Session 10

Soil Dynamics

Dynamique des Sols

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V.A. Ilyichev, Chairman

OPENING REMARKS

I have the honour on behalf of the Organizing Committee to open the IOth Specialty Session on Soil Dynamics.I am glad to greet all participants of our meeting.

During last I2 years the interest to soil dynamics is permanently growing.Specialty Session at 8 International Conference in Moscow, USSR,in I973,the Main Session at 9 International Conference in Tokyo,Japan,I977 and now again the Specialty Session at IO International Conference here in the hospitable Stockholm,Sweden,are the striking illustration of it.

In average two conferences on Soil Dynamics were hold annually in the last four years. What does attract scientists,researchers at these international forums? To my mind the problems,which soil dynamics solved,solves and is going to solve,are very important in the engineering practice.

Soil dynamics reminds about itself even if engineers forget it. It should be noted with regret that people have to suffer much from earthquakes,destructing earth dams,embankments and structures,built on liquefacting sands in order to attract the attention of researchers all over the world to soil dynamics.

This branch of soil mechanics has been extremely developed during I5 years in connection with the exploration of off-shore soils.

The off-shore technology required new investigations in soil mechanics and especially in soil dynamics.

Constitutive equations of soils are the key to solve the practical problems. We can't speak about "dynamic properties" of soil without putting down the ade-quate mathematical term in the constitutive equation. And it seems sometimes that we have a lot of "properties".This impression of non-authors of these equations may be too individual. And joint experience has to help us to choose the acceptable theory or theories. I hope that this session will take a step in this direction.

Speaking about the achievements we should not forget not so impressive problems rather traditional but practically very important ones, such as dynamics of machine foundations. This problem was one of the starting points of soil dynamics,and now it will be useful to revise some recommendations,using a great experience and new theoretical approaches.

The problem of damage caused by vibrations is also traditional one. This problem draws the attention of practical engineers from time to time,and the solution of it is not always obvious and depends on many reasons.

All the problems mentioned above will be discussed at the Session,and I hope they will be interesting for all present.

Thank you for attention.

M.A. Erguvanli, Panelist

EVALUATION OF DYNAMIC SOIL PROPERTIES
Evaluation des Propriétés des Sols

Introduction

For the accuracy of predicting dynamic behaviour and for the improvement of a realistic design, the selection of the relevant soil parameters is of vital importance.

In spite of the complex character of the soil deposits and limitations in sampling and testing procedures, the designer is urged to obtain the reliable soil properties under the relevant dynamic loads (the cause of main problems: the undeterministic acts of Nature).

The determination of the dynamic design data, has special importance in regions where relatively little information regarding past experience exists but where the consequences of damage and risk are high.

The main aspects of determination and evaluation of dynamic soil properties together with the problems associated with testing will be briefly discussed herein.

Determination of soil properties

In a realistic design, emphasis should be given to the reliable determination of the dynamic soil properties. Tools that can be used for this purpose include:

a) observations at site,

b) field and laboratory testing,

c) consideration of case histories

Amongst these, only "testing" can identify the quantitative nature of the problem. For laboratory testing, it is imperative that the choice of appropriate test procedure should reflect the in-situ conditions, also regarding static as well as expected dynamic states of stress. At this point, attention should be payed upon the limitation and controversies of the widely used test procedures. However, recent developments in the techniques of laboratory and in-situ measurement have brought certain precision (3,5).

Problems associated with testing

The designer should be aware of the problems associated with different types of tests, to obtain the same soil properties. Besides, the assumptions made, boundary conditions, possible defects of equipment etc. are the other factors to be considered in the evaluation of raw test data. The values, coming from the relevant tranquility of the laboratory, should be used in caution and weighed with past experience prior to use.

Today (June 1981), when a broad observation is made on the world spread testing procedures, it becomes apparent that a thorough standardization is not yet achieved. Thus, well documented testing procedures should be emphasised upon and well defined "international" test standards should be brought into practice.

Certain critical reviews on different testing approaches, may briefly include,

(a) The well defined, small strain amplitude "Reasonant Column" test, with recent testing standards being prepared, still suffers from calibration problems of the system.

(b) The "dynamic triaxial", widely used tool in engineering practice and research, lacks documentation and no unique test standard is used. Testing still has to overcome (in certain equipment) the problems of the assessment of rod friction, location of the pwp and strain transducers, dimensions of the tested sample thus types and maximum grain size of the soil to be tested etc, which are commonly encountered in laboratory testing.

(c) "Simple shear" testing has similar problems as the dynamic triaxial, together with objections to the development of non-uniform stress distribution during dynamic testing.

(d) Large amplitude hollow specimen "torsional shear" has similar problems of no unique standardization, but samples being hollow, testing gives a better defined and more uniform state of stress during vibrations.

(e) Field tests, naturally, are made in the in-situ stress conditions. The sampling defects, and fabric and stress anisotropy conditions of natural soils, which have to be overcome in laboratory testing, is solved directly by field tests. But, in-situ test data is obtained, at present, in the small strain amplitude range - a serious limitation!

It is apparent that in-situ and laboratory test data should be correlated, and still clear definitions are required for all testing procedures, in order to be able to evaluate the results reliably (2,3).

Evaluation of soil properties

In certain cases, the designer faces the problem of deciding for the choice of which type of test result to apply.

Primarily, the designer should realize the existence of the assumptions made for the idealization of actual soil system. Secondarily, the consistency of these assumptions with the conditions of the applied test procedures should be verified (4).

For example, in field conditions, lateral restrainment for deformation under dynamics (earthquake) loading is often difficult to specify, as may be in saturated slopes of fill dams, reclaimed areas etc.

For the assessment of initial liquefaction, if dynamic testing is applied on anisotropically consolidated specimens, with no constraint for lateral deformation during dynamic loading, (Fig 1a) it is observed that the pwp's do not rise to the value of confining pressure, thus initial liquefaction does not occur (1). Where as, similar test, with samples laterally restrained, yields to the pwp build up resulting in initial liquefaction (Fig 1b). The interpretation of these test results and their application to the behaviour of natural deposits under anisotropic states of stress, becomes the main task of the designer in evaluation (2). This example exhibits the importance of the above argument of the verification of the consistency of in-situ and laboratory conditions.

"Nature doesn't reveal all her secrets at once". Seneca.

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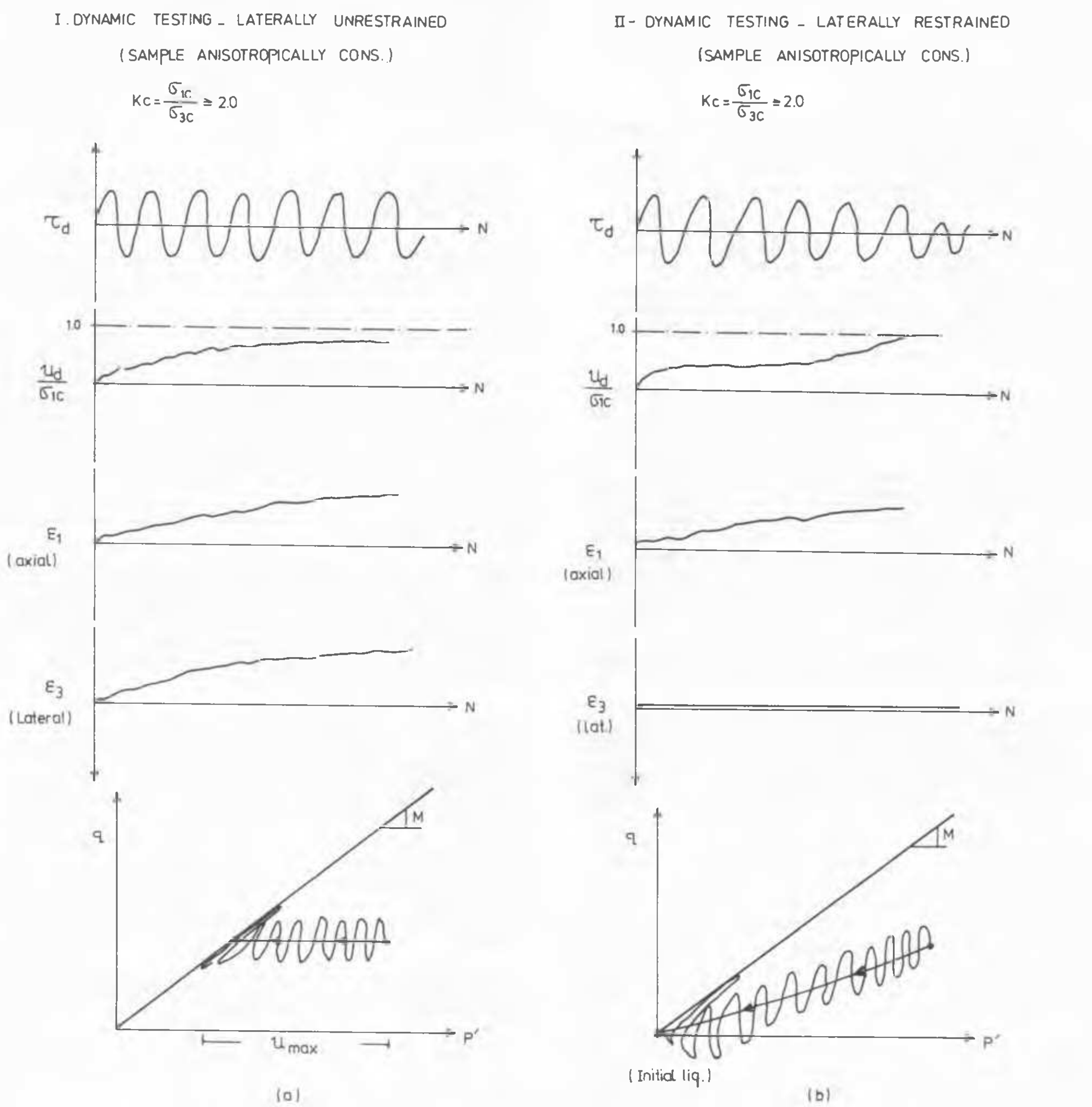

FIG. 1 _ TYPICAL TEST DATA AND q-p′ DIAGRAMS FOR ANISOTROPICALLY CONSOLIDATED TRIAXIAL SAMPLES, UNDER LATERALLY UNRESTRAINED (a) AND RESTRAINED (b) CONDITIONS , DURING DYNAMIC TESTING.

J. Studer, Panelist

DYNAMIC PROPERTIES OF SOILS:SOME BASIC QUESTIONS

In this contribution attention is drawn to the need for further research on the stiffness and damping properties of coarse granular soils, in particular of gravels and rockfill material. The published technical literature furnishes very little data for cohesionless soils in the range of gravels to boulders size. In practice, however, such materials are often encountered as a construction material (e.g. rockfill) in dams or in the subsoil.

Established field and laboratory testing methods to determine the dynamic properties of soils and rock have been used also for coarse materials in recent years. For shear strains in the low strain range ($< 10^{-4}$%) the cross-hole test is probably the most accepted field test, while the resonant column (RC) test is most commonly used in the laboratory. For medium to high strains there is no field test which is really appropriate and one has to rely on laboratory tests, namely on the cyclic triaxial and the cyclic simple shear tests.

To test a granular material in the laboratory the sample size should be at least 5 (better 10) times the maximum grain size, which requires relatively large sample sizes. For example, a 150 mm diameter sample should not contain sizes of grains exceeding 30 mm. Few laboratories can test under cyclic conditions triaxial samples of bigger diameter, which means that larger components have to be removed from the material, and the resulting effect on material properties has to be considered. Thus the first basic question to be answered is

- In how far is a sample of limited grain size representative of the field conditions?

In practice, it is extremely difficult to obtain undisturbed samples of coarse granular material and one attempts to compact the specimen in a mould to achieve a density equivalent to the field condition. The old question always arises of the representativeness of compacted samples. There is considerable doubt as to being able in the laboratory to reproduce the natural matrix (interlocking behaviour) and the cementation of grains, with the associated aging effects. The influence of the latter differs according to the range of strains. With regard to the rockfill in a dam the problem is of less concern than for a foundation material, but it still exists (Seed, 1979 Rankine lecture).

Another factor of relevance here is that of grain crushing. By limiting the grain size in a sample the effects of grain crushing will usually be underestimated. This will influence in turn the stiffness and damping properties.

Much work in the area of the static monotonic loading of gravels and rockfill has been carried out, especially with regard to adjusting the grain size distribution curve to model the in-situ material characteristics. The experience gained here could be of great help in cyclic loading investigations.

Apart from the fundamental questions raised above it should be remembered that there also exist problems in connection with the test equipment to handle large samples and the greater amount of effort required in the preparation of the sample itself. As a result there are not many laboratories with adequate facilities and due to financial reasons usually only a small number of tests are carried out. The technical difficulties met with when increasing the size of the testing equipment should never be underestimated.

The second basic question to be considered is:

- How does one obtain in a consistent manner the shear modulus G and damping properties over the whole range of shear strain?

As mentioned above, different methods of testing are used for low and high ranges of strain. In the low strain range shear wave propagation methods with frequencies of several tens to several hundreds of Hertz are employed, whereas at higher strains the frequencies may be few Hertz or less. This involves thus a difference by a factor of about 100 or more. Fortunately, experience shows that for cohesionless soils the frequency of loading is of minor importance. However, care should be taken to adapt, if possible, the testing frequency to the actual loading conditions in situ. For blast loading, which involves high frequencies, the published data is very scant.

Summarizing our knowledge of dynamic (equivalent linear) soil properties for coarse granular materials, it may be stated that, qualitatively, a gravel behaves like a sand (and for the damping factor the two materials are quantitatively not very different).

Essentially gravel can be treated as a stiff sand (but the stiffness may exceed that of the stiffest sands for well-graded dense gravels). The damping curves fall within the same range. Nevertheless, the empirical equations obtained for sand should not simply be taken over for gravels. This point is brought out in a very interesting paper by Prange (paper 10/22) describing the behaviour of a ballast material in a large RC apparatus, allowing a sample size of 1 m diameter by 2 m height (max. grain size ≈ 70 mm).

Prange gives the following equation for shear modulus variation at strains $\gamma < 10^{-6}$

$$G_{max} = F_1 (e) (\sigma'_m)^{0.38}$$

in which e = void ratio.

This formula is similar to the one proposed by Hardin and Drnevich (1972).

The writer has also investigated well graded gravels (max. grain size < 30 mm) in the RC device. The influence of the mean effective stress σ'_m was found to be much stronger, i.e.

$$G_{max} = F_2 (e) (\sigma'_m)^{0.6}$$

Further, the stiffness value was much higher than predicted by Prange's equation. The reasons for the discrepancy in the results should therefore be further tested. It may be purely a case of two widely different materials and compaction despite exhibiting gravel size range.

In the medium-high range of strain for the well-graded gravel the triaxial test equipment delivered results showing that the exponent 0.5 gave a reasonable fit over the whole range of strain.

Some discrepancies in test results can also arise when comparing field and laboratory data. An example of this is provided in the following.

The laboratory tests described above were carried out for a nuclear power plant site. For the same project shear moduli were also determined by the crosshole method.

In Fig. 1 the full curve represents the laboratory test data. It is noticeable that only a relatively small scatter is obtained for data points. The point A shows the result for the crosshole test, which is about twice as large as an extrapolated value for the triaxial test data. This difference is quite high but not unusual.

There are various possibilities of combining the laboratory and field data to obtain a representative description (i.e. a single curve) of the soil properties. This procedure must be carried out carefully as the influence upon dynamic calculations is important. Because in this case most of the buildings were founded on undisturbed gravel it was decided that the cross-hole result was applicable for the small strain range.

