impact on shell and on the pond wall gave values of acceleration between 36 and 40g. High speed photography (500 frame per second) indicated that the retardation upon impact was in excess of 25g.

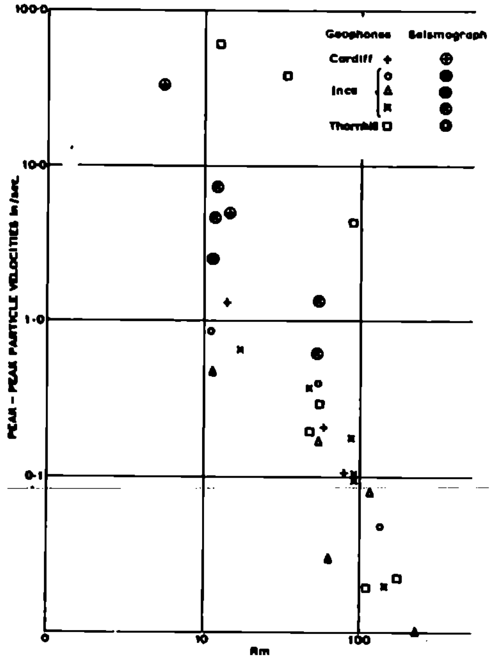

FIG. 1. PARTICLE VELOCITIES VERSUS DISTANCE FROM POINT OF IMPACT $\dot{u}^2 = \dot{u}_x^2+\dot{u}_y^2+\dot{u}_z^2$

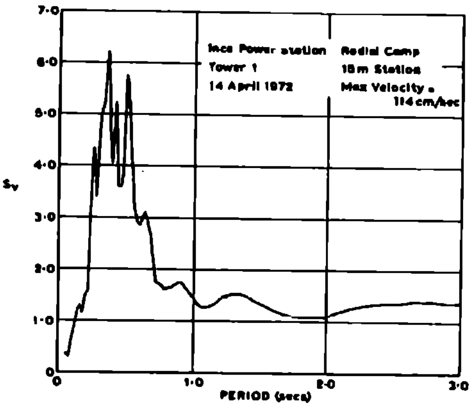

FIG. 2. RESPONSE SPECTRUM (RELATIVE VELOCITY)

At Cardiff Power Station in Wales the cooling towers rested on piles which were driven into weathered Keuper Marl at a depth of 30 ft. and the superficial deposits consisted of fill and loose to medium sands and gravels with a small amount of soft silty clay. The vibrations recorded here by both the geophones and by a strong motion seismograph were much smaller than those encountered at Thorn Hill.

Deceleration on impact was 3.0g. It was also noticed that the accelerations were in excess of 1g as recorded in the strong motion seismograph quite close to the point of impact (5 metres) but the materials in which the devices were embedded was a very loose fill extending to 5 metres below ground level. There was evidence of ground rupture up to 2m away from the edge of the cooling pond wall, that is some 5m from the presumed point of impact.

At Ince Power Station in Cheshire where the towers are on driven concrete piles the vibration levels were recorded on the collapse of three cooling towers each cooling tower having somewhat different soil conditions beneath it, but generally the ground consisted of peat and alluvial deposits with some boulder clay overlying weathered Bunter Sandstone. The presence of peat was most noticeable at No. 2 tower but the overall picture in terms of peak particle velocities has been included on Fig.1 and it can be seen that generally the results were more akin to those noted at Cardiff than at Thorn Hill. Deceleration on impact was 1.0g for Towers 2 and 3 and 4.0g for tower 4.

It is evident that the levels of vibration which can be expected from these demolitions depends a great deal upon the stiffness of the underlying soil. In conditions where piling was necessary for the founding of the structure it is likely that the levels of vibrations in terms of particle velocity will not be excessive. However where the towers have been able to be founded on pad footings it is probable that there will be a greater energy transfer into seismic form and correspondingly higher particle velocities. Nevertheless services buried at Thorn Hill sustained levels of vibrations well in excess of any of the standards normally laid down and it would appear that even in unfavourable conditions the vibrations set up by such demolitions are not likely to give rise to engineering difficulties. From the point of view of the ground response we are carrying out further studies on the power spectrum and the response spectra from the strong motion seismograph. Such a response spectra for a collapse at Ince is attached as Fig.2 and it is interesting to note how closely this follows the form associated with a typical earthquake. It is also interesting to note that such high particle velocities and accelerations could be sustained near the point of impact of the Cardiff Cooling Towers even though the upper layer was of such soft and unconsolidated material.

Acknowledgements

The work reported was carried out for Messrs. Blackburn Contractors and Ogden (Demolition) acting for the C.E.G.B. At Thornhill assistance was provided by Cambridge University Department of Geology and Geophysics. The strong motion seismographs were operated by Imperial College of Science and Technology, Department of Engineering Seismology.

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Foundation Acting as Seismic Shock Absorber and Preventing Buildings Against Destruction during Earthquakes

As is known, cities and towns are mostly comprised of buildings with brick walls.