Based on a broad experience with static tests it is known that for the medium to higher range of strain laboratory test data is more representative, i.e. it would be unrealistic to raise the

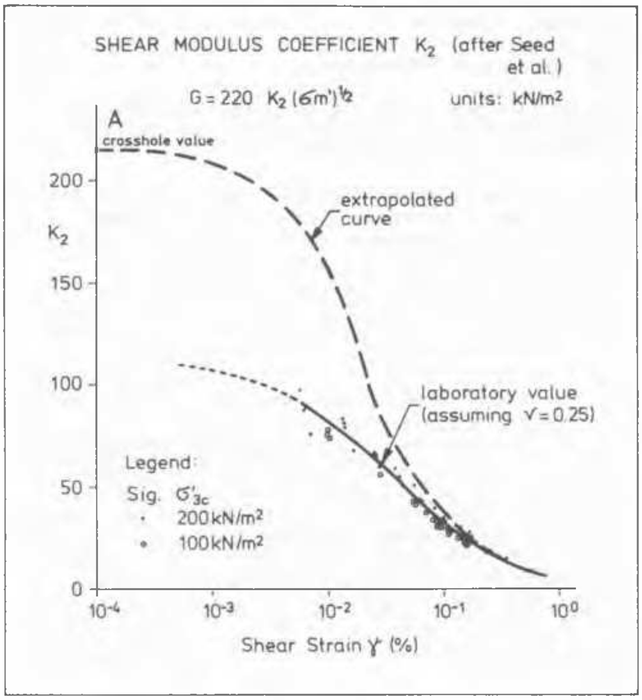

Fig. 1 Proposed curve for shear modulus coefficient K_2 versus shear strain based on field and laboratory data

values corresponding to the crosshole value by shifting the whole curve. Thus there are two reliable data zones and interpolation is made between them as shown in the figure. The broken line shows the recommended soil property curve for the site amplification study.

There is clearly a need for further research in this area, by improving testing procedures, relating field and laboratory conditions more adequately, and developing equipment to apply larger strains in-situ.

V.D.L. Finn, Co-Chairman

INTRODUCTION TO ORAL DISCUSSION

It is my pleasure to chair the floor discussions for this session. Active coherent floor discussion is one of the most vital elements in fostering vitality and interest among participants in a conference such as this. Therefore, we have attempted to accommodate as many requests for discussion as possible. We regret that not all requests for discussion time could be met in the time available.

The number of selected discussers was limited by our concern that each have adequate time to make his point and that time would also be available for occasional comments by the panel or interjections from the floor. We attempted also to ensure that the selected discussions would provide broad coverage of all topics covered by this session. I invite those who cannot have an opportunity to present oral discussions today to submit written contributions for inclusion in the final volume of the proceedings.

Successful execution of our plans for this session depends on the cooperation of the discussers. I appeal to you to abide by your time limits so that we may all enjoy a relaxed but stimulating afternoon.

P.J. Moore, Panelist

DESIGN OF MACHINE FOUNDATIONS

I would like to raise three points relating to matters discussed in several of the papers presented in this session.

The first point concerns the elastic half space solution for determination of the resonant frequency and vibration amplitude of a rigid footing. It should be remembered that many workers have produced elastic half space solutions for the various modes of vibration. These solutions are not always in close agreement. For the determination of resonant frequency (frequency at maximum amplitude) under constant force excitation, several of these half space solutions are compared in Fig. 1 for the case of vertical vibration. Calculations based upon the mass-spring analogy (Barkan (1962), Lysmer and Richart (1966)) have also been included in the figure. It is seen that the dimensionless resonant frequencies (a_r) are in approximate agreement for large values of the mass ratio (b) but wide disagreements are evident for low values of the mass ratio. For the mass-spring analogy to yield similar results to the half space approach it is clear that damping should be taken into consideration.

In practice, many machines involve rotating mass excitation and not constant force excitation. For rotating mass excitation Fig. 2 compares the calculated resonant frequencies according to the Sung and Robertson Solutions and the damped mass-spring analogy. It is seen that the disagreements at low values of the mass ratio are greater than in the case for constant force excitation. For determination of the maximum displacement amplitude, Fig. 3 shows that wide disagreements between various solutions also occur at low values of mass ratio. In view of these disagreements, the particular solution referred to should be identified in any discussion of half space solutions.

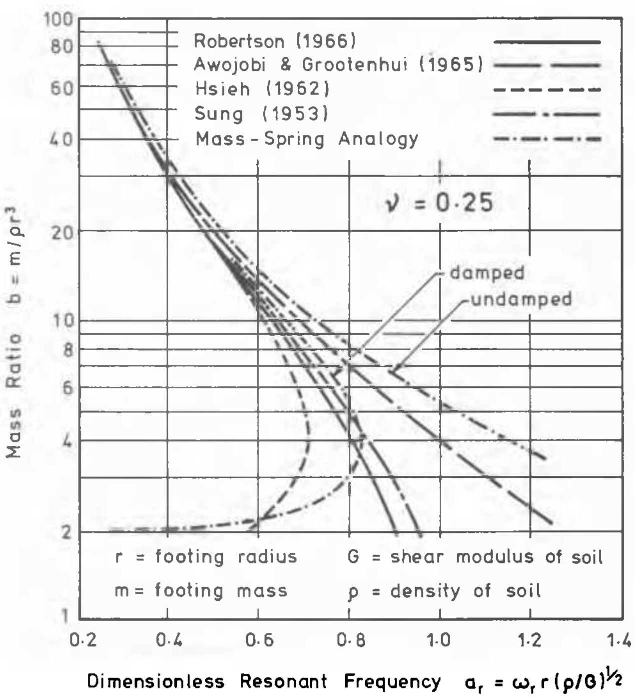

Fig.1 Resonant Frequency for Vertical Vibration - Constant Force Excitation

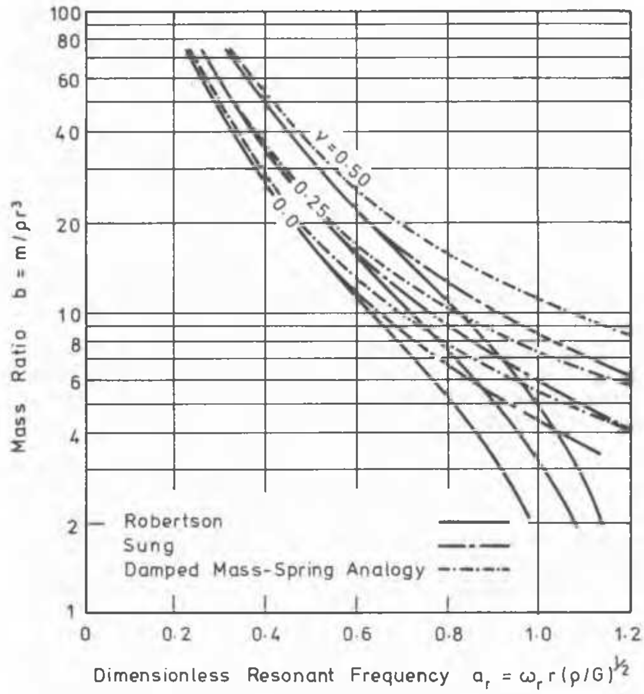

Fig.2 Resonant Frequency for Vertical Vibration - Rotating Mass Excitation

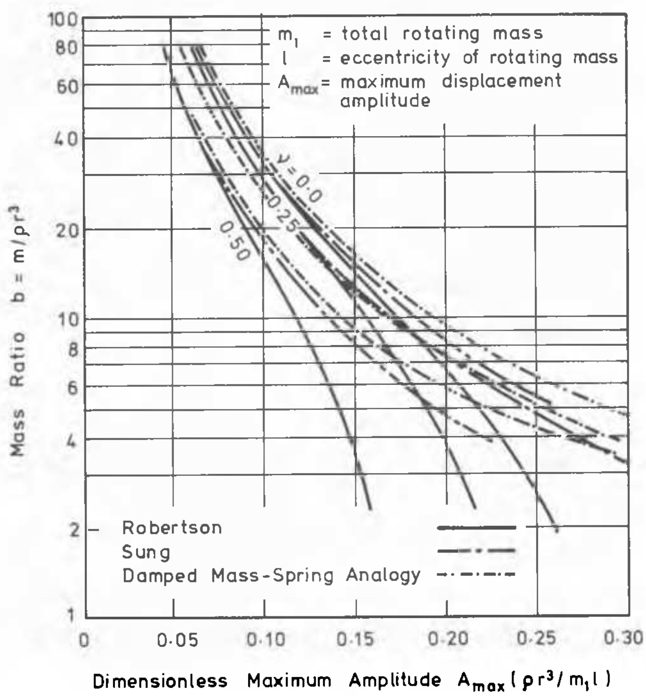

Fig.3 Maximum Amplitude for Vertical Vibration - Rotating Mass Excitation

The second point I would like to raise concerns the interrelationships between the various elastic coefficients in Barkan's (1962) approach to the calculation of vibration frequencies and amplitudes. The coefficients are:

(a) coefficient of elastic uniform compression, c_u,

(b) coefficient of elastic non-uniform compression, c_ϕ,

(c) coefficient of elastic uniform shear, c_τ, and

(d) coefficient of elastic non-uniform shear, c_ψ.

As shown in Table I, Barkan (1962) and Richart et al (1970) have recommended relationships between these coefficients, that could be used for preliminary design purposes. To facilitate better agreement between calculated resonant frequencies according to the Barkan and elastic half space approaches, the last column in Table I provides a slightly different set of ratios between these elastic coefficients.

The third point relates to the use of an apparent mass in the lumped parameter approach to the calculation of resonant frequency. The apparent mass, also referred to as the added soil mass or the in-phase mass, is a mass of soil beneath the footing and vibrating in-phase with it. Prior to 1977 there was some agreement that the apparent mass could be neglected in design calculations (Barkan (1962), Whitman and Richart (1967), Richart et al 1970)). In 1977 Barkan and Ilyichev presented an interpretation of several field observations, which indicated that the apparent mass cannot be neglected in calculations. The reason for the need to use apparent mass in calculations lies in the significant overestimate of the resonant frequency by using a damped or undamped lumped parameter model, particularly at low values of the mass ratio. The lines in Fig. 4 indicate the values of apparent mass that must be used to yield the same resonant frequency that would be calculated by means of the Robertson solution. The field observations quoted by Barkan and Ilyichev have been superimposed on the figure. While there is a large scatter of results, the majority of the points lie in the region of the theoretical lines. This suggests that the Robertson solution provides a reasonable estimate of the resonant frequency and the use of apparent mass provides, in effect, a correction factor to the lumped parameter calculation.

TABLE I

Recommended Relationships Between Elastic Coefficients for Square or Circular Bases

	Barkan (1962)	Richart et al (1970)	Moore
c_ϕ/c_u	1.73	2	2.5
c_τ/c_u	0.5	0.5	0.75
c_ψ/c_u	0.75	1.5	1.8

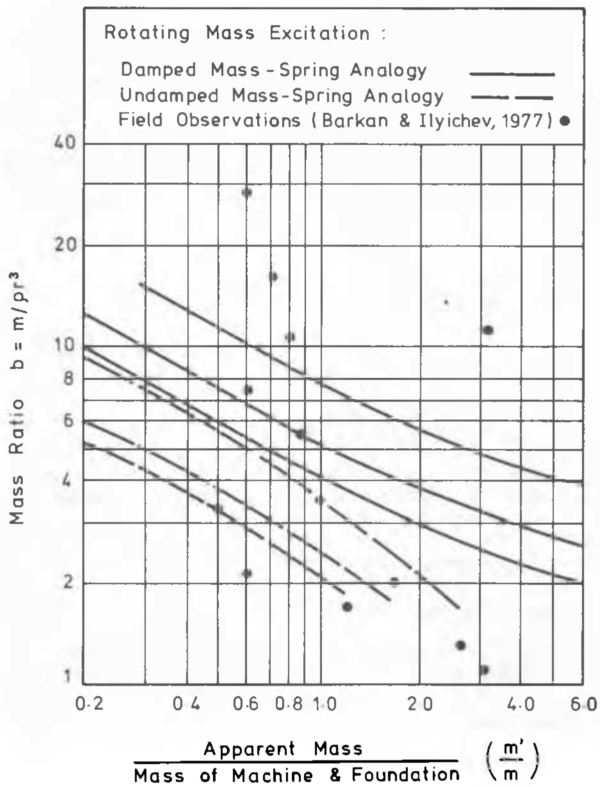

Fig.4 Apparent Mass to be used with Mass-Spring Analogy for Calculation of Resonant Frequency

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S. Prakash, Panelist, V.K. Puri and W.D. Horst

SOME ASPECTS OF MACHINE FOUNDATION DESIGN

SYNOPSIS Two important factors in predicting the response of foundations for machines are input soil parameters and the mathematical model adopted for analysis. Either the weightless linear spring or elastic half space model is usually adopted for design. This report compares the dynamic response characteristics of bases of three different sizes as predicted by using linear weightless spring theory and elastic half space model vis-a-vis the observed response using soil stiffness parameter determined by in-situ steady state vibration test. The study was restricted to the case of vertical vibrations only.

INTRODUCTION

Two approaches generally followed for design of foundations for machines are:

1. Linear weightless spring theory (Barkan, 1962).
2. Elastic half space approach (Richar, Hall, and Woods, 1970).

The key factor in both the approaches is the evaluation of input soil parameters. Soil stiffness parameters are affected by a number of different factors (Richart, Hall and Woods, 1970; Prakash, 1981) the most important of which are confining pressure and strain amplitude. Method for determination of soil stiffness parameters after accounting for the effects of confining pressure and strain amplitude has been discussed elsewhere (Prakash and Puri, 1977, 1981; Prakash, 1981).

Comparison of vibration test data with computed response of footing has also been reported by some investigators (Fry, 1963; Richart and Whitman, 1967; Prakash and Puri, 1980, 1981).

In this panel report the steady state vibration test data of Fry (1963) for the Eglin site for four bases of different sizes has been analyzed using the linear weightless spring approach and elastic half space approach for the case of vertical vibrations. Test data obtained from base no. 1, Table 1, was used for obtaining the dynamic shear modulus G and the response of bases no. 2, 3 and 4 for the cases of equal weight and equal static pressure has been worked out. A comparison of the predicted response using the two design approaches with the observed response has been made.

SOIL DESCRIPTION

Soil at the Eglin test site is a nonplastic uniform fine sand (SP) (Richart and Whitman, 1967).

DATA ON BASES

Data pertaining to test bases at Eglin site is summarized in Table 1.

TEST DESCRIPTION

Steady state vibration tests were conducted by Fry (1963) on the bases described above using a rotating mass type mechanical oscillator and data on amplitudes at different frequencies of excitation was monitored. Four different values of eccentricity 0.105", 0.209", 0.314" and 0.418" had been used.

TABLE I. Data on Test Bases at Eglin site (Fry, 1963) Vertical Vibration Tests.

Base No.	Diameter Inches	Weight Pounds	Area Sq Ft	Contact Pressure psf	Remarks
1	62	12820	20.97	611	Reference Base
2	87-5/8	25640	41.88	608	Equal Static Pressure below the Base
3	108	38460	63.62	605	
4	124	51280	83.86	611	
2′	87-5/8	30970	41.88	740	Equal Weight of Bases
3′	108	30970	63.62	487	
4′	124	30970	83.86	369	

EVALUATION OF DYNAMIC ELASTIC CONSTANTS OF SOIL

Linear Spring Theory

(1) Coefficient of Elastic Uniform Compression C_u

The value of C_u was determined from the observed natural frequency of base no. 1 at the lowest eccentricity setting (Table I) for use in analysis of data of bases no. 2, 3 and 4. Equation 1 shows the relation between C_u and the observed natural frequency f_{nz}

$$f_{nz} = \frac{1}{2\pi}\sqrt{\frac{C_u A}{m}} \qquad (1)$$

where A = area of contact of the base and m = mass of the base.

(2) Corrections to Value of C_u

The values of C_u obtained were then corrected for effective confinement (Prakash and Puri, 1977) and area (Barkan, 1962).

Elastic Half Space Approach

Dynamic Shear Modulus G

The determination of dynamic shear modulus involved the following steps:

(1) Mass ratio $= B_z = \frac{(1-\nu)}{4}\,\frac{m}{\rho r_0^3}$ (2)

where ν = Poisson's ratio,
m = mass of the base,
ρ = mass density, and

r_o = radius of the base contact area.

(2) Magnification Factor M_r

$$M_r = \frac{MA_z}{m_e e} \qquad (3)$$

where A_z = observed amplitude of vibration,
m_e = rotating eccentric mass, and
e = eccentricity

(3) Using the values of M_r and B_z the dimensionless frequency factor a_o was determined from Fig. 1.

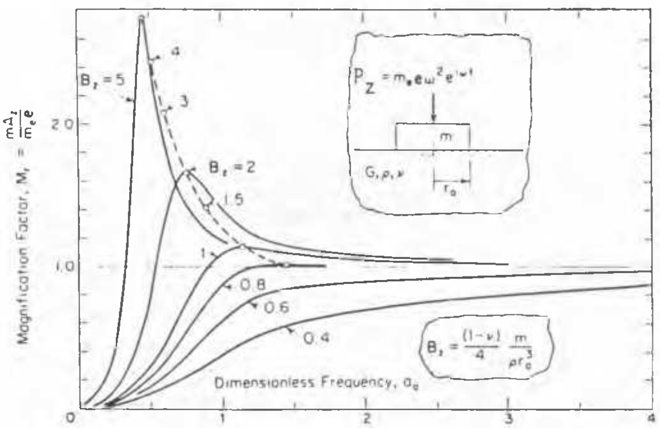

Fig. 1. Response of rigid circular footing to vertical force developed by rotating mass exciter $P_z = m_e e\omega^2$ (Richart et al. 1970)

(4) Using Equation 5, the value of dynamic shear modulus G is computed

$$a_o = \omega_n r_o \sqrt{\rho/g} \qquad (4)$$

or

$$G = \frac{\omega_n^2 r_0^2}{a_0^2} \rho \qquad (5)$$

(5) The value of G so obtained may be corrected for effective confinment (Prakash and Puri, 1981)

The value of C_u and G need correction for shear strain levels. The magnitude of shear strain associated with the test data from which the value of G has been determined is of the order of 1.6×10^{-5} (defined as ratio of amplitude of vibration to width or diameter of the footing). The shear strain levels associated with vibration of bases 2, 3 and 4 range from 1.499×10^{-5} to 8.04×10^{-5}. In this low range fo shear strains, the value of G will be little effected by shear strains. Hence no correction for shear strains was made in this case.

COMPUTATION OF DYNAMIC RESPONSE CHARACTERISTICS

Linear Weightless Spring Theory

Using the values of C_u corresponding to conditions of mean effective confining pressure and the area of the base, the value of natural frequency and undamped amplitudes of vertical vibration were computed following the procedure given by Barkan (1962).

Elastic Half Space Approach (Equivalent lumped parameter model (Lysmer and Richart, 1966).

Using the corrected values of G, the natural frequency and amplitudes of vibration at any desired frequency have been computed by the procedure after Richart, Hall and Woods (1970).

The values of natural frequencies for bases 2, 3 and 4 using the linear weightless spring approach and the elastic space model have been shown in columns (2) and (3) in Table II. The corresponding observed values of natural frequency (Fry, 1963) are shown in column (4) of Table II.

TABLE II. Comparison of Observed and Predicted Natural Frequencies

	Predicted Natural Frequency H_z		Observed Natural Frequency H_z	
Base No.	Linear Spring	Elastic Half Space		Remarks
1	2	3	4	5
2	14.4	12.9	16.0	Equal
3	13.4	12.0	16.0	Static
4	12.7	11.4	16.5	Pressure
2′	13.4	12.1	16.0	Equal
3′	14.6	13.1	16.0	Weight
4′	15.6	14.6	16.5	

Typical data on the ratio of computed amplitude/observed amplitude vs frequency for the bases analyzed has been plotted in Figures 2 through 5

DISCUSSION AND CONCLUSIONS

(1) The natural frequency of vertical vibrations computed using linear weightless spring model (Table II, Col. 2) and elastic half space model (Table II, Col. 3) are comparable with each other. The values of natural frequencies computed using linear weightless spring model show a somewhat better agreement with observed natural frequencies.

(2) Amplitudes computed by using linear weightless spring approach are consistently higher than the observed amplitudes. This may be explained since damping has not been taken into consideration.

(3) Computed amplitudes using elastic half space

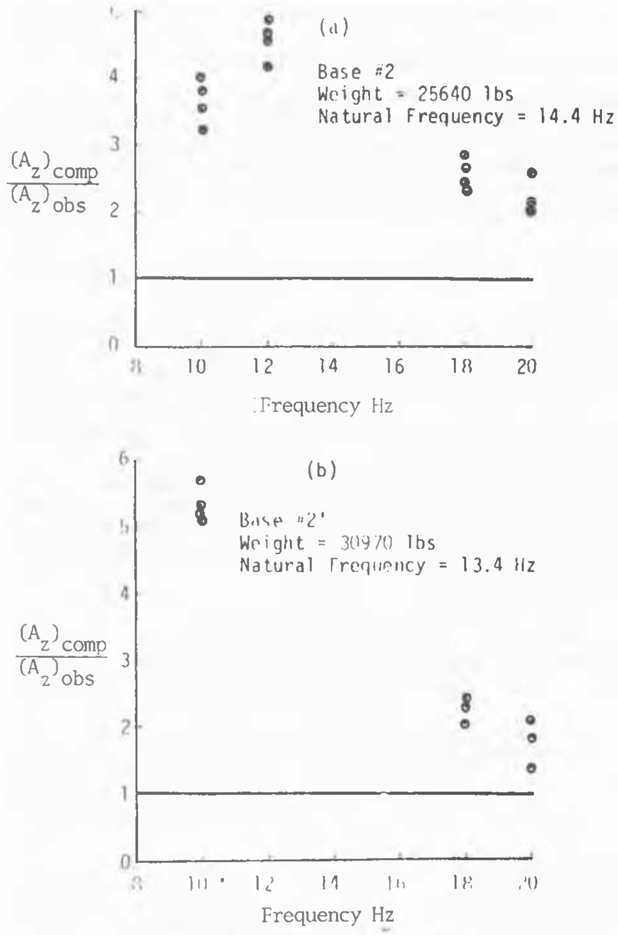

Fig. 2. $\frac{\text{Computed Amplitude}}{\text{Observed Amplitude}}$ vs Frequency (Linear Weightless Spring Approach)

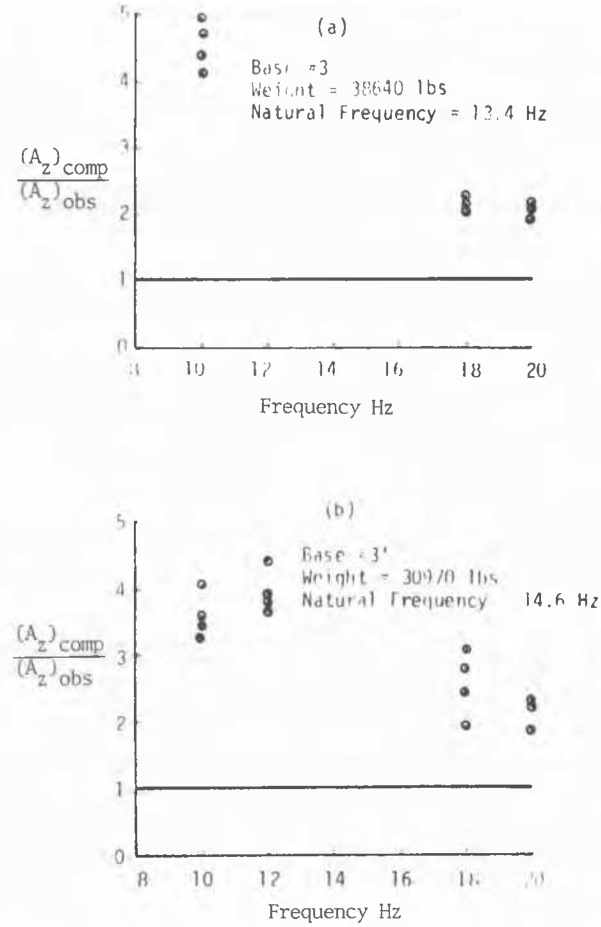

Fig. 3. $\frac{\text{Computed Amplitude}}{\text{Observed Amplitude}}$ vs Frequency (Linear Weightless Spring Approach)

approach are generally larger than the observed amplitudes but are considerably larger than the observed values for frequencies smaller than the computed natural frequency of the system. It is difficult to comment on the amplitudes at low frequencies because at very low frequencies there may possibly be some errors in the measured values.

Amplitudes larger than the measured values are to be expected since in the analysis only geometrical damping is being accounted for.

(4) There seems in general a good agreement between observed and measured amplitudes for frequencies beyond the computed natural frequency of the system according to elastic half space. However in a few cases the computed amplitudes are smaller than the measured amplitudes and are within 80% of the measured values. Using simple models it seems difficult to explain the phenomenon of computed amplitudes being smaller than the observed amplitudes. In the data reported by Richart and Whitman (1967), it is observed that the computed amplitudes for vertical vibrations could differ by a factor of 2 in comparison with the observed amplitudes using the values of soil constants corrected for confining pressure, it has been observed that the computed amplitudes show a better agreement with observed amplitudes.

(5) The computation here is limited to the case of vertical vibrations only. There is necessity to study the performance of other bases subjected to combined rocking and sliding and torsional modes of vibration. The reported data of Fry (1973) and Prakash and Puri (1981) is being analyzed for these cases as well and the information will be disseminated as soon as possible.

(6) Presently the data does not permit any general conclusion about the performance of one approach relative to the other.

(7) Performance data on prototype machine foundations need to be procured to decide about the approach yielding best results. Such a data will be useful only if systematic soil data is also procured simultaneously since the soil parameters are the key factors in influencing the predicted response of soil foundation systems.

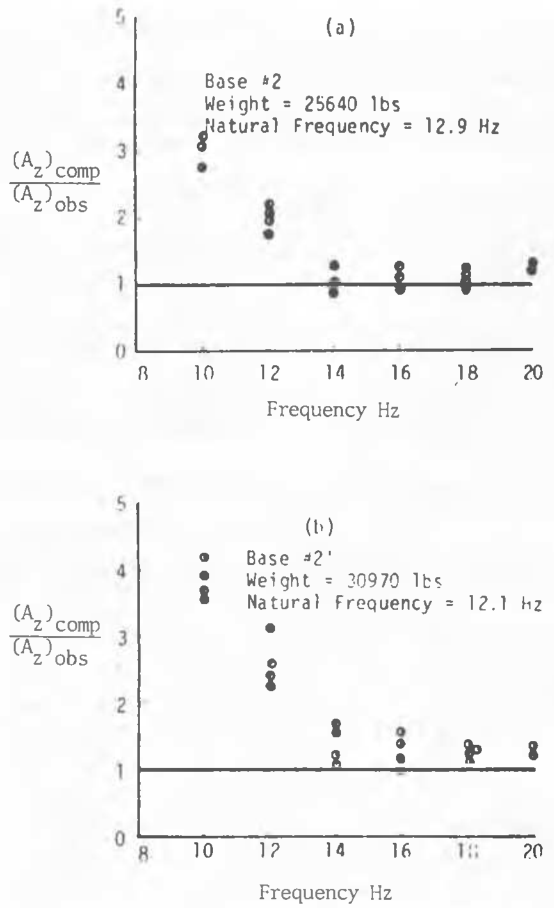

Fig. 4. $\frac{\text{Computed Amplitude}}{\text{Observed Amplitude}}$ vs. Frequency

(Elastic Half Space Approach)

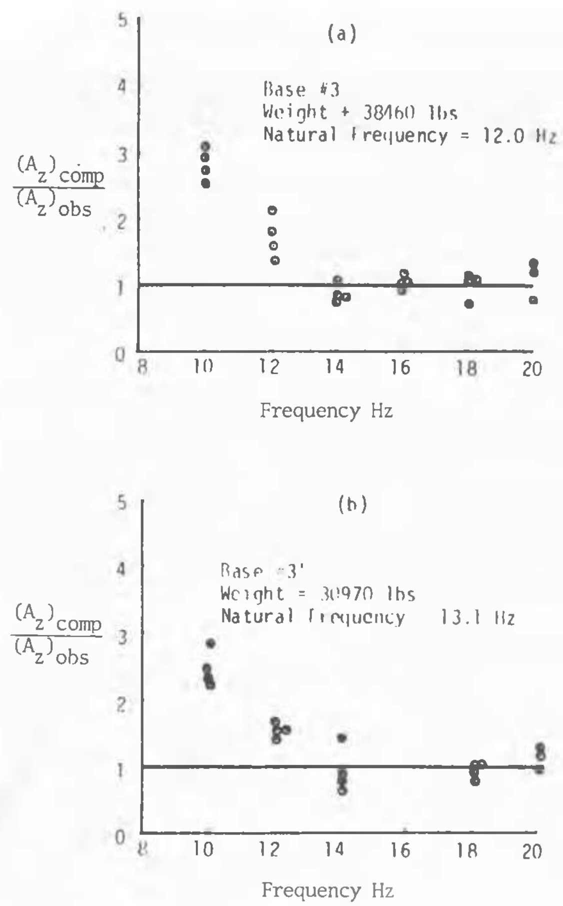

Fig. 5. $\frac{\text{Computed Amplitude}}{\text{Observed Amplitude}}$ vs. Frequency

(Elastic Half Space Approach)

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V. Perlea, Panelist

My contribution to this panel discussion concerns the failure of saturated cohesionless soils induced by earthquakes and some methods to prevent it.

As Prof. H. Bolton Seed emphasized in 1977 during the Tokyo Conference, the period 1964 to 1971 was rich in dramatic failures of earth structures during earthquakes, most of them due to soil cyclic liquefaction. After a relatively more quiet period, many detrimental events produced in the last six years, at least during the listed strong earthquakes:

Tangshan	China	July 28,	1976
Vrancea	Romania	March 4,	1977
Izu Ohshima Kinkai	Japan	Jan 14,	1978
Miyagi Ken Oki	Japan	June 12,	1978
Thessaloniki	Greece	June 20,	1978
Montenegro	Yugoslavia	April 14,	1979
Enmedio Island	Mexico	March 14,	1980

The papers submitted to this Conference do not present case histories, but the literature published since the 1977 Tokyo Conference is very rich from this point of view, beginning with:

ASCE Annual Convention, Philadelphia, Pa., U.S.A., September 1976

International Symposium on Dynamical Methods of Soils and Rocks, Karlsruhe, Germany, September 1977

Sixth European Conference on Earthquake Engineering, Dubrovnik, Yugoslavia, September 1978

International Conference "Protection of Structures in Seismic Areas", Bucharest, Romania, November 1978

and continuing with the Conferences mentioned in the General Report.