Since brick walls are characterized by a very low earthquake resistance, large-scale construction is not earthquake resistant.

This fact is evident from great destructions caused to cities and towns by seismic forces during earthquakes in past years.

A specially designed foundation acting as a seismic shock absorber to be placed between the building and the earth can prevent brickwork buildings from collapsing and can contribute to saving cities and towns from destruction due to earthquake shocks.

The foundation consists of reinforced concrete structures of two types-Nos 1 and 2 (Fig.1).

Structures No.1 rest on the earth, while those of No.2 are suspended from structures No.1 by means of steel suspensions (C), with springs (H) at their ends and reinforced-concrete girders (P) (Figs 1 and 2).

The building walls (S) are carried by suspended structures No.2 (Figs. 1,2,4).

Thus the building does not actually rest on the earth but is freely suspended (Figs.2,4). Consequently earthquake forces (Q) will be acting on the suspension points, i.e. structure No.1 (Figs 1 and 2), rather than on the building proper.

Air gaps left between structures Nos 1 and 2 ensure absorbtion of lateral amplitudes of seismic vibrations of buildings during earthquakes (Figs 2 and 3).

Since the magnitude of seismic forces acting on building is dependent of the magnitude of vibration accelerations of buildings (with the acceleration decreasing the above forces are also decreasing and vice versa), the seismic force can be reduced by decreasing vibration accelerations to a value which is safe for brick buildings.

To avoid destruction of buildings the suspension length is, thereforce, increased which, results in increase of their vibration period.

The increased period of free vibrations of buildings will reduce both the vibration acceleration and the complex seismic force acting on the buildings, to a value safe for brick buildings. It will also eliminate a resonance phenomenon between the parameters of free vibrations of buildings and those of seismic vibrations of the earth.

Use of the above mentioned foundation will ensure safety of a brick building during earthquakes whose magnitude is 9 points and over by the Mercalli-Cancani scale.

In the USSR the described foundation has been used to construct a 3-storeyed brickwork block of flats (Fig.4).

The specific feature of the foundation in question has been confirmed by instruments during artificially-induced vibrations of the earth under the foundation (Fig.5).

The author has received the Inventor's Certificates Nos 53663 and 70385 for this foundation design.

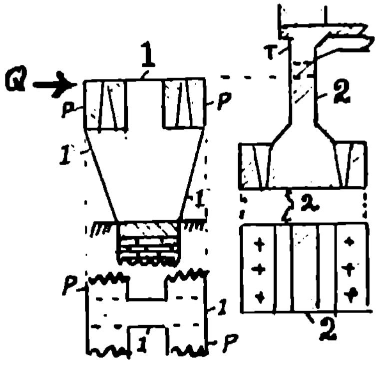

Fig. 1

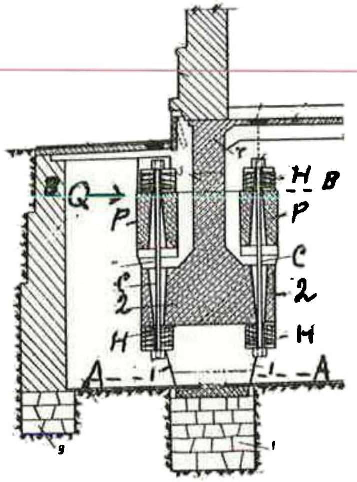

Fig. 2

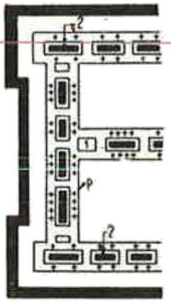

Fig.3

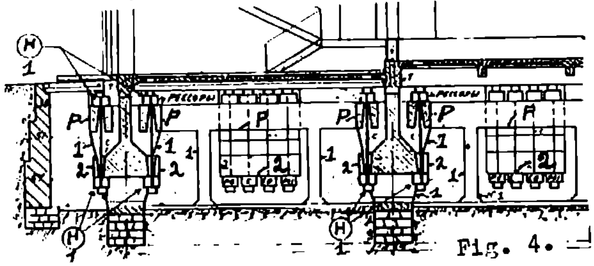

Fig. 4.

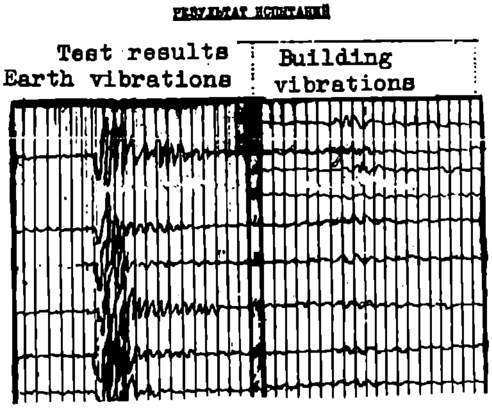

Fig. 5.