My discussion, although takes into account many of these published studies, uses as illustrative material only events occurred during the

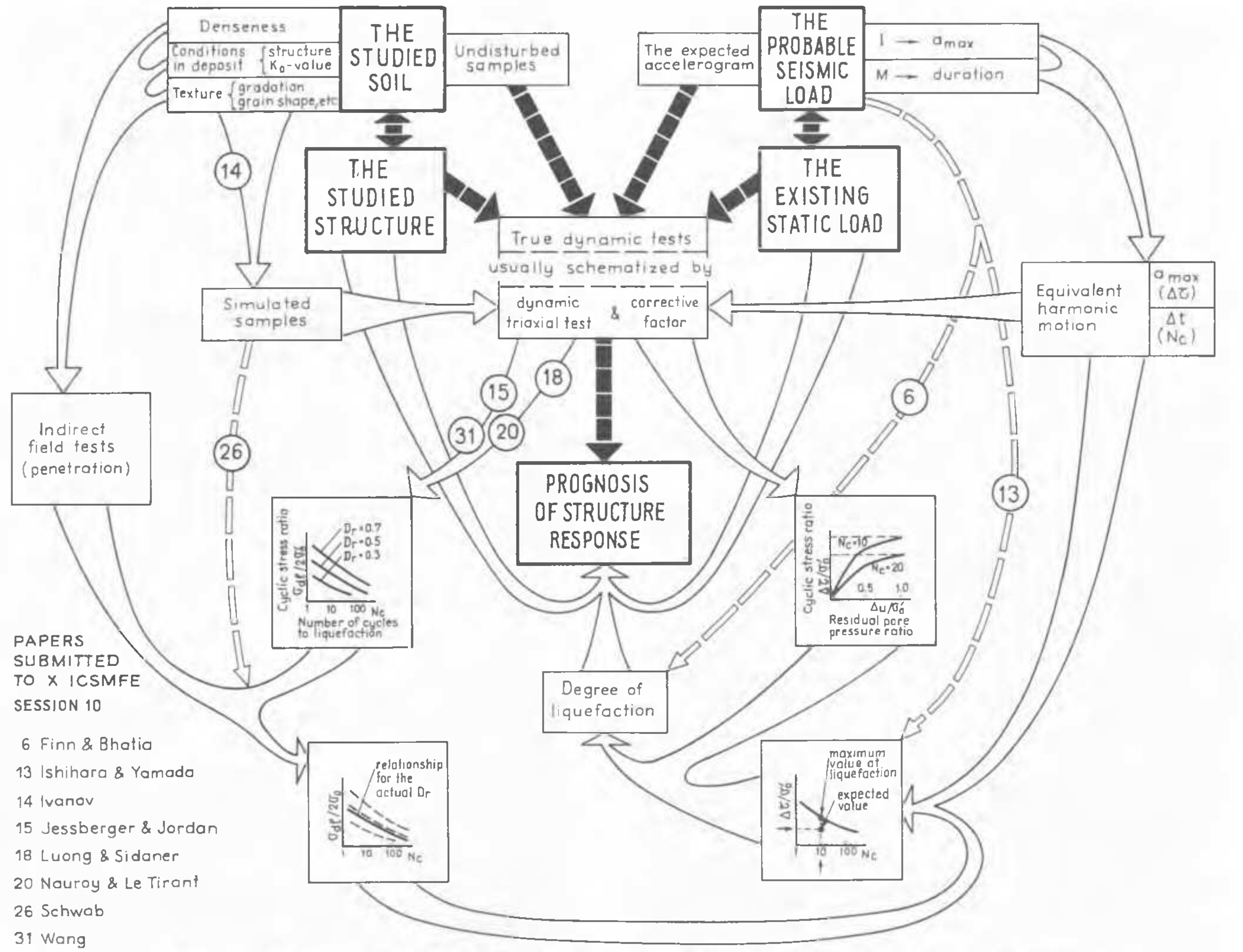

Figure 1 Procedure for Predicting the Liquefaction Effect on Structures

Romanian Vrancea earthquake. I try also to summarize a less discussed item in the cited literature, that is the efficient methods to prevent and control the soil failure and its effects.

A rough classification of damages following the cohesionless soil failure may comprise four categories:

1. Liquefaction of foundation soil manifested mainly by vertical deformations of ground surface. Depending on the initial stress condition the liquefaction may result in lateral - spreading or flow landslides. Usually, this produces the ejection of sand and water from the ground through old or new cracks, forming small sand volcanoes. The ejected sand is not necessarily the liquefied one under the earthquake motion, as usually considered, but would be liquefied by the transient upward flow of water (Perlea, 1978).

The prognosis of this form of liquefaction and its effect on structures may be done by a more or less simplified procedure (Fig.1).

The direct way (the dashed solid arrows in Fig. 1), using the actual material and the expected loading is seldom adopted, implying the most important technical difficulties. Usually devious ways are chosen to reach the desired result; some of the papers presented in this Conference represent valuable contributions towards the development or by-passing of some branches of the liquefaction study scheme.

The lateral spreading, specific to zones with K_o - values differing from place to place or with local additional stresses, is the main cause of the levees degradation. During the Vrancea earthquake scores of kilometers of flood banks, along the Danube and other rivers, were affected by longitudinal cracks on crest or benches, accompanied however by unsignificantly settlements (Botea et al, 1980).

A stabilizing method based on increasing of confining stresses in the liquefiable layer under the toe of slopes was considered in the design of levees for a reservoir in a seismic area, under the form of stabilizing benches, as compared to other more used methods based on sand densifying.

Pseudo-static stability analyses by the Swedish limit equilibrium method were performed, taking into account the change in pore pressure distribution following the earthquake action. As for instance, in Fig.2 the distributions for the reference cross section (designed for static loads) are shown. The four studied stabilizing methods had in view on one hand the establishing of the necessary width of the compacted zone, and on the other hand, the effectiveness of benches as compared with compaction methods (Fig.3). Among the conclusions:

(i) the extending of the compacted strip beyond the embankment reservation or to the interior on more than a rather narrow strip near the slope toe, does not lead to a significant increase of the safety factor;

(ii) the allowable safety factor for the upstream slope is ensured by a flattening of the slope as for as 2.5 : 1 only; when the height of the embankment reaches 15 meters an additional compaction under the toe would be necessary;

(iii) for the downstream slope, the same stabilizing effect is obtained by: the flattening of the slope as for as 5.6 : 1; the densifying by compaction on a 12 meters width strip under a 3 : 1 slope; and a 12 meters width bench. Based on an economic study the bench was preferred, although the safety factor corresponding to it is less than one, being probable the bench failure and its restoration needed after an earthquake.

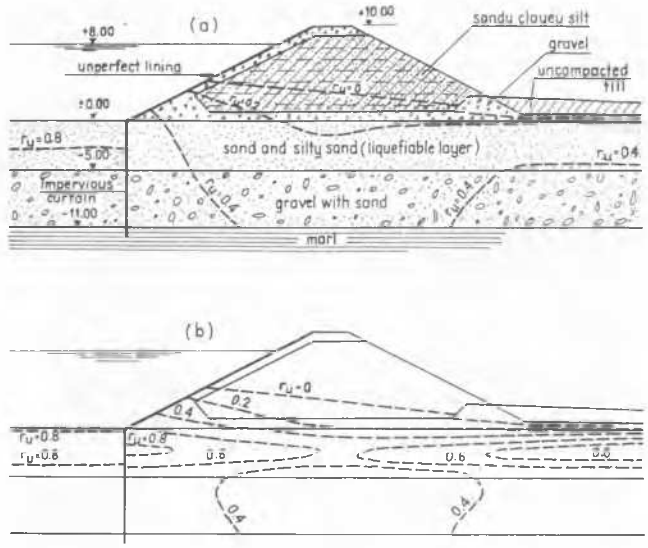

Fig.2. The Distribution of Pore Pressure Ratio, r_u: (a) Before the Earthquake; (b) During the Liquefaction of Foundation Soil.

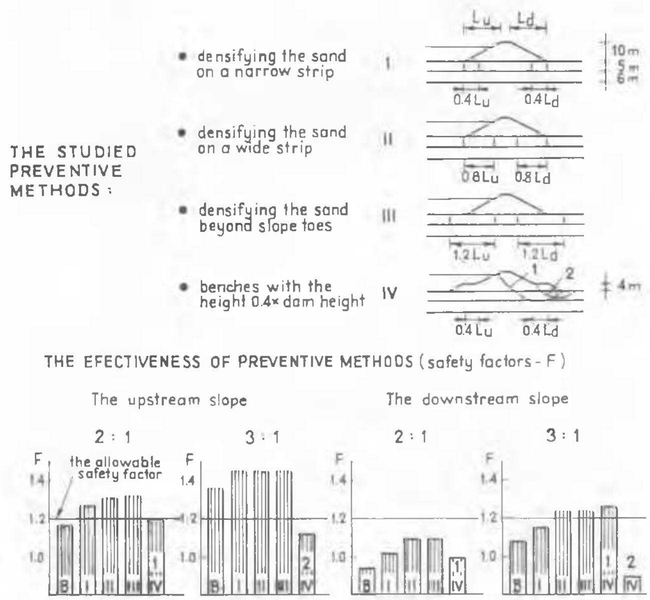

Fig.3. The Comparative Study of Stabilizing Methods

2. Another kind of damages are landslides of cohesive mass underlain by a layer or lens constituted by uncohesive liquefiable soil. A landslide of this type occurred during the Vrancea earthquake, affecting some houses in the Danube low plain (Zaharescu et al, 1977).

This kind of damage, although easy to explain after it is produced, is extremely difficult to predict, and, consequentially, to prevent.

3. Liquefaction of cohesionless soil used as fill material in earth structures, for example embankment dams, especially when hydraulically placed or spoil heaps resulted in mining industry. During the Vrancea earthquake a small scale failure of the inner slope of a lagoon dike in gravel took place (Fig.4).

The accidents are very likely when the resistant part of the structure is founded on liquefiable material, as in the upstream method used in spoil heap construction. The results presented in Fig.4 are relevant, showing that 50% liquefaction produces a decrease in the shear resistance along the potential slip surface sufficient to generate the failure.

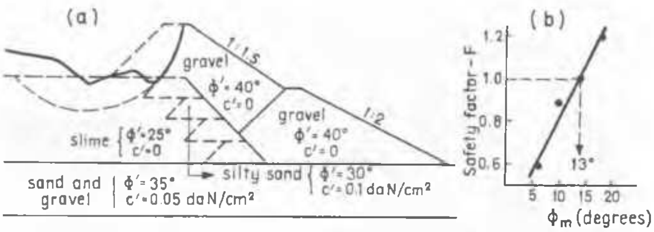

Fig.4. Failure of Lagoon Dike: (a) Representative Cross Section; (b) The Mobilized Internal Friction in Slime, Φ_m, for Limit Equilibrium

4. Sinkholes in sandy deposits following the flow down of alluvial material in unsealed faults, cracks or caverns in the bedrock. Such a sinkhole with a diameter of 40 meters (Fig.5) developed during the Vrancea earthquake in the Danube flood plain (Stoica, 1981).

Fig.5. The Sinkhole, Some Hours after it Occurred.

In the affected area, the most of the alluvial quaternary strata, 15 - 19 meters thick, is constituted of medium dense sand; it is followed by cretaceous cracked limestone, down to some hundred meters.

The phreatic level, controlled by the Danube river, is not as a rule directly connected with the ground water level in the limestone system of cracks which depends mainly on the Bulgarian rivers supply (Fig.6, left). The rise of the pore water pressure in sand during the earthquake action, and the oscillation of the pressure head in crack system might cause the clea-

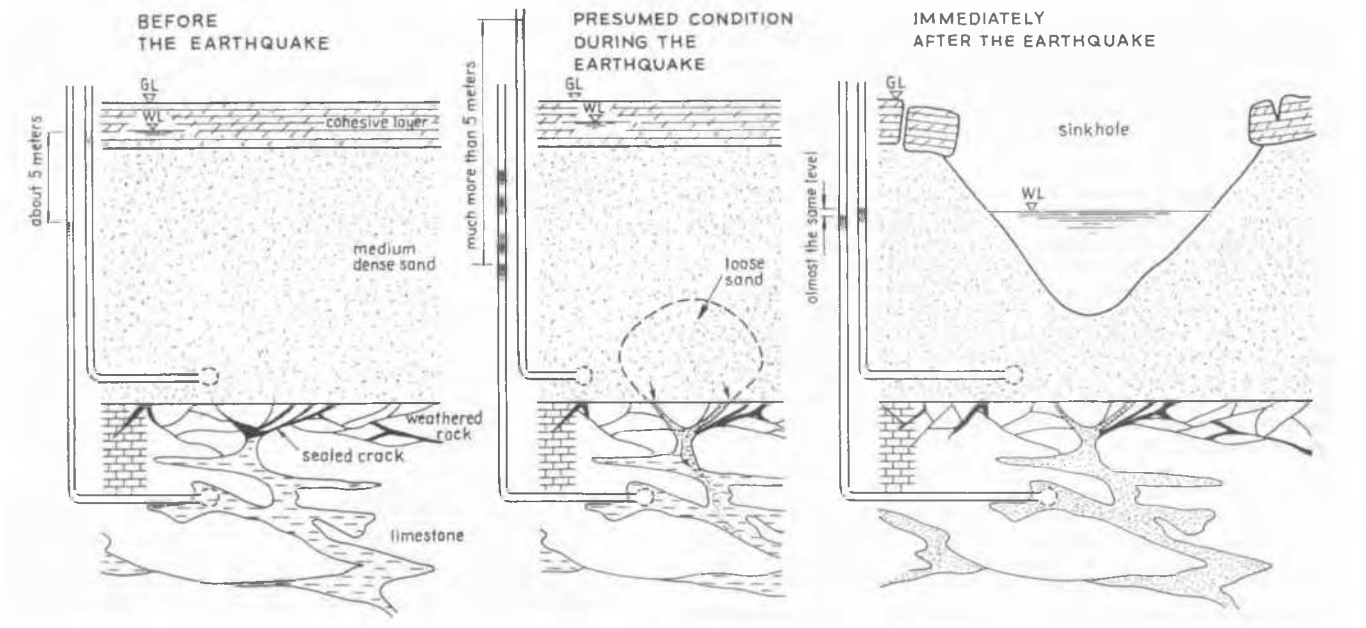

Fig.6. The Probable Phenomenon of Clearing the Sealed Cracks during an Earthquake, Followed by Sinkhole Occurring

ring of the communication ways between the two water layers, accompanied by the flowing down of the sand into cracks. Actually, the two levels were found almost the same immediately after the sinkhole occured (Fig.6, right).

The phenomenon was not isolated. A smaller sinkhole developed during the earthquake in a neighbouring area, in the same geological conditions. Another two sinkholes were discovered in a desert island on Danube only after the fallen trees got dry. The proximity of a sinkhole to only 15 meters from the river bank pleeds for the explanation of the phenomenon by a mainly vertical movement of the soil. Moreover, the drilling of a surveying borehole near the main sinkhole, although the hole was lined throughout the cohesionless layer, induced the flowing down of the sand into holes in limestone, when the boring riched some 60 meters in depth, that is some 40 meters in limestone (Fig.7).

Fig.7. The Sinkhole Induced by the Surveying Borehole.

The loosening of the sand deposit in the vicinity of the cleared cracks might not have been immediately followed by sinkhole occurring, and may constitute a failure source in the future. This statement is supported by the finding of unexpected decrease of the penetration resistance near the bottom of the sand layer in some locations(Fig.8).

In the close vicinity of the zone with subsidence phenomena, and in the same geological conditions, important industrial structures have been built. It is imperative to ensure the structures against the possibility of sinkhole development. For that purpose a sealing of the contact zone between alluvium and limestone under the structures on deep foundations (precast concrete driven piles) is advocated.

A protective zone outside the structure contour, horizontally or vertically developed, aims at exclusion of the damage risck when a sinkhole develops near the structure (Fig.9).

The sealing by grouting requires three stages: the grouting with ash - cement mixture has in view the clogging of the cracks in limestone; for the case that grouting of cracks in limestone is not resistant to increased gradient

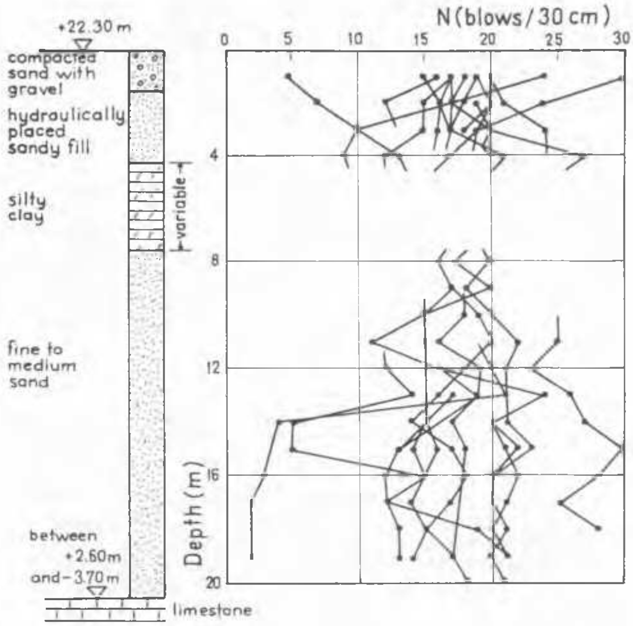

Fig.8. Soil Profile in the Affected Area and the Results of Some Standard Penetration Tests.

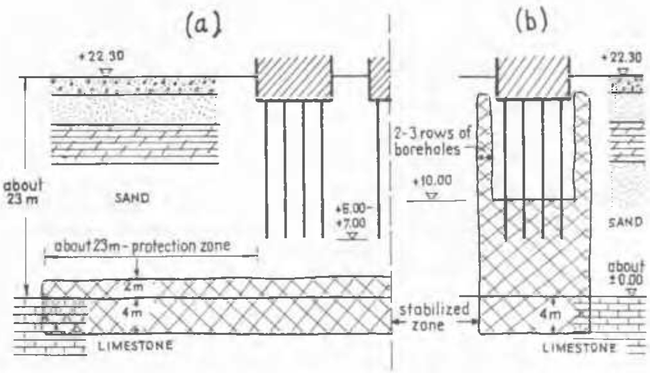

Fig.9. The Shape of Protective Zone: (a) First Case - Large Structures or Many Important Structures Side by Side; (b) Second Case - Izolated Structures of Small Extent.

during earthquakes, the internal erosion of alluvium is prevented by grouting sodium silicate with organic reagent (ethyl acetate); an intermediate stage, comprising the cement-bentonite grouting of the contact zone is necessary to prevent an exaggerated consumption of the expensive grouting material used in the last stage (Fig.10).

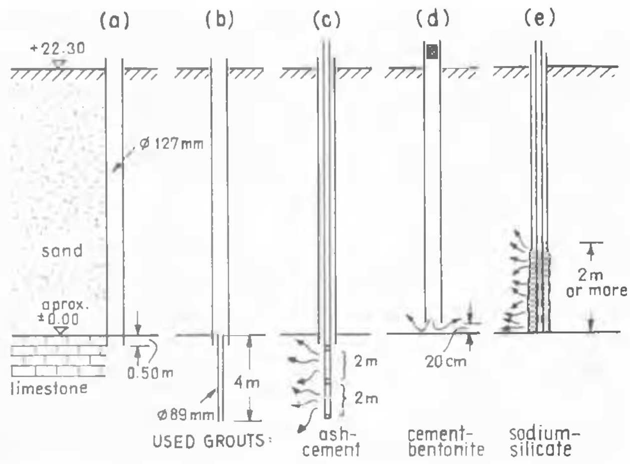

Fig.10. The Phases of Stabilization Work: (a) Drilling and Lining; (b) Drilling without Lining; (c) Grouting in Two Steps with Packers; (d) Grouting with Presse - Etoupe; (e) Grouting with Tube à Manchette.

Y. Yoshimi (Oral discussion)

BEHAVIOUR OF SATURATED SAND SUPPORTING HEAVY STRUCTURES

This discussion concerning the behavior of saturated sand supporting a heavy structure is addressed to the last page of the General Report for Session 10 in relation to model tests in normal gravity. Interaction between heavy structures and liquefied soil is demonstrated in Fig. 1 in which S is the average settlement of a reinforced concrete building during the Niigata earthquake of 1964, B is the width of the building, and D is the estimated depth of liquefied soil in the free-field (Yoshimi and Tokimatsu, 1977). The figure clearly shows the trend that the settlement ratio decreases as the width ratio increases. The fact that the relationship is independent of the number of stories shows that this is not a simple bearing capacity problem.

In order to get some insight into the problem, shaking table tests on a model structure placed on saturated sand were carried out. To minimize restraint by the end walls, the length-to-depth ratio of the model ground was made fairly large (L=6.5D), and foam rubber was placed as shown in Fig. 2. The bottom of the container was made rough to prevent slippage, and glycerin was added to the pore water to simulate seepage conditions in the field. The model was made two-dimensional so that the test results could be directly compared with the results of numerical analyses. The relationship among the natural frequency of the structure, the natural frequency of the ground, and the vibration frequency was selected in such a way that the model ground would experience cyclic shear roughly proportional to the depth. The effect of low confining stress was expected to be minimized by using medium dense sand. Fig. 3 shows typical test results. The excess pore water pressure ratio directly below the structure was considerably lower than that away from the structure. This

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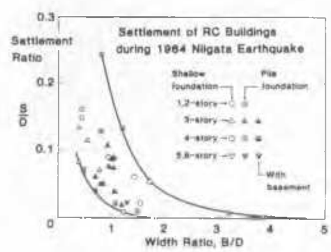

Fig.1

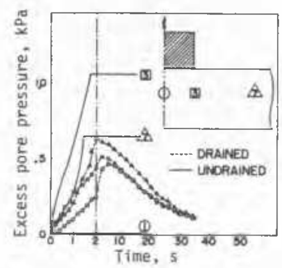

Fig.4

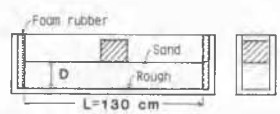

Fig.2

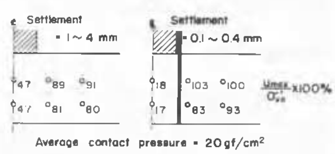

Fig.5

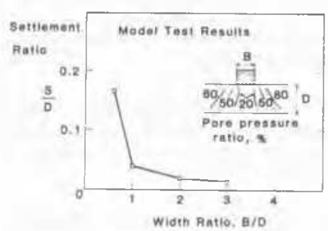

Fig.3

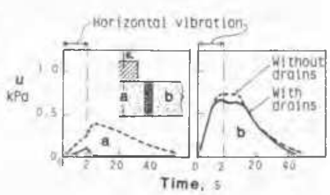

Fig.6

explains why the settlement is reduced as the width is increased.

Our analysis is based on the Biot consolidation equation with a pore pressure generation term added (Yoshimi and Tokimatsu, 1978). In order to make the analytical results comparable with the test results, the soil properties used in the analysis were evaluated under the prevailing low stress conditions in the model ground. Fig. 4 shows computed time histories of excess pore pressure at three points in the sand, the dotted curves for the drained condition and the solid curves for a fictitious, undrained condition in which the coefficient of permeability was assumed zero. The flat top for the undrained curves at Points 3 and 7 indicate that the sand at these points had liquefied. Note the very low pore pressures at Point 1 below the center of the structure for the undrained case. The differences in pore water pressures at the three points diminish considerably as a result of allowing seepage. It is emphasized that as far as Point 1 is concerned, the assumption of an undrained condition causes large errors on the unsafe side. Therefore, one must take into account seepage effects when one deals with pore water pressures below a heavy structure.

Fig. 4 seems to suggest the possibility that the stability of saturated sand could be improved by cutting off the influence of high pore water pressures, either by impermeable walls or by vertical drains. Our model tests showed that a pair of walls did reduce the excess pore pressures directly below the structure by about 50%, as shown in Fig. 5. The effect of the walls on the settlement was much more remarkable. Gravel drains installed outside the structure had similar effects on the excess pore pressures and settlement of the structure, as shown in Fig. 6 (Tokimatsu and Yoshimi, 1980).

In conclusion, it is pointed out that small-scale tests of the kind described above can provide a first approximation of what happened in the field, or what could happen in the field during future earthquakes. Such information can be useful in explaining the field behavior, and can help us devise remedial measures in a rational way.

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V. Perlea, Panelist

I would like to comment on the very interesting experimental results presented by Prof. Yoshimi. His contribution represents a convincing confirmation of previous findings of Ishihara and Matsumoto (1975) and of speaker himself (Yoshimi and Tokimatsu, 1977) about the stabilizing effect of a surcharge on a liquefiable layer.

The settlement of structures during liquefaction may be regarded as a bearing capacity problem, taking into account the decrease of the mobilized angle of internal friction due to pore pressure increase. After Okamoto (1973), the bearing capacity in regard to local shear failure of a cohesionless soil under a continuous footing is:

$$q_a = 0.5\,\gamma\,D_f(N'_q - 1) + 0.25\,\gamma\,B\,N'_\gamma + \gamma\,D_f$$

where:

- q_a : allowable bearing capacity of unit area during earthquake;
- γ : unit weight of soil (submerged below water table);
- D_f : depth of embedment of foundation;
- B : width of loading area;
- N'_q, N'_γ : bearing capacity factors depending on seismic coefficient, k_s, and reduced angle of internal friction, Φ' ;

$\Phi' = \Phi - \theta$;

Φ = angle of internal friction;

$\theta = 0$ when N - value (blows/feet in Standard Penetration Test) is greater than 20;

$\theta = \Phi$ when N - value is less than 5;

$\theta = \frac{20 - N}{15} \tan^{-1} k_s$ for intermediate N - values.

The increase of bearing capacity with increasing width of loading area is due not only to increase of the second term in the formula, but also to increase of bearing capacity factors as a consequence of a confining effect by the surcharge. This phenomenon is not evidenced by the formula.

The methods of failure prevention studied by Prof. Yoshimi, lateral walls or vertical drains, aim at bearing capacity factors increasing, by the increase of mobilized part of internal friction. An alternative stabilizing method would be also the increase of embedment, D_f, i.e. the increase of surcharge $q = \gamma D_f$, which lead to increase the first and the third terms in the bearing capacity formula. An earth fill 2.50 meters high on all the area between multistory reinforced concrete buildings as preventive method was recommended, in a zone in Bucharest where liquefaction phenomena occurred during the strong 1977 earthquake.

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R. Dahlberg (Oral discussion)

DESIGN OF FOUNDATIONS AGAINST FAILURE IN CYCLIC LOADING
Calcul des Foundations en Considération de "Failure in Cyclic Loading"

When doing a stability analysis for the foundation of an offshore gravity structure the traditional approach has been to consider a so-called quasi-static stability failure to be the critical failure mode. In this type of analysis the effects of cyclic loading is considered by introducing in the calculations the static undrained shear strength $c_{u,r}$ measured immediately after the cyclic loading phase.

A design procedure which introduces rational safety considerations for dynamically loaded foundations of gravity structures was presented by Foss, Dahlberg and Kvalstad (1978). Based on the knowledge of the cyclic behaviour of clays and with the support of results from centrifuge model tests (Andersen et.al., 1979) it seems likely that the failure mode in many cases of practical importance is horizontal sliding. The failure mode referred to is called "Failure in cyclic loading".

Let us consider a soil element under the centre of the platform at the skirt tip level. If this element is subjected to a sufficient number of load cycles of a given amplitude the effective stress path may reach the failure line. This development is accompanied by an increase in cyclic shear strain amplitude. Since the structure will have to move with the soil there is a limit as to which strains the structure can sustain. In fact, you may be very close to a failure in cyclic loading even if the displacements are small due to the special features of this type of failure.

The procedure proposed to design against failure in cyclic loading may be summarized as follows. Carry out cyclic laboratory tests for representative stress conditions and construct strain contour diagrams. The effects of random loading is then evaluated by using the so called "strain accumulation method" proposed by Andersen (1976),see Fig. 1. The response spectrum for our soil element is expressed in terms of the highest shear stress amplitude $\tau_{c,max}$ in the element during the storm and the cyclic shear stresses are assumed to be Rayleigh distributed. In the calculations the irregular amplitudes in the storm are arranged in blocks, each block representing wave loads of about equal amplitude.

Assume that the accumulated cyclic shear strain is calculated to be 2.1 % at the end of the storm. This gives one point on a stress-strain curve (curve 2 in Fig. 2), where the ordinate represents different sea states characterized by the parameter $\tau_{c,max}$. By varying the sea state, the shape of the curve can be established. This curve may be looked upon as a cyclic stress-strain curve for accumulated storm loading. Let us define the ultimate cyclic shear strength $\tau_{c,ult}$ as the assymptote to this cyclic stress strain curve. Due to the fact that failure in cyclic loading can occur during the course of a single storm with the same consequences as a quasi-static stability failure the same safety requirements shall apply in both types of analyses. As shown in Fig. 2 failure in cyclic loading will govern the design for shear strains γ_c equal to or less than γ_c^* .

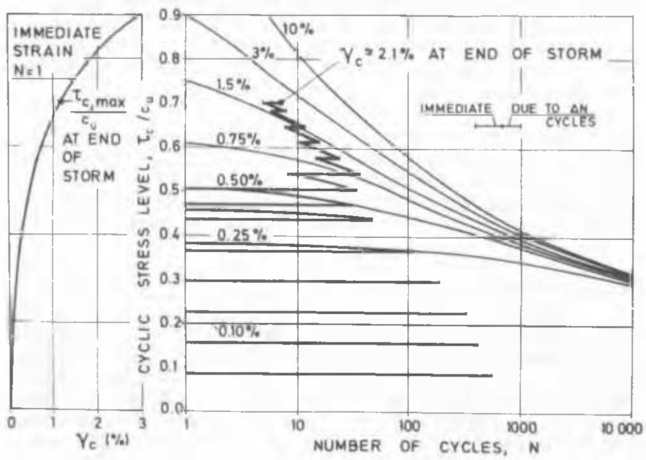

Fig. 1. Accumulated cyclic shear strains.

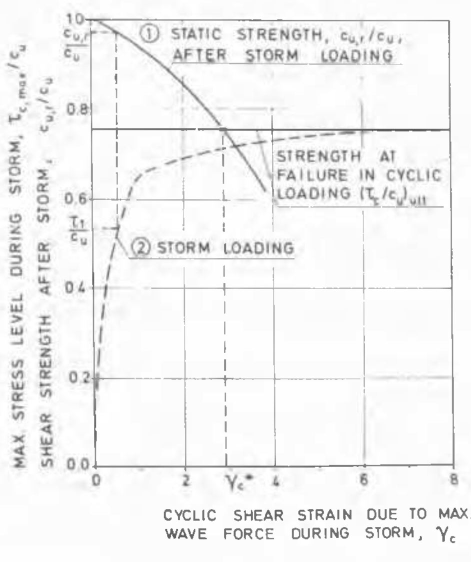

Fig. 2. Effects of cyclic loading on undrained shear strength

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B. Prange (Oral discussion)

STOCHASTIC EXCITATION OF ROCK CORES
Excitation Stochastique de Carottes Forées de Roches

In the Resonant-Column test (RC), the complex shear modulus is determined from the resonant frequency f_e and the damping ratio D of samples subjected to sinusoidal torsional vibrations. Digital signal analysis performed by computers enables the determination of these values immediately from stochastic excitation data, where the sample is subjected to a band of frequencies simultaneously. The calculation of the transferfunction (response-curve) of the RC-sample between the input (torsional moment) and the output (rotation) by a computer can be performed at very low signal-noise ratios and hence at extremely small shear strains.

Dissipation of rock materials is generally very low. The design of the RC-apparatus should therefore ascertain the smallest possible apparatus damping. To achieve very small wave radiation into the apparatus, a large inertia mass is provided under the sample cap resting on an axial ball bearing, where almost all wave energy is reflected back into the sample, fig.1. The remaining apparatus damping is of the order of 0.2% and can be calibrated.

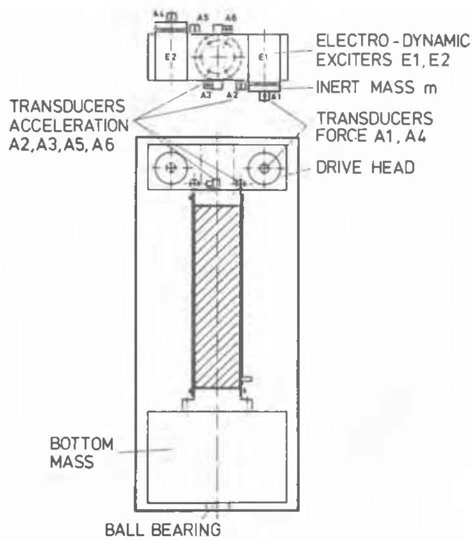

Fig.1 Resonant Column Test for Rock Cores
Schematic diagram of cross-section and top-view of drive-head

The drive-head consists of two electro-dynamic exciters operating against low-tuned inertia masses. The system is subjected to a pressure of max. 1 MN/m^2.

The transferfunction between tangential force (inert mass·acceleration; A_1,A_4) and tangential acceleration (A_2,A_3,A_5,A_6) is calculated by a digital signal analyzer HP 5420, which also provides the stochastic pulse-train for the excitation, fig.2.

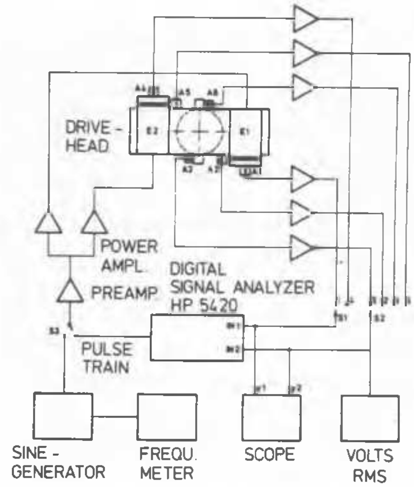

Fig.2 Electronic Components of RC-Test
(Stochastic Excitation)
S=Switch; E=Exciter; A=Accelerometer

The selection of different acceleration transducers ensures that only torsional vibration

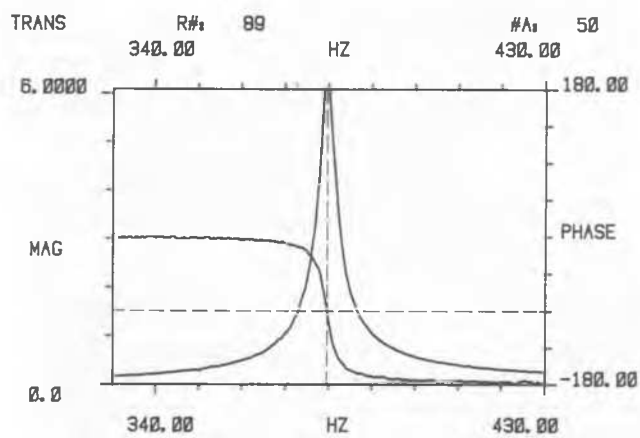

Fig.3 Computer Plot of Transfer Function
f_e = 379.61 Hz ; D = .45726%
(cursors f_e and -90° phase added)

modes of a sample are investigated in the band selective analysis. The shear strain level is checked by a slow RMS voltmeter. If desired, sinusoidal excitation can be used as well.

The calculation of the polar moments of inertia of drive-head and bottom-mass yielded the location of the plane of no rotation at 6% of the sample height consistent with experimental results during the calibration of the apparatus.

Different rock materials were tested from a site where the dynamic subsoil properties are investigated. The rock-cores were of 101 mm diameter and 200 to 400 mm height. To avoid any slippage between sample and caps the rock-cores were glued to the caps with epoxy raisin. The confining pressure p was varied between 0.1 and 1.0 MN/m^2, the effective shear strain γ between approx. 10^{-9} and 10^{-4}.

Fig.3 shows the results for a rock-core of gypsum for p = 0.2 MN/m^2 and $\gamma = 3.2 \cdot 10^{-7}$. It can be seen, that due to the small damping ratio the frequencies for peak magnitude and -90° phase shift are identical (cursors).

From the computer data f_e = 379.6 Hz and D = .46% the dynamic properties of gypsum for this pressure and strain level were calculated as:

G_{dyn} = 11 000 MN/m^2 ; D = .26% ; v_s = 2200 m/s

The calculation of G_{dyn} , D and v_s versus γ was performed on-line by a computer HP 9835.

K.R. Massarsch (Oral discussion)

DYNAMIC AND STATIC SHEAR MODULUS

The determination of stress-strain properties of soils is important for the correct analysis of geotechnical problems. Conventional geotechnical laboratory methods can measure soil modulus accurately only at strains larger than about 10^{-1} percent. Various dynamic field and laboratory methods have been developed which can measure dynamic soil properties over a large strain range. Fig. 1 shows in a semi-logarithmic diagram the stress-strain relationship of clayey sand determined by a resonant column test. Shear modulus decreased dramatically when a critical value of shear strain (about 10^{-3} percent) is exceeded. At 10-1 percent shear strain, when at a static test usually the first date point is obtained, the shear modulus is only about 30 percent of the initial value.

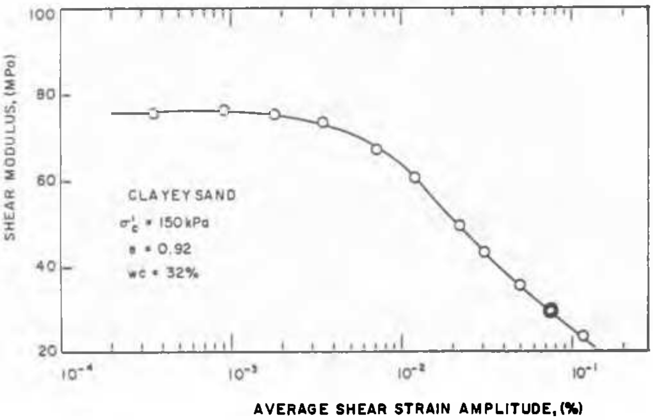

Fig. 1 Variation of secant shear modulus with shear strain determined from resonant column test.

The shear modulus determined from "seismic tests" is generally referred to as a "dynamic modulus". Its significance for static geotechnical problems is not yet generally appreciated. It can be shown, however, that at small shear strain, the "dynamic" shear modulus actually is determined at a strain rate which corresponds to static loading conditions.

The frequency at dynamic field or laboratory tests (e.g. cross-hole or resonant column tests) is for most soils in the order of 20 Hz. At small shear strains (10^{-4} percent) the average strain rate is thus about 0.5 percent/min. This strain rate is similar to that of conventional static laboratory tests.

Another important aspect of dynamic soil testing is that at small strain levels (smaller than about 10^{-4} percent shear strain) no volume change occurs and thus excess pore pressure does not develope. Thus, the "dynamic" shear modulus (maximum secant modulus) is the same for both the drained and undrained soil behaviour.

Hardin and Drnevich (1972) have proposed to normalize shear strain by "reference strain" γ_r, which corresponds to a value of shear strain at which in a stress-strain diagram the maximum shear modulus intersects maximum shear strength

$$\gamma_r = \frac{\tau_{max}}{G_{max}} \qquad (1)$$

The practical significance of reference strain becomes apparent when the inverse of reference strain is calculated

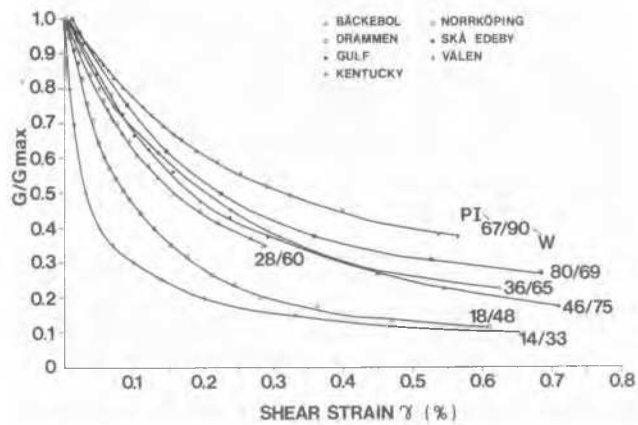

Fig.2 Variation of secant shear modulus with shear strain for different plasticity index, PI, determined by resonant column tests, Massarsch and Drnevich (1979).

$$\frac{1}{\gamma_r} = \frac{G_{max}}{\tau_{max}} \quad (2)$$

This term has been widely used by practising engineers to estimate the shear modulus from shear strength. Drnevich and Massarsch (1979) demonstrated that a narrow band of stress strain curves is obtained if shear strain is normalized using reference strain.

In Fig. 2 the variation of secant shear modulus with strain for soils with different plasticity index and water content is shown in arithmetic scale.

From Fig. 2 the shear modulus at 0.25 percent shear strain can be determined. In Table 1, a modulus reduction factor is shown which suggests that the reduction of shear modulus with shear strain is strongly influenced by the plasticity index, PI.

Table 1. Reduction of secant shear modulus at 0.25 percent shear strain

Plasticity index PI	Reduction factor, R (reduction to
10	0.15
30	0.30
50	0.40
70	0.50

Although the rate of strain at 0.25 percent shear strain at a resonant column test is higher than at a "static" test, it can be assumed that an equivalent static modulus can be calculated from dynamic tests and applying the proposed reduction factor

$$G_{static} = R \cdot G_{dyn} \quad (3)$$

The rapid development of testing methods in soil dynamics and earthquake engineering has provided extensive data on the stress-strain behaviour of soils. This valuable information has not yet been applied by many geotechnical engineers.

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J. Biarez, P. Cousin and P. Dowlatyari (Oral discussion)

FILM SUR LA COMPARAISON DES CALCULS PAR ELEMENTS FINIS EN DYNAMIQUE

Film showing a Comparison of Finite Elements Computations in Dynamic Mode

Le film projeté a montré la cinématique des grandes déformations ("rupture") pour des écrans de soutènements, fondations et talus dans un milieu sans cohésion.(Fig. 1). Le but de cette recherche est de contrôler la validité des méthodes de calculs par éléments finis ou équations intégrales selon les complications de la loi de comportement utilisée. Le modèle bidimensionnel permet la comparaison des déformations et des accélérations en chaque point ainsi que des mesures de contraintes sur le contour.
Le calcul élastique non linéaire donne une approximation acceptable des contraintes sur les parois verticales fixes du massif où les déformations sont petites (Fig. 2) et un ordre de grandeur logique pour les accélérations dans le massif pour des valeurs assez petites; au delà de 0,3 g pour le sol sous la fondation et 0, 1 g pour le sommet de la fondation, le calcul par éléments finis avec plasticité est nécessaire pour expliquer le changement de comportement. Un exemple de ce calcul a été donné pour les parois moulées dans la communication écrite.

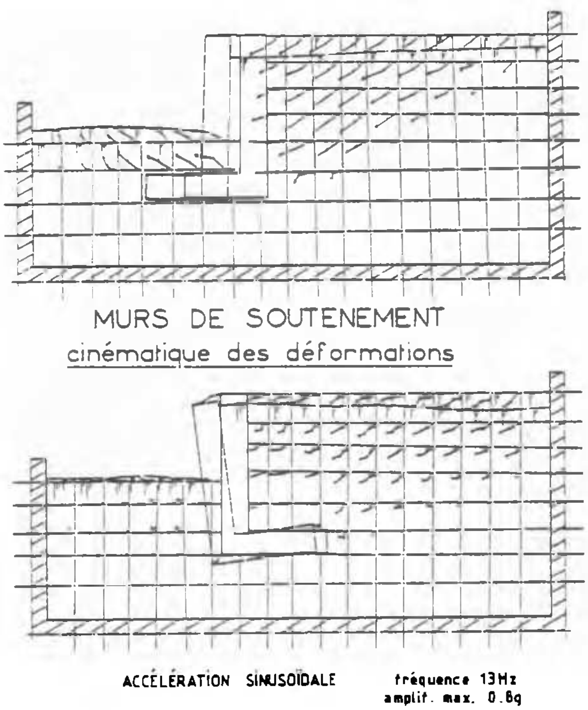

fig.1 -

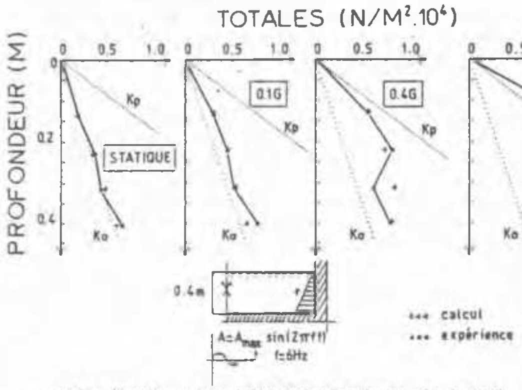

Fig. 2 - Variations de contrainte horizontale le long de la paroi latérale (obtenue par l'expérience et le calcul pour A_{max} = 0.1g, 0.4g et 0.8g

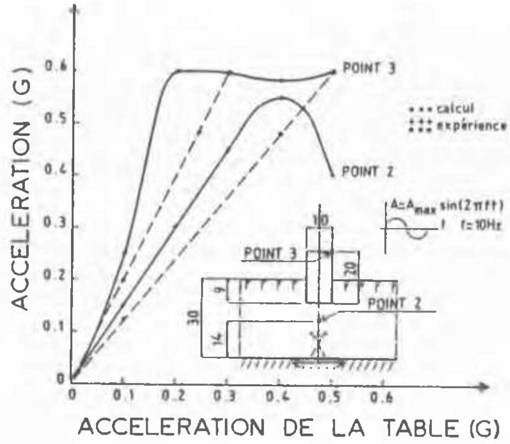

Fig.3 -

B.S. Browzin (Oral discussion)

IMPORTANCE OF RESEARCH IN SOIL DYNAMICS AND SOIL STRUCTURE INTERACTION FOR NUCLEAR LICENSING AND DEVELOPMENT
Importance des Recherches en Dynamique des Sol et du Comportement des Structures en vue des Licenses et du Dévelopment des Industries Nucléaires

Because the seismic load is governing for the design of nuclear power plant containments (cylindrical shelves with a dome, typically with a 150 ft. diameter, 200 ft. height and a wall thickness of 3.5 ft.) and other structures, it is a most urgent technical problem to provide methodology for determining characteristics at nuclear power plant sites depending on the distance from the epicenter. When seismic wave characteristics are obtained at the site, transfer functions must be known or assumed in order to analyze stresses in structures and displacements for adequate performance of the equipment. The same applies to the reactor. Stress and strain at a given location must provide a safety design, but not a conservative one. Safety and economics are both necessary considerations. Safety requirements for nuclear power plants are specific, not encountered in any other industry. For this reason, soil dynamics research and studies of dynamic soil-structure interaction are important. This is well realized by designers of the nuclear power facilities. However, the geotechnical engineering community, as evidenced by this Conference, is not well aware of the soil dynamics research needs for the nuclear industry. This Conference has only two papers on this subject: one French paper on pile foundation for a nuclear power plant and one German paper from T. H. Karlsruhe on gipsum properties tested for a nuclear power plant foundation.

Consequently, it is suggested that, at the San Francisco XI ICSMFE Conference of 1985, a session be included on Nuclear Power Plant Foundations for the purpose of bringing together scientists and engineers working on nuclear power licensing and development, particularly those working as contractors for the U.S. Nuclear Regulatory Commission.

D. Weiner (Oral discussion)

NEW ASPECTS OF MACHINE FOUNDATION DESIGN - A CASE RECORD

Too many practising designers believe that the use of expansion joints eliminates the spreading of vibrations from machine foundations to adjoining structures. In practice, despite the use of expansion joints, severe vibrations can spread via the foundation and soil to the adjoining building (Fig. 1).

These vibrations can damage machinery and the building and its foundations, they can damage or interrupt production, and they can cause discomfort in working localities.

The "golden" rule of always arranging expansion joints between foundations and the adjoining structure was created by the pioneers in the machine foundation sphere, Rausch (1959, 1968) and Barkan (1962). This use of ex-

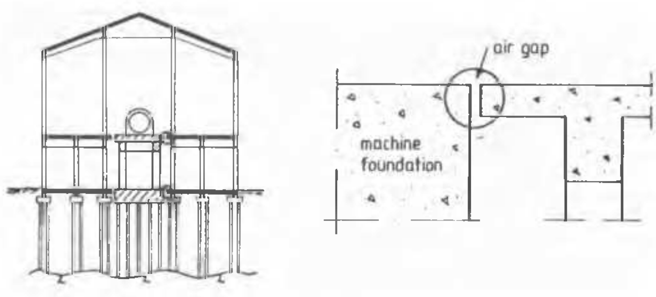

Fig. 1 Section of building (left) and detail of expansion joint (right) between machine foundation and floor.

pansion joints became in time a principle which

- was stipulated in several machine foundation standards (DIN 4024, 1955)
- was recommended by machinery suppliers, etc.
- was quoted in books on machine foundations, Major (1962), Srinivasulu & Vaidyanathan (1978).

Practical example

This example concerns a gangsaw foundation resting on point-bearing piles. Fig. 2 shows a five year old sawmill, one of the largest in Europe, with four separate and identical foundations, each with a volume of 190 m^3 of concrete.

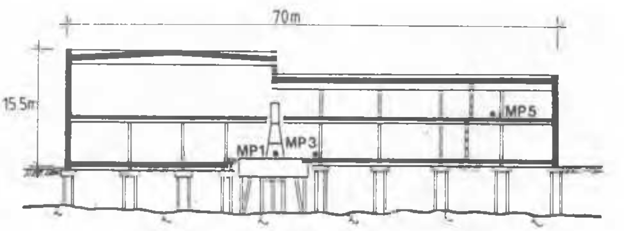

Fig. 2 Section of the sawmill showing measuring points on the gangsaw foundation (MP1), on the lower floor (MP3) and on the upper floor in the office (MP5).

Fig. 2 shows one of the four gangsaws. Each gangsaw foundation is supported on 22 precast concrete piles. The lower floor and sawmill building are also supported on point-bearing piles. The speed of the gangsaw flywheel is usually 360 RPM, i.e. 6 Hz.

As can be seen in Table 1, the introduction of a force-transmitting expansion joint (FTEJ) between the machine foundation and the adjoining structure has reduced the vibration level in the whole vibration system, i.e. in the vibration source and the adjoining structures.

In this case the force-transmitting expansion joints were only intended to transmit horizontal forces. The reduction in vertical vibrations in the adjoining structure was due to reduction of rocking vibrations in the machine foundation.

Before the force-transmitting expansion joint was introduced in normal production it was tested by transient

Table 1. Amplitude of vibrations using conventional expansion joint (CEJ) and after the introduction of force-transmitting expansion joint (FTEJ) between gangsaw foundation (9 x 4 x 2.5 m) and floor.

Measuring point	Measuring direction	Velocity amplitude $\hat{v}$, mm/s CEJ	FTEJ
Mp 1	horiz.	37.9	5.7
MP 3	horiz.	1.5	0.5
	vert.	1.5	0.5
MP 5	horiz.	1.8	0.6

excitation. The response spectrum showed that the natural frequency of the foundation increased from 9.2 Hz to 12.8 Hz.

Conclusions

By using force-transmitting expansion joints the dynamic stiffness of the machine foundation can be increased, and one can as a result, when necessary

- reduce the area and/or volume of the machine foundation
- reduce the number of structural elements (piles, columns, beams) in the machine foundation and/or the adjoining structure
- reduce vibration levels in the machine foundation and in some cases also in the adjoining building as, for example, in the case described.

From the economic point of view force-transmitting expansion joints are especially suitable for foundations with large horizontal and at the same time small vertical forces (also for constructions exposed to earth quakes).

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W.D.L. Finn, Co-Chairman

CONCLUDING REMARKS AFTER FLOOR DISCUSSION

The papers presented to the main session and the floor discussions describe, for the most part, the mainstream of current geotechnical engineering practice in the areas of soil dynamics and earthquake engineering. They do not illustrate the major developments since the last international conference in Tokyo in 1977 nor do they suggest the directions of future development. Before bringing this session to a close, I would like to make a few comments on these topics.

Soil Dynamics, as we know it today, began developing under the stimulus of the challenging problems posed by the earthquakes in Niigata, Japan and in Alaska in 1964, especially those associated with liquefaction. Development was nurtured until the mid-1970's by the demands of the nuclear power industry for acceptable analytical methods for predicting structural and foundation response during earthquakes and for reliable test procedures to determine the parameters required by the analytical methods. A different stimulus operates today. Now the greatest demands on the skill and judgement of the geotechnical engineer are being made by the offshore industry. The foremost problems are those associated with the cyclic loading effects of waves and earthquakes on soils and foundations of offshore structures.

Offshore engineering is developing so rapidly that the geotechnical engineer has a very limited fund of experience of prototype behaviour to draw upon when exercising his judgement. Jacket structures are now being designed for water depths of over 300 m, new types of structures and foundations are evolving rapidly, and unusual problems such as the cyclic loading response of offshore piles in friable materials such as calcareous or volcanic sands are emerging. Many of the recent developments in soil dynamics have been in response to the challenges of offshore engineering.

Developments since 1977 may be classified under 5 headings:

(i) Correlation studies of field experience;

(ii) Analytical methods;

(iii) Laboratory testing;

(iv) In-situ testing;

(v) Offshore engineering.

I will attempt to indicate a few of the important developments in each of these areas and suggest the directions of future developments.

Correlation Studies

Over the past few years, major efforts have been made to assess critically the large body of data on response of ground and earth structures to earthquake excitation and to draw general conclusions and guidelines to assist the geotechnical engineering profession in arriving at sound judgements, without always having to resort to sophisticated testing and analytical computations. Two contributions which have had a major impact on engineering practice are the liquefaction assessment chart by Seed (1979) and the guidelines as to the probable performance of earth dams during earthquakes by Seed, Makdisi and De Alba (1977). Also notable are the studies by Youd and Perkins (1978) to establish criteria for regional assessments of liquefaction potential, using broad scale geological and seismological parameters.

Analytical Methods

The major development has been the emergence and acceptance of dynamic non-linear effective stress analysis. Effective stress concepts of soil behaviour have long been recognised as essential for a fundamental understanding of soil behaviour. Their use had been limited to static behaviour because of the problems associated with predicting porewater pressures under cyclic loading conditions. The development of a porewater pressure model by Martin, Finn and Seed (1975) which could be coupled with a non-linear hysteretic model of soil behaviour (Finn, Lee and Martin, 1977) led to the emergence of non-linear effective stress methods of analysis (Lee and Finn, 1975, 1978). Comparative studies of non-linear effective stress methods, total stress methods and the commonly used iterative elastic methods of dynamic have been conducted by Finn, Martin and Lee (1978). The studies generally substantiate the desirability of using non-linear effective stress methods where possible.

In the last few years, other methods of non-linear effective stress analysis have been developed. Methods by Ishihara and Towhata (1980), Ghaboussi and Dikmen (1978), Zienkiewicz, Chang and Hinton (1978) have been critically reviewed by Finn (1979). More recently, Martin and Seed (1979) developed a model, MASH, which they showed to give results very similar to the DESRA models. A two-dimensional non-linear hysteretic total stress method, TARA-1, has been developed by Siddharthan and Finn (1981) and will be available shortly in an effective stress option, TARA-2.

Non-linear effective stress analyses have been used in the last 2 years in engineering practice to predict surface motions, liquefaction potential, ground motions and porewater pressures for input into analyses of cyclic lateral motions of piles and stiffness degradation studies in saturated sands.

The acceptance of one-dimensional non-linear hysteretic effective stress models has created a demand for two-dimensional models. I would expect a number of such models to be developed in the next few years.

Laboratory Testing

The major change in the last few years has been the spread of cyclic simple shear testing from the research laboratories to commercial testing establishments. The test models many features of cyclic response in the field and its use will continue to grow.

Studies by Martin, Finn and Seed (1978) showed that system compliance could introduce very significant errors into the results of undrained cyclic loading tests and lead to a gross overestimation of liquefaction resistance. To avoid compliance problems, Finn, Vaid and Bhatia (1978) designed a very low compliance (so called constant volume) cyclic simple shear test and provided quantitative data on the effects of compliance.

The work of Moussa (1975) and Pickering (1973), as well as the above study, demonstrated the superiority and greater ease of constant volume testing. In the last few years, some of the major commercial testing laboratories have acquired the capability to conduct constant volume cyclic simple shear tests and I would expect this trend to continue.

In Situ Testing

The most significant developments in relation to the measurement of *in situ* dynamic properties of soils have been in attempts to measure such properties as shear modulus and damping at larger strains than the conventional 10^{-4}-10^{-3}%. These developments are mainly in the hands of commercial firms and details on progress are not readily available. Finn (1977) surveyed new emerging techniques in this area. Because of the difficulty in getting good samples of sand and maintaining small strain stiffness during handling and testing, there will always be a strong incentive to develop reliable *in situ* methods to determine the dynamic properties of sands and silts over wide strain ranges. However, due to expense of field work in this area and the cost of associated equipment, research and development will be in the hands of relatively few organisations.

Offshore Engineering

I have already discussed the key role being played by the offshore industry in stimulating developments in the geotechnical aspects of soil dynamics and liquefaction potential. Here, I will simply list some of the major problems related to soil dynamics for which better solutions are needed by the offshore industry:

(i) Lateral and axial response of long piles to cyclic loading taking into account the development of residual porewater pressures and true non-linear behaviour. These problems are especially difficult in friable materials such as calcareous or volcanic sands;

(ii) Vertical settlements and horizontal displacements of gravity structures under cyclic loading;

(iii) Soil-structure interaction analyses for platforms under wave and earthquake loading, especially pile-supported platforms;

(iv) Development of remote-controlled in situ testing procedures;

(v) Effect of hurricane associated waves on soft bottom sediments;

(vi) Underwater slope displacements under earthquake loading.

These problems challenge every aspect of our capability to handle problems in soil dynamics and cyclic loading; field investigation, laboratory and in situ testing, modelling of soil properties and analytical and computational capacity. They ensure that soil dynamics will continue to challenge the skill and judgement of the geotechnical engineer for many years to come and that it will provide a field of endeavour rich in potential for professional growth and accomplishment.

It is now time to bring this session to a close. Before doing so I wish to express my great appreciation to the floor discussers and panellists for a very stimulating and enjoyable afternoon. Quite apart from good technical content, the floor discussions were remarkable for the clarity of the presentations and the strict adherence to time allotments. This cooperation from the discussers made my duties very easy and did much to creat an atmosphere conducive to lively and interesting discussion. Finally, I wish to thank the organizing committee for the honour of co-chairing the main session on soil dynamics.

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V.A. Ilyichev, Chairman

CLOSURE OF SESSION

Our Session has come to the end. We have heard interesting reports and received new information on the problem of liquefaction of soils, on the determination of soils properties, being used in dynamic calculation, and other problems.

The work of our Session could not be successful without efforts of officers of the Session and I am grateful to them for the work they have done and for the interesting reports.

I believe that I express the opinion of all participants of the Session to thank the Organizing Committee and all those persons and organizations sponsoring this Session. I would like to thank all those who were responsible for the excellent arrangements for our Session and for the hospitality.

J.-L. Boelle. J. Biarez and J. Meunier (Written discussion)

MESURES SILMUTANEES DU MODULE D'YOUNG ET DU COEFFICIENT DE POISSON EN PETITES DEFORMATIONS
Silmultaneous Measurements of Young's Modulus and Poisson's Coefficient for Small Strains

Le but de cet essai est de mesurer simultanément en petites déformations ($\varepsilon \leqslant 10^{-6}$) les paramètres de l'élasticité linéaire isotrope E et ν des sols par l'interprétation des résonnances axiales successives d'un échantillon cylindrique libre à ses deux extrémités (nous avons admis que pour de très faibles variations de contraintes autour d'un état initial donné, la loi de comportement d'un sol pouvait être assimilée à une loi élastique linéaire).

La possibilité de mesure simultanée est due à la décroissance de la vitesse de phase lors des résonnances axiales successives (relation de dispersion). Ce phénomène qui a été étudié par Pocchammer pour un échantillon de longueur infinie, a donné lieu à plusieurs relations approchées, la plus simple d'entre elles étant due à Love:

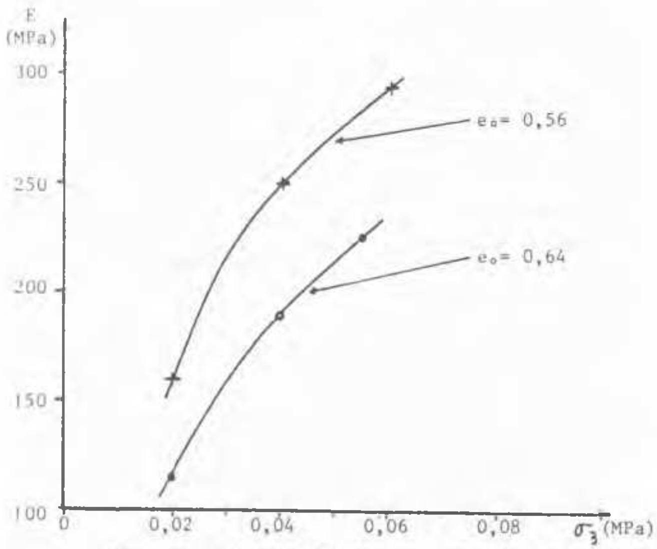

Figure 1 : Module d'Young en fonction de la contrainte isotrope σ_3 pour deux valeurs de l'indice des vides e_o.

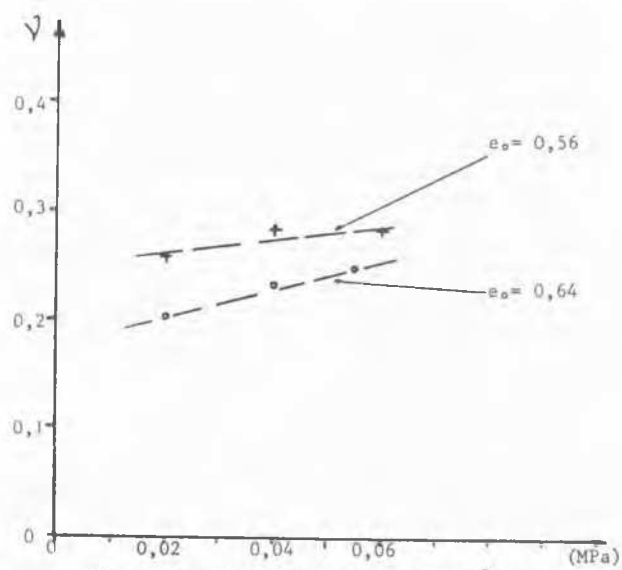

Figure 2 : Coefficient de Poisson ν.

$$c^2 = c_L^2 - \frac{1}{2}\nu^2 a^2 \omega^2$$

c : vitesse de phase
$\omega = 2\pi f$, où f est la fréquence
a : rayon de l'échantillon
ν : coefficient de Poisson
$c_L = \sqrt{\frac{E}{\rho}}$, où E est le module d'Young et ρ la masse volumique

Résultats obtenus

Actuellement seuls des essais sur matériaux pulvérulents secs ont été réalisés; la contrainte isotrope σ_3 appliquée sur l'échantillon étant inférieure à 0,06 MPa.

Il apparait que les valeurs du module d'Young peuvent être approchées par une loi du type:

$$E = A\ (\sigma_3)^n$$

avec n = 0,63 pour un sable de granulométrie serrée (d_{60} = 2,20 mm) et $\sigma_3 < 0{,}06$ MPa.

Comme le montre la figure 1, A est une fonction de l'indice des vides initial e_o du matériau.

Les valeurs du coefficient de Poisson qui présentent une légère croissance avec la contrainte isotrope se situent entre 0,2 et 0,3 (voir figure 2).

R.P. Chapuis (Written discussion)

VOID RATIO, POOR REPRESENTATION OF THE STATE OF PACKING
L'Indice des Vides, Mauvaise Image de la Structure Interne

The dynamic soil properties which must be evaluated for design purposes are usually assumed to depend on the void ratio and the mean effective stress. It is well-known however that in situ deposition of soils results in an anisotropic arrangement of particles which should be reflected in the shear modulus and damping characteristics. It is known also that the resistance of saturated sands to liquefaction depends on their state of packing which will influence their behavior even when the relative density is the same.

The purpose of this discussion is to present recent results related to the description of the state of packing, with emphasis placed upon the critical density concept, which may be useful for further research in soil dynamics.

After developing statistical definitions of stress and moment stress tensors, obtained from intergranular forces and geometric characteristics (Chapuis, 1976), it appeared that the spatial distribution of the normals to contact planes was acting as the internal structural link between stress and strain. In fact the contact points are simultaneously transmitting forces (stress at macroscopic level) and controlling relative movements (strain at macroscopic level).

For a better understanding of the internal structure of granular materials, a special equipment was designed to perform plane compression or distortion tests on bidimensional stackings of coaxial cylinders (four diameters). Many new microscopic results (Chapuis, 1976) were obtained concerning : the mean number of contacts per grain; the individual and mean rotations; the creation, disparition and displacement of contact points; the spatial distribution of the normals to the contact planes described by the internal structure tensor. Most of these results were presented in a recent paper by Chapuis and Soulié (1981).

Those shown on Fig.1 contain interesting and unusual aspects.

During the first loading (OA), a dense behavior is observed. After reaching the isochoric (constant volume) stage of deformation, it is possible to unload while $\sigma_H = \sigma_2$ remains constant. Then a sharp bend is observed on the curve (B) when there is an inversion of major and minor

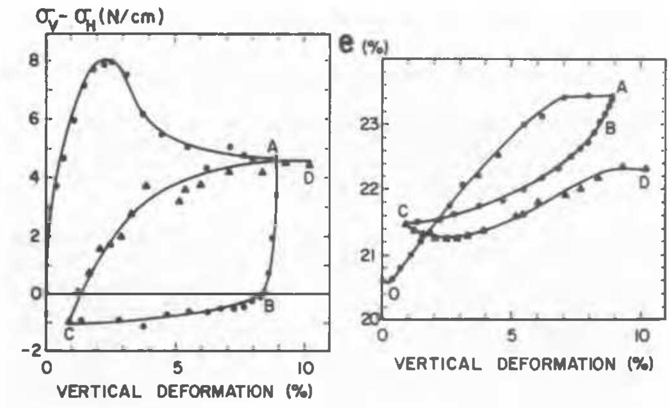

Fig.1 Results of a biaxial compression test

stresses; no peculiar behavior is noticed on the e vs ε_V curve during the stress inversion. After the unloading phase, the sample is loaded again and a loose behavior is registered (CD). It is worth noting that the void ratio during the second isochoric stage is quite different from the one obtained in the first loading (A). This invalidates the well-accepted notion that e_{cv} is only a function of the mean stress, and gives an evidence that it depends also on the initial state of anisotropy.

This was confirmed by microscopic results which may be summarized as follows :

a) The spatial distribution of the normals to contact planes may be represented by an internal structure tensor S, the parameters of which have been determined for various tests. The first invariant of S represents the mean number of contacts per grain.
b) During all analyzed biaxial compression tests, it was registered that the principal directions of structure coincide with those of stress and strain; the fact that the sample was prepared by gravity and that the vertical is a principal axis of σ and ε probably influenced this result.
c) During a test on a dense packing, the mean number of contacts per grain drops markedly after the peak of stress, and then remains approximately constant until the stage of isochoric deformation is reached, where the internal structure is invariant.

d) The ratio S_1/S_2 of principal values, which characterizes the structural anisotropy, is a constant for isochoric deformation of initially dense or loose packings, although the void ratios may have different values.
e) There is no direct relationship between e and the mean number of contacts per grain.

These experimental results have shown that many opinions about interlocking, state of packing, flow structure, critical density... should be reviewed according to the notion of the internal structure tensor. When a soil undergoes liquefaction, which is an example of isochoric deformation, its structural anisotropy is constant. In fact such flow occurs when the internal structure becomes unable to adapt itself to the sollicitation and can only reproduce itself identically. It should be understood that the tensor internal structure is acting as a permanent memory, and that the scalar void ratio is a poor representation of the state of packing. Suggestions on how to handle this tensor S and introduce it in constitutive equations, may be found in the paper by Chapuis and Soulié (1981). This theoretical problem is waiting for solutions, as the practical problem of measuring the in-situ internal structure parameters of a soil.

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D. Lefebvre and J.L. Lecoufle (Written discussion)

VALIDATION D'UNE METHODE DES FONDATIONS DE MACHINES VIBRANTES PAR DES ESSAIS IN-SITU EN GRANDEUR REELLE
Validation of a Calculation Method of Vibrating Foundations by In-Situ Full-Size Tests

Les calculs visco-élastiques par éléments finis conduisent à la définition d'impédances réduites du sol sous les fondations de machines vibrantes, pour différents modèles de sols. A l'aide de ces impédances et d'une interprétation par paramètres concentrés, les courbes de réponse de ces fondations peuvent être calculées. Une étude in-situ en vraie grandeur a permis de contrôler ces calculs par comparaison des courbes de réponse calculées et des courbes de réponse mesurées sur deux cas de fondations et sur trois types de sols (sable de Fontainebleau, argile de Provins, gravier de Petite Seine).

INTRODUCTION :
Pour contrôler le calcul par éléments finis des impédances réduites du sol sous les fondations de machines vibrantes, les courbes de réponse ont été calculées à partir de ces impédances et comparées aux courbes de réponse de fondations en vraie grandeur (1).

EXCITATION :
L'excitation est un "bruit blanc 10 Hz - 80 Hz", généré par un vérin hydraulique asservi; on peut choisir le niveau de force F imposé à la fondation. Deux cas de fondations ont été étudiés (1 x 1 x 0,4 m^3 et 2,5 x 3 x 0,5 m^3). Deux directions d'excitation ont été considérées : verticale et horizontale (tamis-roulis).

PARAMETRES DU SOL :
Les modules à entrer dans les calculs sont des modules moyens calculés à partir des mesures "cross-hole". A chaque fréquence, une "profondeur de pénétration des ondes dans le sol" est définie en fonction du mouvement généré (vertical, tamis-roulis). Suivant la valeur de cette profondeur et la lithologie du terrain, un modèle de calcul est choisi (modèle semi-infini, couche sur substratum rigide, bi-couche, remblai en cuve) et une valeur moyenne de module est attribuée à chaque couche du modèle. La non linéarité du sol est prise en compte par un calcul itératif fondé sur la connaissance des courbes $G = G(\gamma)$ mesurées au triaxial en petites déformations.

CALCUL :
Les impédances réduites du sol k_{ij} et c_{ij} à la base de la fondation sont fournies par un calcul visco-élastique par éléments finis avec 12 éléments sous la base de la fondation et avec amortisseurs aux frontières (2). Des abaques $k_{ij} = k_{ij}(a_0)$ et $c_{ij} = c_{ij}(a_0)$ (avec $a_0 = 2\pi f r_0 \sqrt{\frac{G}{\rho}}$) sont fournis. Les impédances globales K_{ij} et C_{ij} (paramètres concentrés) sont données en fonction des impédances réduites, des propriétés mécaniques de la première couche de sol et de la surface de base de la fondation.

CONCLUSION :
Pour les mouvements vertical et de tamis-roulis, le calcul des courbes de réponse fait l'objet d'un programme sur calculatrice TI-59. La fiabilité de ce programme est bonne en excitation verticale : voir figure. Au contraire, le cas du mouvement couplé de tamis-roulis ne peut être retrouvé de façon satisfaisante, à moins d'opérer une réduction importante des modules, due vraisemblablement à l'anisotropie du sol.

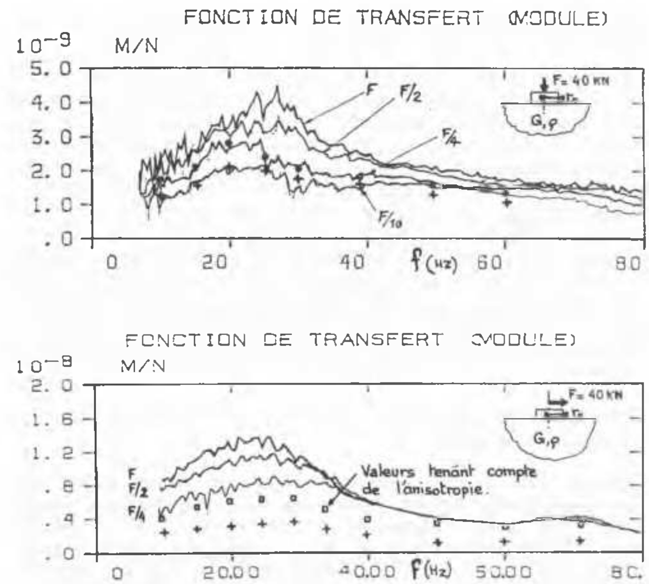

Fig. 1

Courbes de réponse (amplitude de déplacement sur force excitatrice) d'une fondation rectangulaire 2,5 x 3 x 0,5 m^3 posée sur un sable de Fontainebleau sec (E_{sism} = 200 MPa). Excitation verticale et horizontale.

∿∿ essais pour plusieurs niveaux d'excitation (influence de la non-linéarité)

+ + + calcul avec $G = G_{sism}$

∘ ∘ ∘ calcul avec $G = G(\gamma)$ (correction de non-linéarité)

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T. Matsui and N. Abe (Written discussion)

CYCLIC BEHAVIOR OF SATURATED CLAY

We wish to discuss the cyclic behavior of saturated clay from viewpoint of effective stress, comparing with that of saturated sand. In order to discuss this point, it is essential to accurately measure the cyclic pore water pressure (Matsui, Ohara and Ito, 1980). Generally speaking, the distribution of the excess pore pressure is not uniform in the triaxial specimen, because of the end restraint. In the conventional method, to avoid this difficulty of uneven pore pressure measurement, the axial load is applied very slowly to the clay specimen. However, we usually observe waves of around 1 Hz of frequency for earthquakes, around 0.1 Hz for ocean waves. Therefore, it is necessary to accurately measure the cyclic pore pressure within a clay specimen subjected to such higher frequency loadings, for quantitative discussions of the behaviors.

For this porpose, we have developed a reliable measuring technique of the cyclic pore pressure, in which a small miniature transducer was inserted at the center of a triaxial clay specimen (Matsui and Abe, 1981). The size of the transducer is 5 mm in diameter and 13.5 mm long, and that of the specimen is 50 mm in diameter and 125 mm long. As the diafragm of this transducer is located at almost near the tip, it was confirmed that its response within the specimen was very well for an isotropic cyclic pressure (Matsui and Abe, 1981).

In the following, are shown some results of cyclic tests, in which compression and extension stresses are alternately applied to a specimen under a constant mean total principal stress. Fig.1 shows the behavior of cumulative excess pore pressure, which is represented by the value at no shear stress and is normalized by the effective consolidation pressure. In this figure, it is seen that the excess pore pressure for each type of soil increases almost linearly with increasing number of cycles, regardless of such soil types as dense and loose sands or normally consolidated clay, and then approaches to the confining pressure. Fig.2 shows the behavior of peak-to-peak axial strain. Those for clay and loose sand do not significantly increase at the early stage of number of cycles, but increase rapidly after a certain number of cycle, while the axial strain for dense sand increases more rapidly at the early stage, and then the increasing rate gradually decreases. Even after around 30 cycles, in which the excess pore pressure reaches the confining pressure, the strain does not rapidly increases, that is, cyclic mobility may occur.

Comparing qualitatively cyclic behaviors of saturated clay and sand, those of excess pore pressure and axial strain of normally consolidated clay are similar to those of saturated

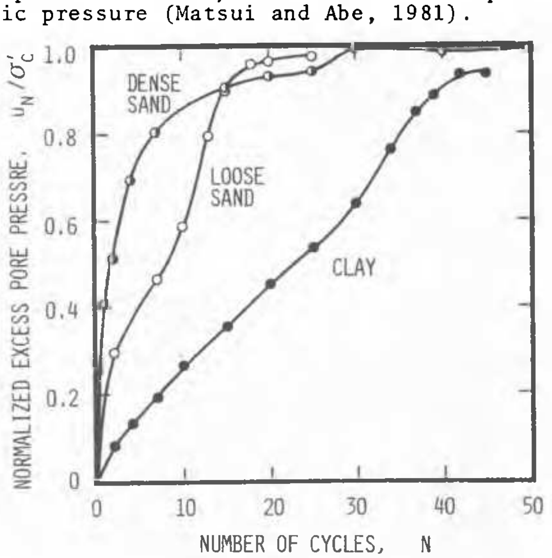

Fig.1 Behavior of Cumulative Excess Pore Pressure

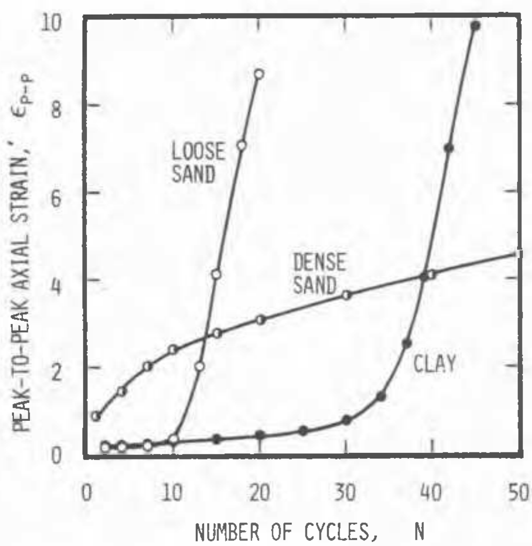

Fig.2 Behavior of Peak-to-Peak Axial Strain

loose sand, although they are different in mechanism from those of saturated dense sand. Paying attention to cyclic behaviors of a soil element, those facts described above may substantiate that even in the saturated clay the similar phenomenon as in saturated loose sand can occur. However, this does not immediately imply that liquefaction in a saturated clayey ground occurs as the same as in saturated sand ground.
To reach a conclusion on this matter, we should be furthermore discuss the effects of the low permeability of clay and the intrinsic cohesion between clay particles on cyclic behaviors of clayey ground.

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